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# Estimation of Seismic Response Parameters and Capacity of Irregular Tunnel-Form Buildings

**Vahid Mohsenian[1], Ali Nikkhoo[2*], Iman Hajirasouliha[3]**

1. Postgraduate Researcher, Department of Civil Engineering, University of Science and Culture, Tehran, Iran.
2. Associate Professor, Department of Civil Engineering, University of Science and Culture, Tehran, Iran.
3. Associate Professor, Department of Civil and Structural Engineering, University of Sheffield, Sheffield, UK.

*** Corresponding author**

Associate Professor,

Department of Civil Engineering,

University of Science and Culture, Tehran, IRAN.

Tel and Fax: +98 2142863000

Email: nikkhoo@usc.ac.ir

## Abstract

Insufficient information about the seismic performance of tunnel-form buildings and limited relevant design codes and standards are the main barriers towards application of these systems in seismically active areas. Vertical and horizontal irregularity of typical tunnel-form buildings is another cumbersome challenge restricting the application of these systems. To address these issues, this study aims to evaluate the seismic behaviour of tunnel-form buildings with horizontal irregularity and develop appropriate design methodologies. Based on the results of 3, 5, 7 and 10-storey buildings, new response modification factors are proposed as a function of seismic demand and expected performance level. Fragility curves are also derived for various levels of intensity, and simple equations are introduced to estimate uncoupled frequency ratios. The results, in general, demonstrate the flexible torsional behaviour of irregular tunnel-form structures and their adequate seismic resistance capacity. The buildings studied herein, managed to satisfy the Immediate Occupancy (IO) performance requirements under design-basis earthquake, which implies that the plan regularity requirement for tunnel-form buildings in seismic codes may be too conservative. Moreover, it is concluded that using response modification factor equal to 5 can generally result in sufficient stability and adequate performance level under both design basis and maximum considered earthquake scenarios.

**Keywords**: Tunnel-Form Structural System, Irregularity, Response Modification Factor, Fragility Analysis, Uncoupled Frequencies Ratio.

## Introduction

The modern construction industry is quickly moving towards more efficient structural systems and technologies to reduce costs, constructional time and human resources, and also to promote the quality and safety of the structures under extreme loading events such as strong earthquakes. In this respect, the newly-developed tunnel-form structural systems can offer several advantages such as competent capability for planning, shortening the construction time and consequently leading to a rapid asset return. In the tunnel-form structures, slab and wall elements are employed as the main lateral and vertical load-carrying systems, and the beam and column elements commonly used in typical structural systems are excluded. Moreover, since the walls and slabs are simultaneously constructed in each storey, there is no need to use cold joints to ensure an integrated 3D performance of the system during a seismic event. The considerable length of wall elements in this system, helps to prevent stress concentrations at wall to slab connections, which are usually observed in common beam-column systems. In addition, tunnel-form structures generally can provide a good level of resilient under extreme load conditions. This is confirmed by the observations from Kocaeli ($M_w$=7.4) and Duzce ($M_w$=7.2) earthquakes, where most tunnel-form buildings managed to withstand the strong earthquake excitations and generally performed better than other commonly used RC systems (Balkaya and Kalkan 2004a).

Due to the above mentioned advantages, this type of structural system is increasingly become popular especially for mass construction projects in seismically active areas. Despite extensive use of these structures, the available codes and standards do not consider them as independent structural systems. Moreover, very limited studies have been conducted to investigate the seismic performance of these systems. In the following, some of the most notable studies including their outcomes are briefly presented.

Previous studies on the behaviour of tunnel-form buildings, have demonstrated that the empirical equations for calculation of fundamental period in current design guidelines, do not generally yield to accurate predictions. This can result in improper estimation of the earthquake-induced loads for tunnel-form buildings (Goel and Chopra 1998; Lee et al. 2000). To address this issue, through a number of eigenvalue analyses on reinforced concrete (RC) buildings with different plans and number of storeys, Balkaya and Kalkan (2003a) proposed a new equation to acceptably estimate the fundamental period of tunnel-form buildings. Based on the outcomes of their analyses, in most cases, torsional modes were precedent to the translational ones. Due to the complexity and limitations of their proposed relationship, in a follow-up study they attempted to develop another equation which was direction-independent (Balkaya and Kalkan 2004a).

In another relevant study, Balkaya and Kalkan (2003b; 2004b) carried out pushover analysis on 2 and 5-storey tunnel-form buildings with the same plan and found the 3D membrane action as the dominant mechanism for tunnel-form buildings. They concluded that the 3D coupled tension-compression performance, plays an important role in load-carrying capacity of these systems. Moreover, the structures analyzed in their research, managed to meet the requirements of the Turkish Seismic Design Code at the performance level of immediate occupancy (IO). Based on the analytical results, they proposed to utilize response modification factor (R) of 5 and 4 for shorter and taller tunnel-form buildings, respectively.

To investigate the nonlinear seismic behaviour of tunnel-form buildings, Tavafoghi and Eshghi (2005) carried out studies on two 1-5 scale specimens. During the cyclic lateral loading process, a brittle behaviour was observed. The structural damages were mainly developed in the slabs as well as the slab to wall and wall to foundation connections. The forced vibration tests also indicated that the cracks developed in the slabs clearly affected the period of the first vibration mode. Based on their findings, the response modification factor of 4 was suggested to be a reasonable value for these systems.

Yuksel and Kalkan (2007) carried out a number of experimental tests on intersecting walls under lateral cyclic pseudo-static loads at both principal directions. Although their tested specimens had minimum percentage of longitudinal reinforcement, they exhibited a brittle shear failure. Subsequently, a verification study was performed to analyse models with different percentage of longitudinal bars. The results demonstrated that increasing the longitudinal bars concentrated at the corner of walls, has positive effects on their seismic performance. In another study, Tavafoghi and Eshghi (2008) investigated the seismic behaviour of tunnel-form concrete building structures with different plans and heights. It was concluded that the fundamental period of these systems in each direction is directly dependent on the total height and the aspect ratio, while number of storeys does not considerably affect the results. Furthermore, the first three modes of vibration were reported to be independent of the height and number of walls in plan.

In another relevant study, Balkaya et al. (2012) investigated the effect of soil-structure interaction on the mechanical characteristics of the tunnel-form structures with different geometries making use of eigenvalue analysis. According to the results, several relations for calculation of the fundamental vibration period of these structures were developed by taking the effect of the soil-structure interaction into account. Through a case study on a 12-storey building with tunnel-form system in Croatia, Klasanovic et al. (2014) demonstrated that while the structure is in the linear domain, the measured fundamental period of is close to the period obtained from EC8.

In a more recent study, Beheshti-aval et al. (2018) evaluated the seismic performance of tunnel-form system subjected to a set of near and far-field earthquake records including forward directivity effects. It was shown that the forward directivity can influence the failure modes of tall tunnel-form structures and reduce the reliability of the design. Mohsenian and Mortezaei (2018a) also evaluated the seismic reliability of tunnel-form structures subjected to accidental torsions. According to their results, eccentricity of mass centre by up to 10% of the plan dimension does not considerably affect the performance of these systems. 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.

## Problem Definition and Research Novelty

Due to the special construction process of tunnel-from buildings and obligation to provide sufficient space to take the formworks out of the perimeter sides of the building, it is not generally possible to construct structural walls in these areas. This can lead to reduction in torsional stiffness of the typical tunnel-from buildings and make them susceptible to exhibit a soft torsional behaviour. As discussed in the previous section, the results of the eigenvalue analysis on several buildings using tunnel-form systems, imply that the torsional modes can occur at frequencies lower than the translational ones, which indicates a flexible torsional behaviour. To control this undesirable response, current design standards generally suggest using regular and symmetric plans, which is followed by architectural limitations. Therefore, the above mentioned studies on tunnel-form structural system have been mainly focused on estimation of the fundamental period and evaluation of the seismic behaviour and design parameters of horizontally regular buildings. Moreover, currently there is no agreement on behaviour factors suitable for seismic design of tunnel-form buildings. Due to the lack of information, in most seismic design guidelines the tunnel-form structural system is categorised as a subcategory of load-bearing wall structural system. However, due to the interaction between well and slab elements, the seismic performance of tunnel-form buildings can be completely different with conventional load-bearing wall systems.

To bridge the above mentioned knowledge gaps in this area, this study aims to investigate the seismic performance and reliability of irregular tunnel-form building by using 3, 5, 7 and 10-storey structures subjected to design earthquakes with different intensity levels simultaneously applied in the two principal directions. A novel approach is also utilized to develop multi-level behaviour factors on the basis of earthquake hazard level and performance limit. The proposed behaviour factors can be efficiently used for performance-based design (PBD) of these systems to achieve specific performance targets. Finally, the reliability studies and fragility curves developed using different damage measures should provide useful insight into the nonlinear shear behaviour and seismic reliability of tunnel-form building structures as a new class of structural systems.

## Methodology

### ○ *Specifications of numerical models*

In this study, the seismic performance of 3, 5, 7 and 10-storey tunnel-form buildings is investigated. Fig. 1 shows the general plan view of the studied buildings as well as the 3D View of the 10-Storey Model. The dotted lines in this figure represent coupling beams with length and height equal to 1 and 0.7 m, respectively. The storey heights are considered to be 3 m. The buildings are assumed to be in high seismic zones with soil type "II" (the shear wave velocity ranges from 375 to 750 m/s) according to ASCE-07 (2016). To ensure that the buildings are irregular in plan, the reentrant corners are around 40% and 50% of the plan dimension in X and Y directions, respectively. It should be mentioned that similar criteria are used in the Iranian Code of Practice for Seismic Design of Buildings (Standard No. 2800).

The buildings were designed based on ACI 318 (2014) by means of ETABS (CSI 2015) Software. Besides, all the requirements prescribed by the Iranian Building and Housing Research Center (BHRCP 2007) for tunnel-form buildings were satisfied except the requirement for horizontal and vertical regularity.

Fig 2 shows the schematic view of detailing and arrangement of reinforcing bars in the walls and coupling beams for the 10-storey building. The thickness of the wall and slab elements was 20 and 15 cm, respectively. Vertical and horizontal reinforcing bars ($\phi_v$ and $\phi_H$) were placed in two layers. The longitudinal bars in the first four storeys of the 10-storey building and the first two storeys of the 7-storey building had 12 mm diameter. For the rest of the elements, that diameter of the longitudinal bars was 8 mm. To provide enough ductility and increase the shear strength of the coupling beams (with free length to height ratio of less than 2), in addition to the special transverse reinforcement ($\phi_D$), diagonal reinforcement ($\phi_A$) was also utilized as suggested by Paulay and Binney (1974) and Zhao et al. (2004). The compressive strength of concrete material and yield strength of steel bars were 25 and 400 MPa, respectively.

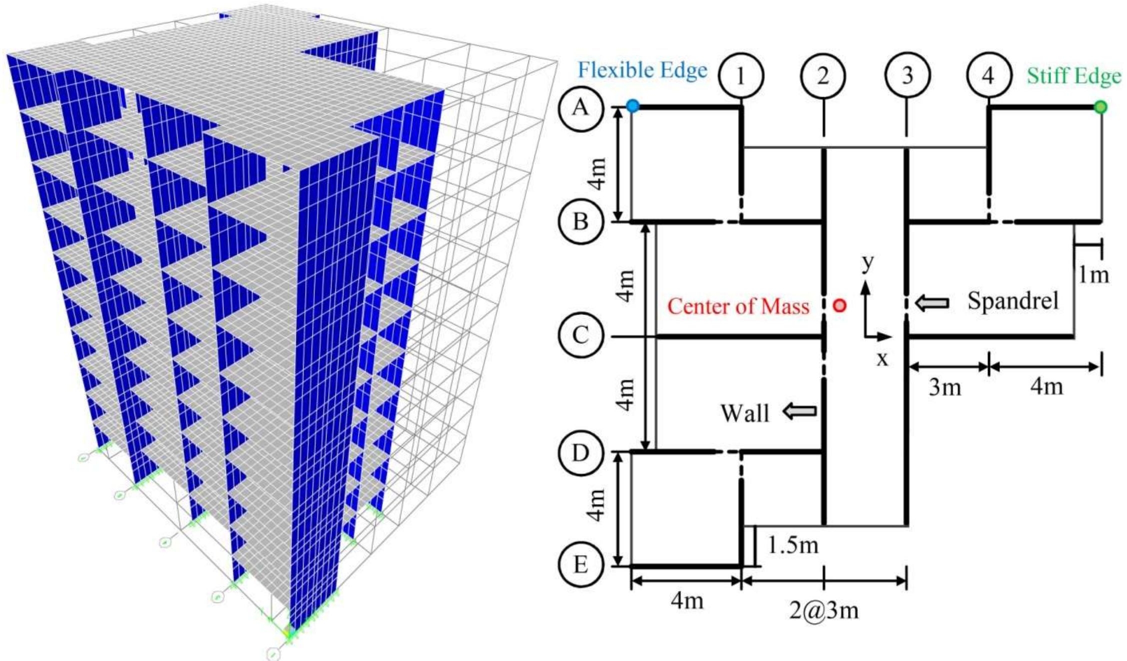


**Fig (1): Plan view of the studied tunnel-form buildings and 3D view of the 10-storey model**

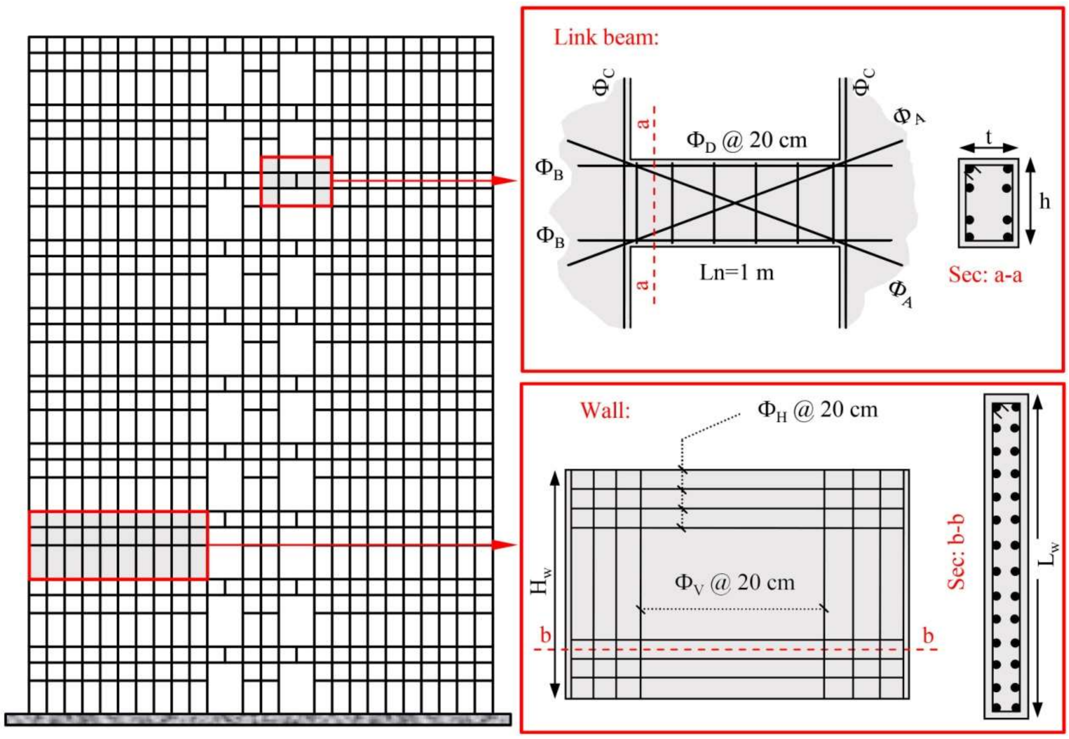


**Fig (2): Schematic representation of detailing and arrangement of reinforcing bars in the walls and coupling beams**

- ***Nonlinear modelling and determination of strength and deformation parameters***

In this study, PERFORM-3D (CSI 2016) Software was utilized to carry out nonlinear analyses on the designed tunnel-form structures. Since the walls and coupling beams were modelled by using "Shear Wall" elements, the shear strain has been adopted as the deformation-controlled parameter for these elements (Allouzi and Alkloub 2017). Fig (3) shows the nonlinear shear behaviour defined for walls and coupling beams. The parameters required for modelling as well as their acceptance criteria were specified in accordance with the general load-displacement relation developed for the shear-control concrete elements prescribed by ASCE14-13 (2014).

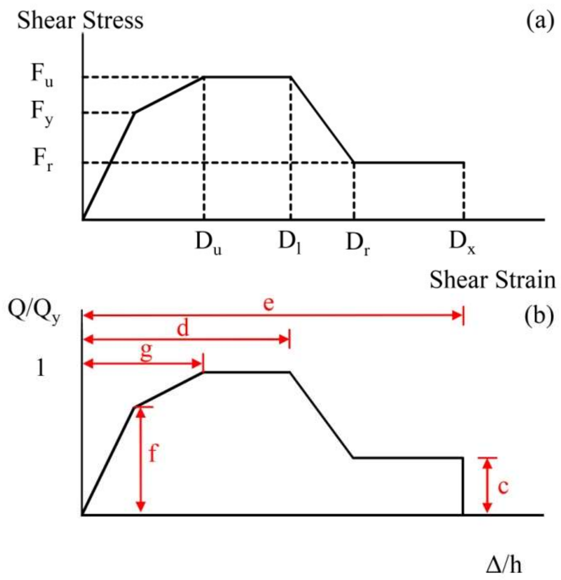


**Fig (3): Nonlinear shear behaviour of walls and spandrels (a) adopted in the software, and (b) proposed in ASCE41-13 (2014) for the shear control members**

In case of walls and shear-control beams, in which ductility is mobilized by means of shear failure, drifts (θ) and chord rotation (γ) were used as the main performance response criteria in accordance with ASCE14-13 (2014). Fig. 4 shows the schematic view of the selected deformation control parameters. It should be noted that the other internal actions in these elements (i.e. axial force and bending moment) are considered as force-control parameters.

Nominal shear strength was considered for modelling the nonlinear shear behaviour of elements. It should be mentioned that the relations used for deep beams, were applied to calculate the nominal strength of the coupling beams due to their notable length to height ratio (Paulay and Binney 1974; Zhao et al. 2004). The slabs were modelled as rigid diaphragms using shell elements. The walls were assumed to have rigid connections at their base, while the foundation uplift was neglected.

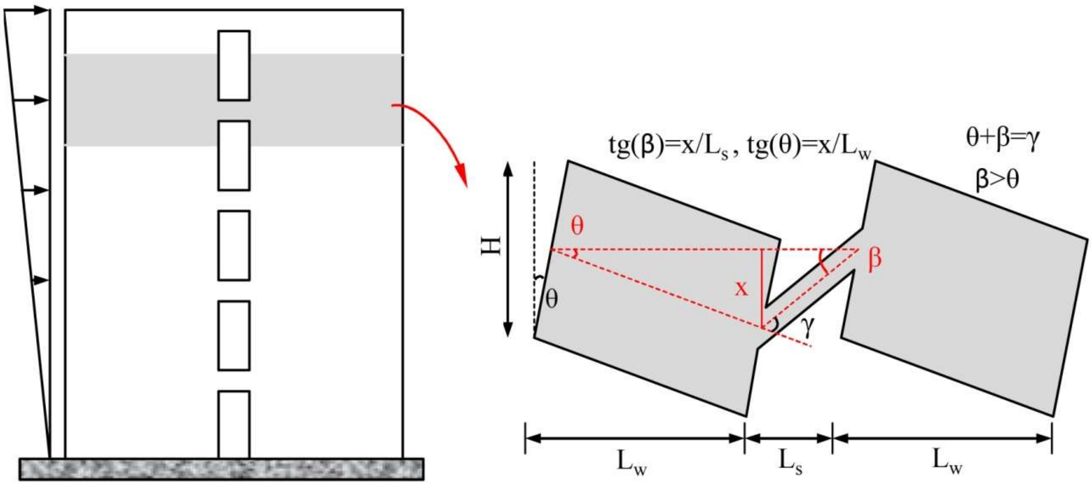


**Fig (4): Introduction of the deformation parameters (θ and γ)**

○ ***Nonlinear Analyses***

The assumptions made for gravity loading in the preliminary design phase were also considered for nonlinear analyses. The upper limit of gravity load effects was accounted for the gravity and lateral load combination based on Equation (1) as recommended by ASCE 41-13 (2014):

$$Q_G = 1.1[Q_D + Q_L] \tag{1}$$

where $Q_D$ and $Q_L$ denote the dead and effective live loads, respectively.

Considering the position of mass centre and centre of rigidity as well as the percentage of walls distributed in the plan, it is found that stiffness and strength of structures and eccentricity of the mass in proportion to the centre of rigidity, is greater in longitudinal (x) compared to the transverse (y) direction. On this basis, the transverse direction was considered as the principal direction of the structures.

The results of eigenvalue analysis on the 3, 5, 7 and 10-storey designed buildings are given in Table (1). The values of the coefficient of translational effective mass in longitudinal and transverse directions (x and y, respectively) indicate the flexible torsional behaviour of the models. It can be also seen that translational and torsional displacements are coupled in the first vibration mode.

**Table (1): Vibration period (T) and coefficient of translational effective mass factor (M)**

| Mode No. | 3-Storey | | | 5-Storey | | |
|---|---|---|---|---|---|---|
| | T(sec) | Mx (%) | My (%) | T(sec) | Mx (%) | My (%) |
| 1 | 0.1067 | 0 | 10.6 | 0.2352 | 0 | 7.5 |
| 2 | 0.0693 | 21.2 | 54.3 | 0.1431 | 7.5 | 65 |
| 3 | 0.0636 | 52 | 27.0 | 0.1182 | 66.3 | 7.2 |
| 4 | 0.0285 | 0 | 3.06 | 0.0550 | 5.6 | 15.3 |

| Mode No. | 7-Storey | | | 10-Storey | | |
|---|---|---|---|---|---|---|
| | T(sec) | Mx (%) | My (%) | T(sec) | Mx (%) | My (%) |
| 1 | 0.4153 | 0 | 6.1 | 0.7833 | 0 | 5.2 |
| 2 | 0.2450 | 3.9 | 66.3 | 0.4524 | 2.3 | 69 |
| 3 | 0.1822 | 66.4 | 4.0 | 0.2971 | 65 | 2.3 |
| 4 | 0.0895 | 4.1 | 10.8 | 0.1564 | 2.4 | 13 |

Mx → Effective translational mass factor in “x” direction.
My → Effective translational mass factor in “y” direction.

In the following section, the performance level of the selected tunnel-form buildings is evaluated subjected to the design basis earthquake (DBE) and maximum considered earthquake (MCE) hazard levels using fragility and incremental dynamic analysis (IDA). It is of note that all models were simultaneously excited in both principal directions. In nonlinear dynamic analyses, the second-order effects (i.e. P-Δ) were taken into account and the Rayleigh damping model with a constant damping ratio of 0.05 was assigned to the models.

## Incremental Dynamic Analysis (IDA)

Incremental dynamic analysis (IDA) is a computational analysis method in which the concept of scaling ground motion records is used to estimate the demand and capacity of a structure in a wide range of behaviour from linear to failure phase (Vamvatsikos and Cornell 2002). By using a number of earthquake records in IDA, the impact of variation in the parameters related to the accelerograms (e.g. amplitude, strong-motion duration, frequency content) can be studied. The selection of appropriate earthquake records including their intensity and response parameters are considered as the main requirements of this analysis. By increasing the number of earthquake records used for IDA, the earthquake-related uncertainties are reduced; however, the computational time and volume of the outputs can significantly increase. Based on the recommendations by previous studies (e.g. Shome and Cornell 1999), using at least 10 accelerograms for IDA can lead to satisfactory results. Therefore, in this study 10 pairs of earthquake records were selected from the Pacific Earthquake Engineering Research Center online database (PEER). All the selected accelerograms were far-field earthquakes recorded on the sites with soil class “II” (shear wave velocity ranges from 375 to 750 m/s) in accordance with ASCE-07 (2016). Table 2 lists the characteristics of the records including their closest distance to fault rupture, magnitude and peak ground acceleration (PGA).

By comparison between the spectral response of each pair of accelerogram, the main component was selected based on the greater spectral values in the vibration frequency range of the structures and applied to the buildings in the “y” direction. The less intense component was simultaneously applied to the perpendicular direction (x). Fig (5) compares the acceleration response spectra of the main components of the selected records scaled to their PGA.

**Table2: Selected earthquake records for time-history analysis**

| Record No. | Earthquake & Year | Station | R[a] (km) | Component | $M_W$ | PGA (g) |
|---|---|---|---|---|---|---|
| R1 | Cape Mendocino, 1992 | Eureka – Myrtle & West | 42 | 90 | 7.1 | 0.178 |
| R2 | Northridge, 1994 | Hollywood – Willoughby Ave | 23 | 180 | 6.7 | 0.246 |
| R3 | Northridge, 1994 | Lake Hughes #4B - Camp Mend | 33 | 90 | 6.7 | 0.063 |
| R4 | Cape Mendocino, 1992 | Fortuna – Fortuna Blvd | 20 | 0 | 7.1 | 0.116 |
| R5 | Northridge, 1994 | Big Tujunga, Angeles Nat F | 20 | 352 | 6.7 | 0.245 |
| R6 | Landers, 1992 | Barstow | 35 | 90 | 7.4 | 0.135 |
| R7 | San Fernando, 1971 | Pasadena – CIT Athenaeum | 25 | 90 | 6.6 | 0.110 |
| R8 | Hector Mine, 1999 | Hector | 12 | 90 | 7.1 | 0.337 |
| R9 | Kobe, 1995 | Nishi-Akashi | 9 | 0 | 6.9 | 0.509 |
| R10 | Kocaeli (Turkey), 1999 | Arcelik | 54 | 0 | 7.5 | 0.219 |

[a] Closest Distance to Fault Rupture

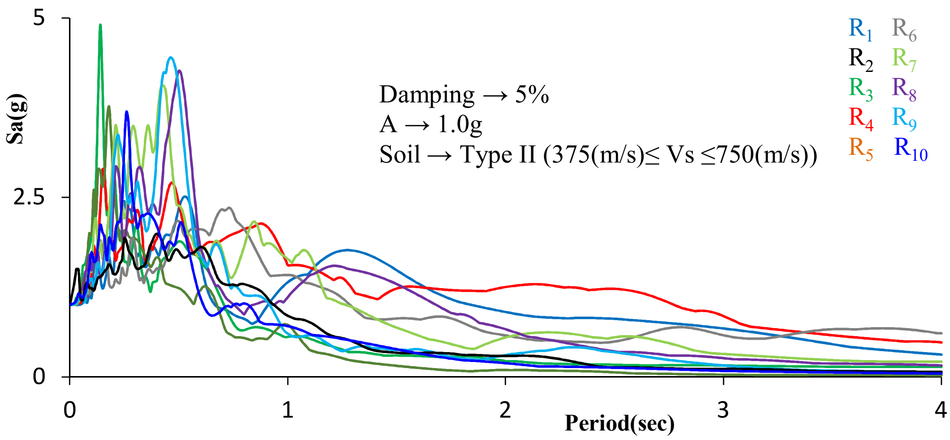


**Fig (5): The acceleration response spectra of the selected records scaled to their PGA**

The earthquake records applied to the structure were incrementally intensified within the IDA, while a similar scale factor was used for both ground motion components. Here, the intensity measure and the structural response to the input motion are denoted by IM and DM, respectively. The fragility curves demonstrate the relation between these two parameters.

It should be noted that, due to the irregularity of the selected buildings, the torsional and translational components of the first vibration mode are coupled in this study (see Table 1). Therefore, using the spectral acceleration of the first vibration mode as the seismic intensity measure would be inadequate. To address this issue, in this study the peak ground acceleration (PGA) was chosen as intensity measure (IM), since it is independent of the structural characteristics.

Different global damage indexes and particularly inter-storey drifts are generally taken as the damage measure parameter (DM) in IDA. For the tunnel-form buildings studied herein, as the elements are shear-control and due to lack of specific values to quantitatively define the global damage indexes for this novel system, maximum drift and chord rotation developed in the walls and coupling beams were adopted as the main damage parameters in IDA (see Fig (4)). It should be mentioned that the global damage indexes proposed by Chobarah (2004) for squat walls could be also employed, but in order to enhance the reliability on the results, the latter parameters were chosen.

The curves obtained from the IDA analyses and the corresponding statistical percentiles are illustrated in Figs (6) and (7), respectively. It is shown that, in general, the PGA level required for the walls and coupling beams to reach various performance levels, is several times higher than that of the DBE hazard level. Thereby, it is reasonable to expect these buildings exhibit an elastic behaviour even during strong ground motions. Additionally, it can be noticed that in comparison with the walls, the coupling beams reach the performance levels at lower PGA levels. As shown in Fig (4), this might be attributed to the larger seismic demand of such elements. The results in Figs (6) and (7) also show that the PGA level corresponding to a certain performance level, is reduced for taller buildings.

It was found that the walls located on the axis 4 of the plan (see Fig (1)), exhibit greater seismic demands and hence, these elements reach the different performance levels earlier than the other walls. This is due to the fact that the torsion induced as a result of horizontal-irregularity intensifies the displacement demands in the perimeter parts of the buildings.

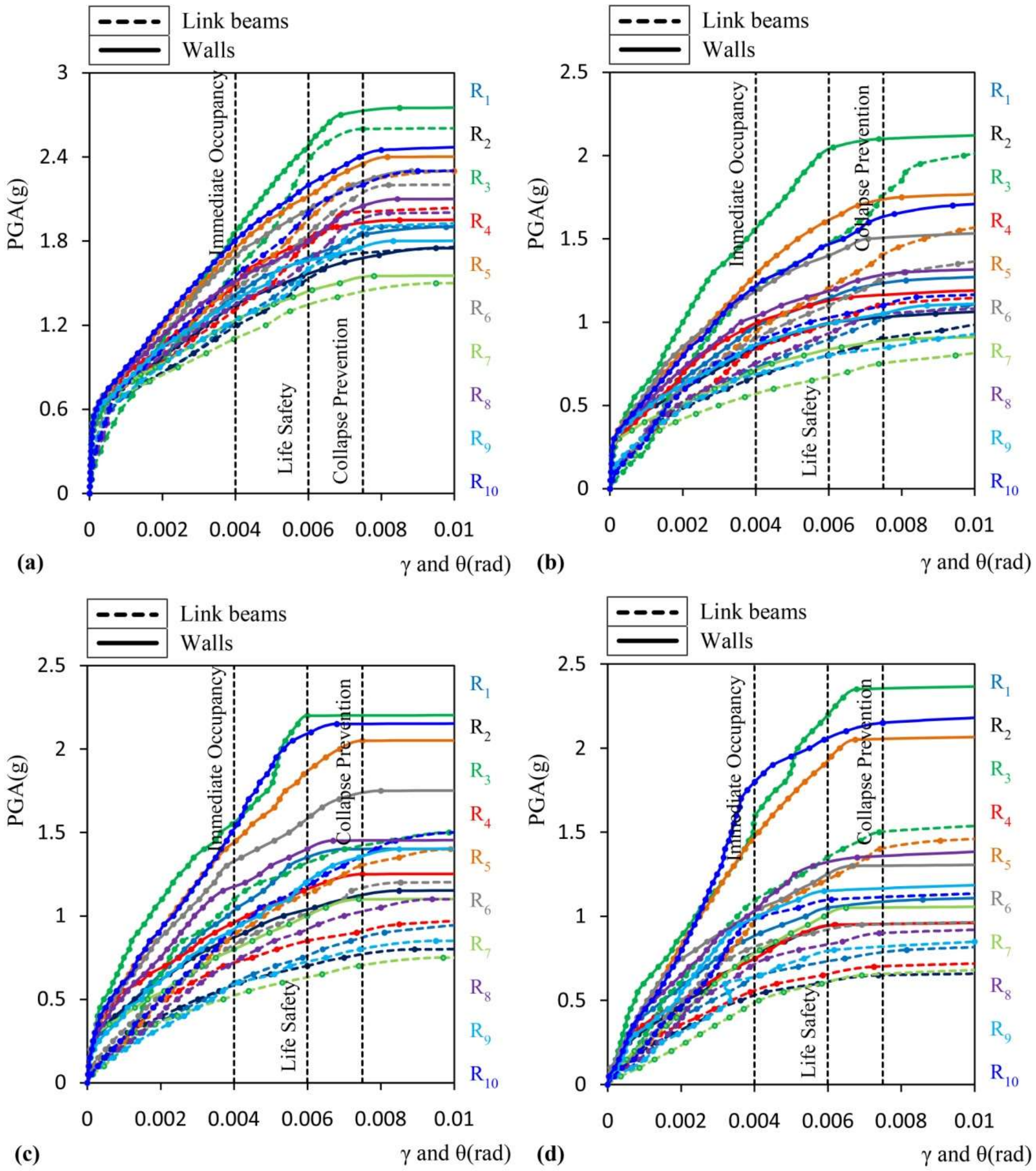


**Fig (6): Incremental Dynamic Analysis (IDA) results and the Limit States for (a) 3-storey, (b) 5-storey, (c) 7-storey, and (d) 10- storey buildings**

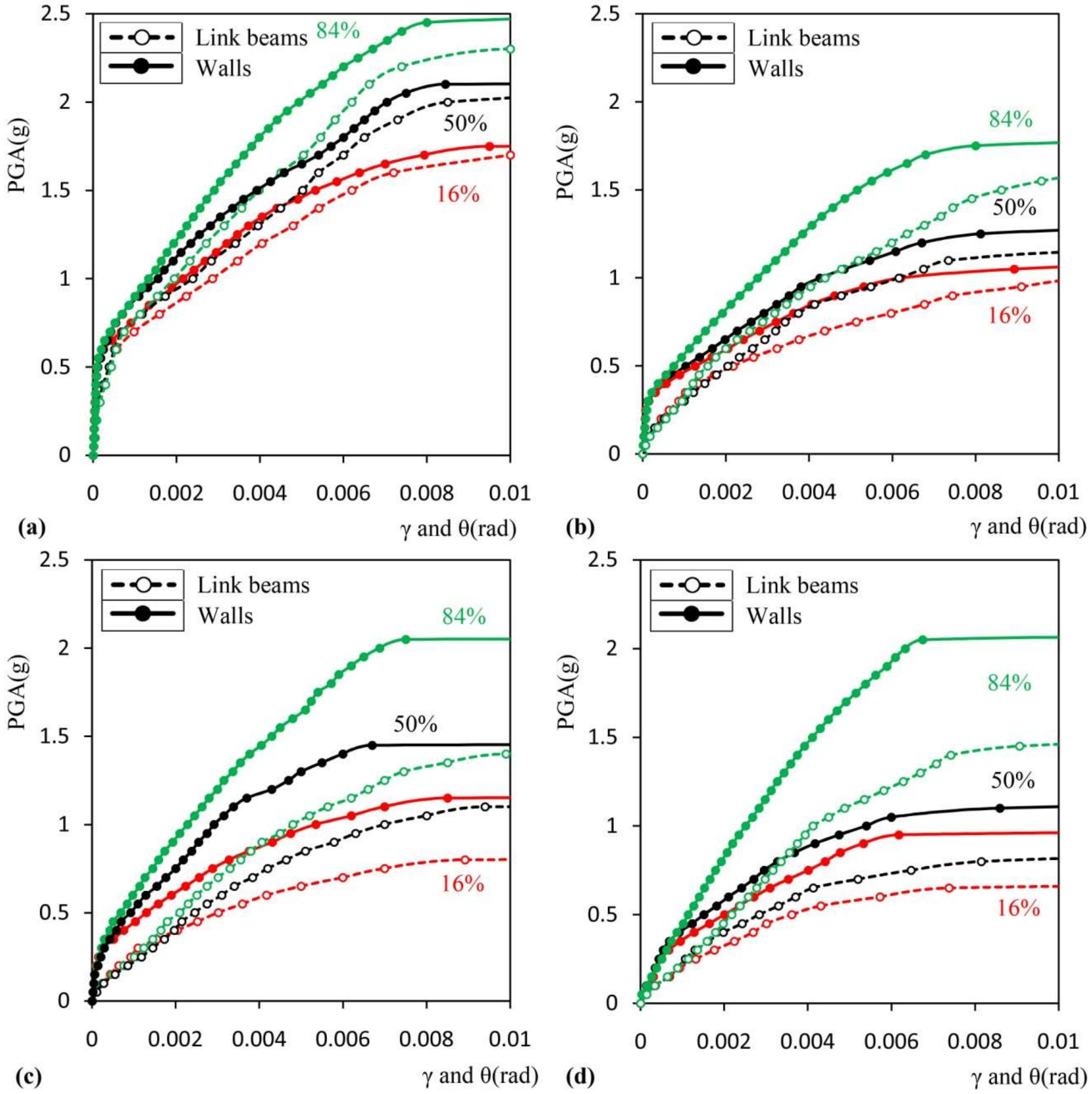


**Fig (7): Comparison of 16, 50 and 84 Percentiles of results obtained by the Incremental Dynamic Analysis (IDA) for (a) 3-storey, (b) 5-storey, (c) 7-storey, and (d) 10- storey buildings**

## Generation of Fragility Curves Using IDA

Many uncertainties can affect the accuracy of the seismic performance assessment of a building under earthquake events (Hajirasouliha et al. 2016). Such uncertainties are generally classified into two groups. The first group deals with the existing uncertainties in nature such as the differences lying in the material properties, ambient effects etc. The second group concerns the uncertainties due to the errors in the computational methods, modelling procedures etc (Ang and Tang 2007; Berahman and Behnamfar 2007). In such conditions, expression of the building's performance in a probabilistic form (e.g. using fragility curves) appears to be the most logical approach. The fragility curves represent the cumulative distribution of loss (Cimellaro et al. 2006), and can be mathematically written as in Equation (2):

$$Fragility = P\left[R > LS_i \mid IM = S\right] \qquad (2)$$

where, $R$ represents the building's response, $LS_i$ denotes the performance level or limit state related to $R$, $IM$ (intensity measure) is the intensity of the input earthquake ground motions, and $S$ is a particular value of $IM$.

The distribution of structural responses at different levels of earthquake intensity can be demonstrated by using fragility curves. The fragility curves can be also utilized as efficient tools to assess the seismic vulnerability of both structural and non-structural elements (Nielson 2005; Kinali 2007). Different methods can be used to generate fragility curves including experts' judgments, empirical-statistical approach, experimental, analytical and combined methods (Khalvati and Hosseini 2008). In this study, the fragility curves were generated by means of analytical or IDA analysis. By using the lateral drift and chord rotation as the damage measure parameters for the walls and coupling beams, the performance levels defined by ASCE41-13 (2014) were considered as the damage criteria (see Fig (6)). Subsequently, fragility curves were generated for each event of exceedance from these damage states as shown in Fig (8).

Table 3 lists the probability of exceeding the performance levels of Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) in DBE and MCE hazard scenarios for the 3, 5, 7, and 10- storey buildings. The results show the early damage in the coupling beams compared to the walls, which indicates these elements can play the role of seismic fuse in tunnel-form buildings. In all the buildings used in this study, the probability of exceeding the IO performance level for coupling beams under DBE and MCE hazard levels was less than 2 and 19%, respectively. Accordingly, these values for the walls in the event of DBE and MCE scenarios were around 0 and less than 2%. Based on the results, it can be concluded that the studied tunnel-form buildings can practically satisfy IO performance level even under very strong earthquake events.

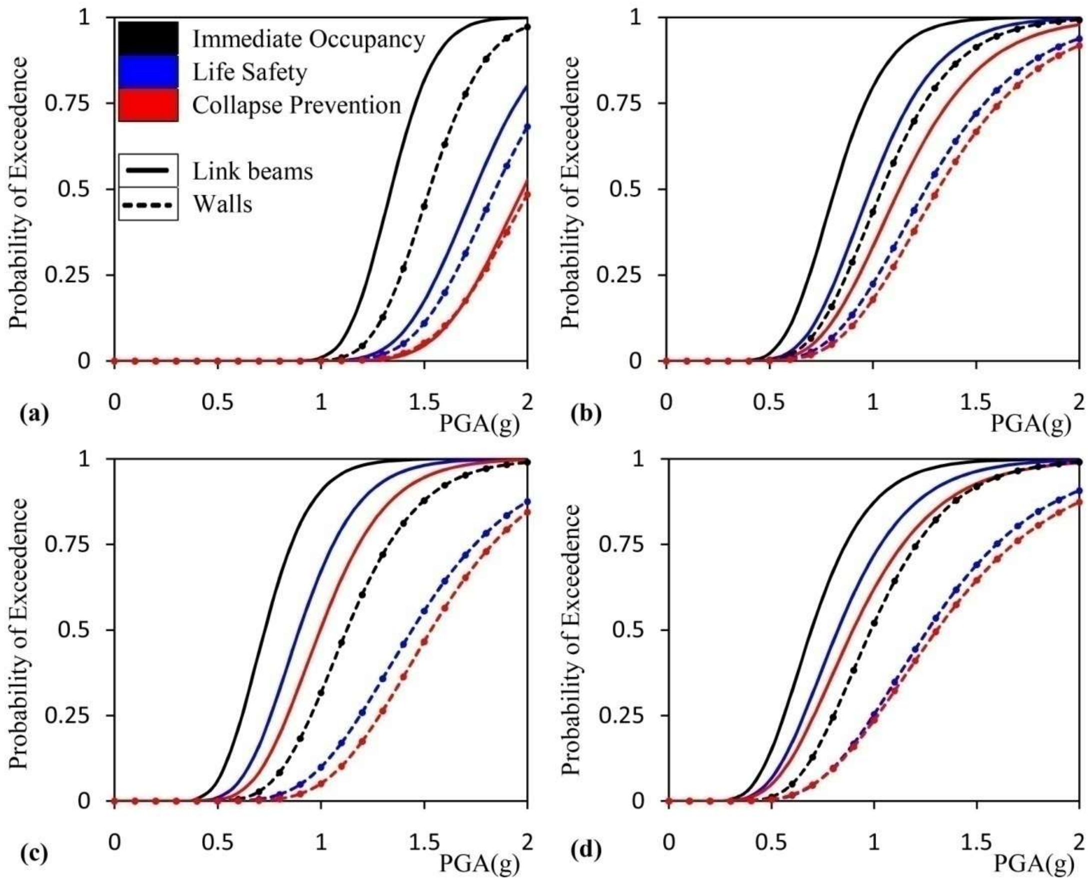


**Fig (8): Fragility curves for (a) 3-storey, (b) 5-storey, (c) 7-storey, and (d) 10- storey buildings**

**Table (3): Probability of exceeding the performance levels of Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) in DBE and MCE hazard scenarios (%)**

| Hazard Levels → | | Design Basis Earthquake | | | Maximum Considered Earthquake | | |
|---|---|---|---|---|---|---|---|
| buildings | Elements | IO | LS | CP | IO | LS | CP |
| 3-Storey | Beam | 0 | 0 | 0 | 0 | 0 | 0 |
| | Wall | 0 | 0 | 0 | 0 | 0 | 0 |
| 5-Storey | Beam | 0 | 0 | 0 | 3.43 | 0.75 | 0.33 |
| | Wall | 0 | 0 | 0 | 0.5 | 0.2 | 0.1 |
| 7-Storey | Beam | 0.15 | 0 | 0 | 12.2 | 2.5 | 0.98 |
| | Wall | 0 | 0 | 0 | 1.83 | 0.6 | 0.58 |
| 10-Storey | Beam | 1.5 | 0.5 | 0.3 | 18.9 | 8.8 | 6.4 |
| | Wall | 0 | 0 | 0 | 2.65 | 0.87 | 1 |

Comparison between the fragility curves depicted in Fig (9) demonstrates that, in general, by increasing the building's height, the probability to exceed various performance levels increases. This trend becomes more profound in the case of coupling beams.

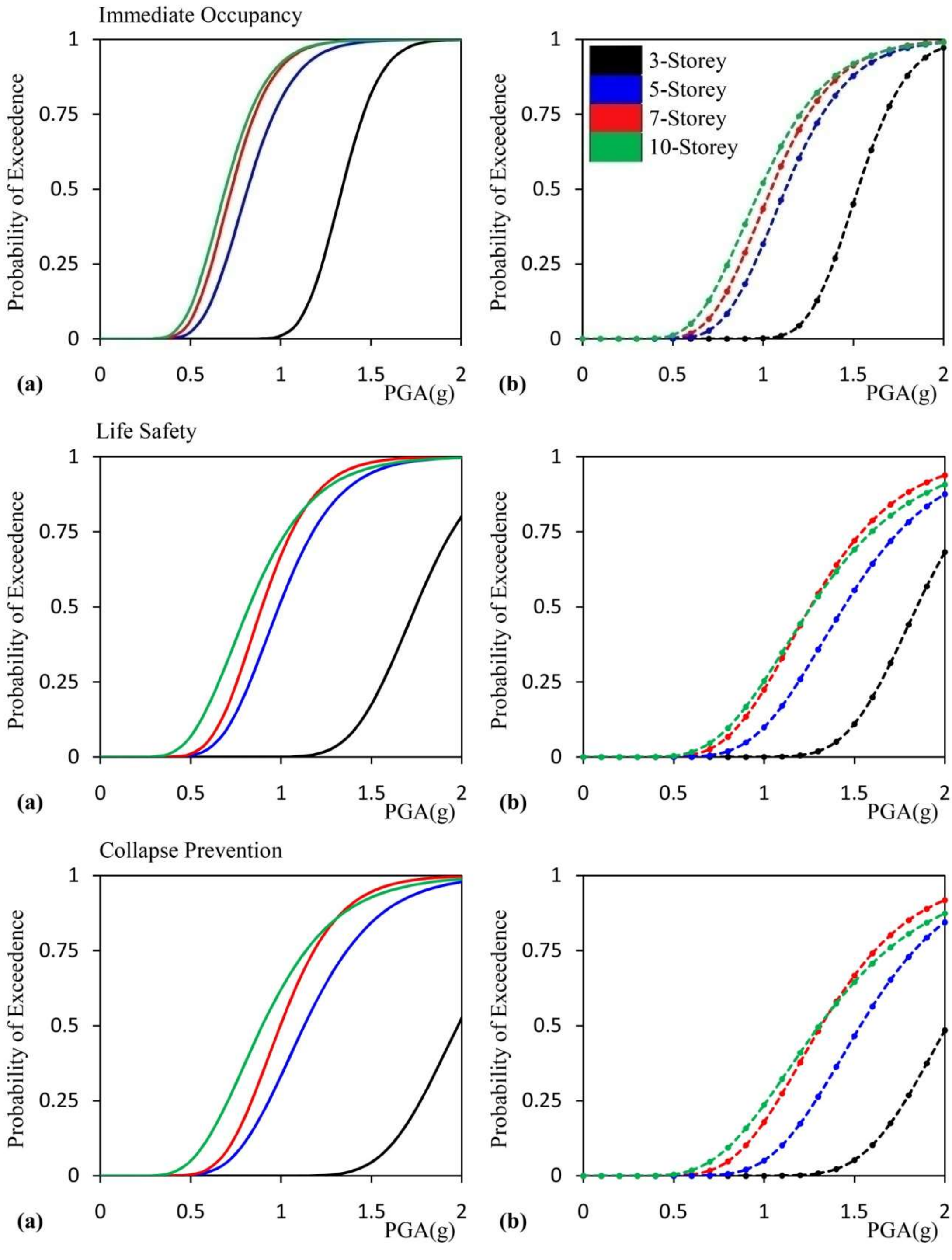


**Fig (9): Comparison between fragility curves of the 3, 5, 7 and 10-storey buildings: (a) Link beams; (b) Walls**

## Estimation of Response Modification Factor

○ ***Code-Based Response Modification Factor ($R_{Code}$)***

The response modification factors provide by the seismic codes are mainly based on engineering judgments, experiences and lessons learned from the past earthquakes. Many researchers have studied the limitations of code-based response modification factors ($R_{Code}$), concluding that a more rigorous estimation can lead to higher reliability in the methods and provisions prescribed by the seismic codes (e.g. Whittaker et al. 1999). One of the problems with the response modification factor introduced by seismic design codes ($R_{Code}$) as "force-based method" is that it is unclear what level of intensity and performance can be achieved.

As tunnel-form structural system has recently emerged, very limited information is available regarding its performance in the past earthquakes. In addition, currently in most seismic codes this system is considered as a subcategory of "reinforced concrete (RC) bearing wall system". Therefore, depending on the level of ductility, the response modification factor for tunnel-form is typically considered to be between 3 to 5 (e.g. BHRCP 2007; Standard No.2800 2014). However, considering the 3D behaviour of this structural system due to the interaction between intersecting walls and floor slab, it is not very logical to adopt the parameters related to the RC bearing wall with a 2D performance. This highlights the need to develop suitable behaviour factors for tunnel-form buildings as discussed in the previous sections.

○ ***Demand-Based Response Modification Factor, $R_{Demand}$ (Displacement/Ductility)***

The value of demand response modification factor depends on site seismicity as well as physical and geometrical specifications of the building. Several studies have indicated that the parameters like earthquake magnitude and focal depth do not considerably influence this factor compared to the other parameters such as ductility, energy absorption, fundamental period, over-strength, redundancy, number of degrees of freedom and soil type (Lia and Biggs 1980; Miranda 1991; ATC-19 1995).

In this study, demand-based response modification factor, $R_{Demand}$, is calculated based on the following equation:

$$R_{Demand} = R_{\mu}^{MDOF} . \Omega_S . R_d \tag{3}$$

where $R_\mu^{MDOF}$ denotes the modification factor originated form ductility and dissipated energy caused by residual behaviour directly extracted from the actual structure comprising of multi degrees of freedom;"Ωs" represents the over-strength factor, by which the effect of redistribution of actions due to redundancy is also considered; and $R_d$ is called the allowable stress factor. It should be mentioned that as the loads and resistance of materials are multiplied by safety factors in allowable stress or ultimate strength design methods, it is required to utilize $R_d$ to reduce the forces to the design strength level. These parameters are calculated based on Equations (4) to (6) (Fanaie and AfsarDizaj 2014).

$$R_\mu = V_e / V_y \tag{4}$$

$$\Omega_s = V_y / V_s \tag{5}$$

$$R_d = V_s / V_d \tag{6}$$

To attain these factors, the following parameters are introduced:

For a certain level of intensity, demand spectrum of the site is prepared and the earthquakes compatible with this spectrum are selected. The selected earthquakes which are called demand earthquakes are applied to the structure assuming a linear behaviour, and then the base shear is recorded. The average of the base shear values obtained, is called elastic base shear ($V_e$). In this study, artificial accelerograms corresponding to the code-based design spectrum were employed, so that the design earthquakes could be compatible with the site hazard as much as possible. In doing so, 10 artificial earthquake records were extracted based on the wavelet transform function from the demand spectrum and then, applied to the structures as shown in Fig (10). It should be noted that the earthquakes given in Table (2), have been utilized to produce the artificial records (Hancock et al. 2006).

In the next step, the demand earthquakes were applied to the structure assuming a nonlinear behaviour and the maximum roof displacement was obtained. Average of the drift values induced by the DBE hazard scenario was taken as the target on the capacity curve. After bi-linearization of this curve on the basis of ASCE41-13 (2014), yield base shear (Vy) is obtained. The shear corresponding to the commencement of nonlinear behaviour ($V_s$), is defined as the point where the capacity curves obtained based on linear and nonlinear behaviour are separated. Design base shear ($V_d$) is calculated by dividing the linear spectral acceleration multiplied by total building's weight to the code-based response modification factor. Fig (11) shows the bi-linearization of the capacity curve and the parameters used to calculate the response modification factor.

For the studied buildings, the demand response modification factors $R_{Demand}$ are obtained according to the above procedure and presented in Table (4).

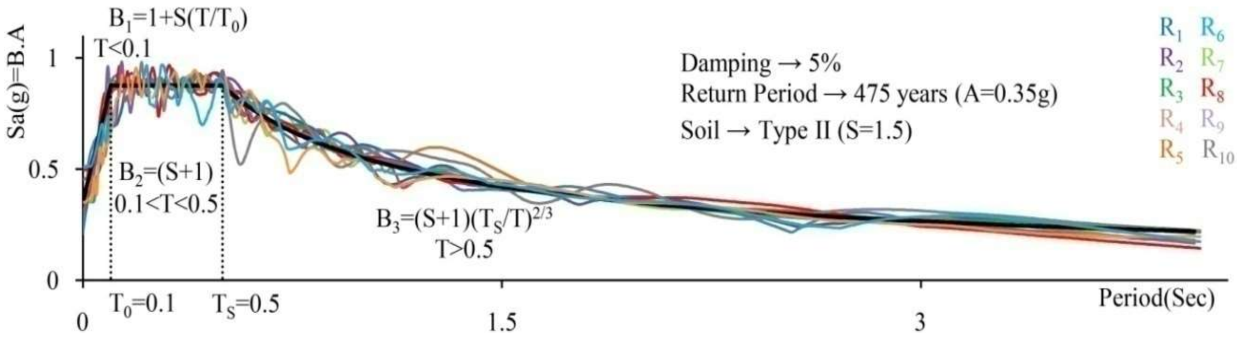


**Fig (10): Comparison between artificial accelerograms and site demand spectra**

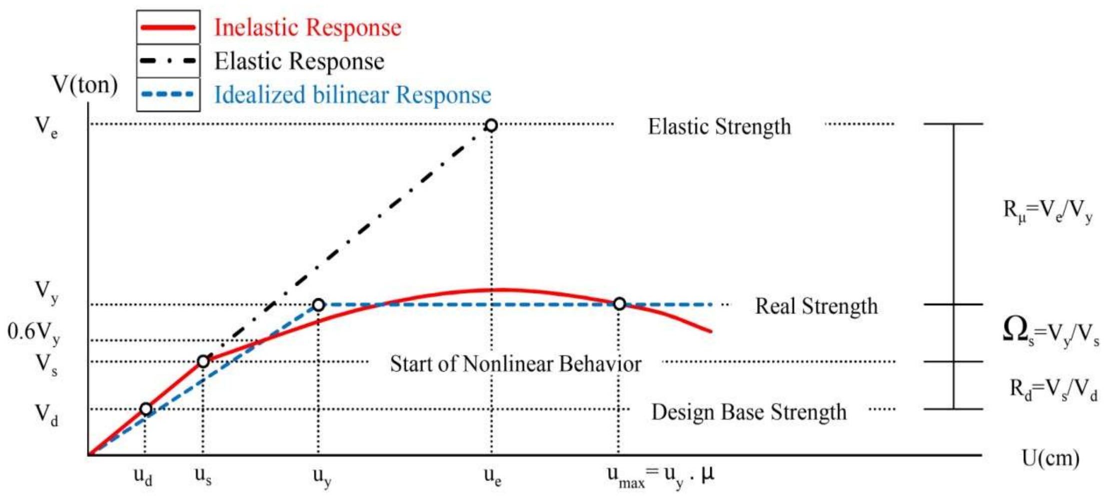


**Fig (11): Bi-linearization of the capacity curve and introduction of parameters used to calculate the response modification factor**

**Table (4): Code and Demand Response Modification Factors for the studied buildings**

| | $R_{Code}$ | $R_{Demand}$ | | | |
|---|---|---|---|---|---|
| | | 3-Storey | 5-Storey | 7-Storey | 10-Storey |
| **PGA(g)** | 0.35 | 0.35 | 0.35 | 0.35 | 0.35 |
| **Ve(ton)** | -- | 540.1 | 878.5 | 1200.3 | 1559.3 |
| **Vy(ton)** | -- | 280 | 465 | 446.8 | 500 |
| **Vs(ton)** | -- | 109 | 220 | 302 | 400 |
| **Vd(ton)** | -- | 132.9 | 228.7 | 324.5 | 468.2 |
| **Rμ** | -- | 1.92 | 1.89 | 2.68 | 3.118 |
| **Ωs** | -- | 2.57 | 2.114 | 1.48 | 1.25 |
| **Rd** | -- | 1 | 1 | 1 | 1 |
| **R** | 5 | 4.955 | 3.993 | 3.975 | 3.898 |

○ ***Supply Response Modification Factor, $R_{Supply}$ (Capacity)***

This factor depends on the building's capacity to withstand nonlinear deformations to satisfy the required performance levels. The buildings can be designed based on the force-based method using a strength reduction factor assuming a certain damage level under DBE hazard scenario (Fajfar 2000). This approach is currently utilized for seismic assessment of existing buildings. The algorithm taken to derive the supply response modification factor, $R_{Supply}$, based on the lateral strength of structures is as follows (ATC-40 1996; Mwafy and Elnashai 2002).

Assuming a nonlinear behaviour for the structure, incremental dynamic analysis (IDA) is conducted on the structure making use of the earthquake records attributed to the site conditions. Subsequently, PGA factors triggering damages (in this study, reaching the structural walls to the performance level of life safety) are obtained. Afterwards, under the PGA values obtained from the previous step, linear dynamic analysis is conducted and the mean value of the resulted base shears is calculated ($V_e$). In the next step, by using modal lateral load distribution, a pushover analysis is performed on the structure to reach the target displacement corresponding to the damage levels obtained from the first step. By bi-linearizing the capacity curve (see Fig (11)), the yield base shear ($V_y$) is identified. The rest of the parameters required to calculate $R_{Supply}$ are similar to those explained in the previous section. Table (5) shows the results of the supply response modification factor for the studied buildings.

**Table (5): Code and Supply Response Modification Factors for the studied buildings**

| | $R_{Code}$ | $R_{Supply}$ | | | |
|---|---|---|---|---|---|
| | | 3-Storey | 5-Storey | 7-Storey | 10-Storey |
| **PGA(g)** | 0.35 | 1.88 | 1.56 | 1.46 | 1.23 |
| **Ve(ton)** | -- | 1653.4 | 2126.2 | 2870.8 | 3599.6 |
| **Vy(ton)** | -- | 696 | 630 | 552 | 500 |
| **Vs(ton)** | -- | 109 | 220 | 302 | 400 |
| **Vd(ton)** | -- | 132.9 | 228.7 | 324.5 | 468.2 |
| **Rμ** | -- | 2.38 | 3.38 | 5.20 | 7.20 |
| **Ωs** | -- | 6.39 | 2.86 | 1.83 | 1.25 |
| **Rd** | -- | 1 | 1 | 1 | 1 |
| **R** | 5 | 15.169 | 9.665 | 9.505 | 9 |

As shown in Fig (12), supply response modification factors for the studied buildings based on the corresponding hazard levels, are smaller than the demand factor. This indicates the high strength of these structures to sustain intense hazard levels in highly seismic areas as discussed before. For each ordered pair in ($A_0$) zone shown in Fig (12), walls as the main load-resisting members in tunnel-form buildings remain in elastic range of behaviour. It means that for the selected DBE hazard level (specified by Standard No.2800) and response modification factor of 4, the walls will exhibit insignificant shear strain under this level of intensity. Selection of an R-factor ranging from demand to supply values corresponding to a specific damage level, will ensure the structure satisfies the desired performance level for the design intensity level. As an instance, for each ordered pair in the red zone (A) shown in Fig (12), the shear strain developed in the walls will be less than the limit values corresponding to the performance level of life safety (LS).

For better comparison, Fig (13) demonstrates the effect of building's height on the code-based, demand and supply response modification factors. For each value of response modification factor in the grey zone shown in this figure, the structures are expected to be rated in the performance levels higher than life safety (LS) under the DBE or events with lower intensities. This implies that using code-based R-factor equal to 5 in the preliminary design process can ensure the structural safety and stability of the buildings under DBE hazard level. It can be noted that this value of response modification factor can also guarantee that the structures satisfy the life safety (LS) performance criteria in the event of MCE scenario (PGA=0.55g).

As it is observed in Fig (13), although increasing the building's height reduces the demand and supply response modification factors, the rate of variations is not significant (except for the 3-storey building). This trend is more profound for the demand response modification factor. The results also indicate that by decreasing the building's height, in general, the safety margin increases. Moreover, parametric analysis of the demand and supply response modification factors shows that as the building's height increases, the modification factors obtained form ductility ($R\mu$) and over-strength ($\Omega s$) are respectively improved and reduced. This is most likely due to the shear and rigid behaviour of shorted buildings and flexural and membrane behaviour of the taller ones.

It should be noted that, with respect to the considerable redundancy and stiffness of tunnel-form buildings, in most cases (especially when low-rise structures are of concern), the minimum code requirements will govern the design of structural elements. This can lead to oversized sections, which increases the constructional costs of these structures. Therefore, the results suggest that tunnel-form structural system is more suitable for construction of the mid and high-rise building structures. While more studies may be required to develop more accurate response modifications factors for irregular tunnel-form buildings, the results of this study should prove useful in the preliminary performance-based design of these systems.

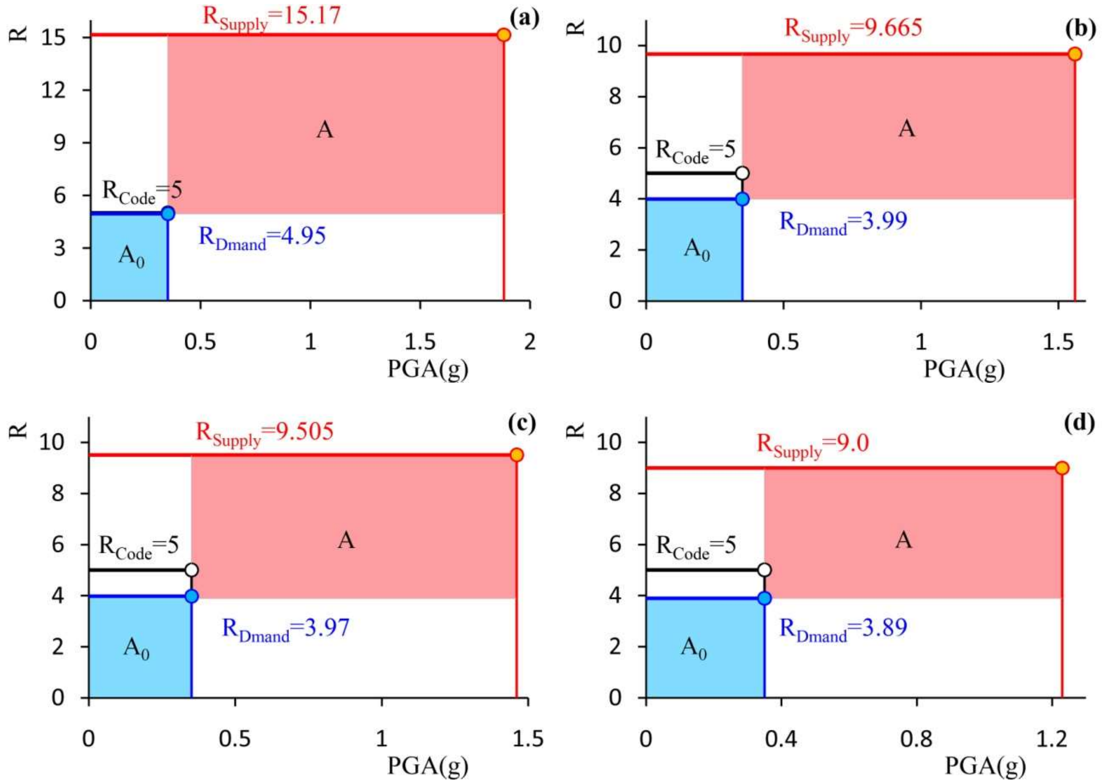


**Fig (12): Code-Based, Demand and Supply Response Modification Factors for (a) 3-storey, (b) 5-storey, (c) 7-storey, and (d) 10- storey buildings**

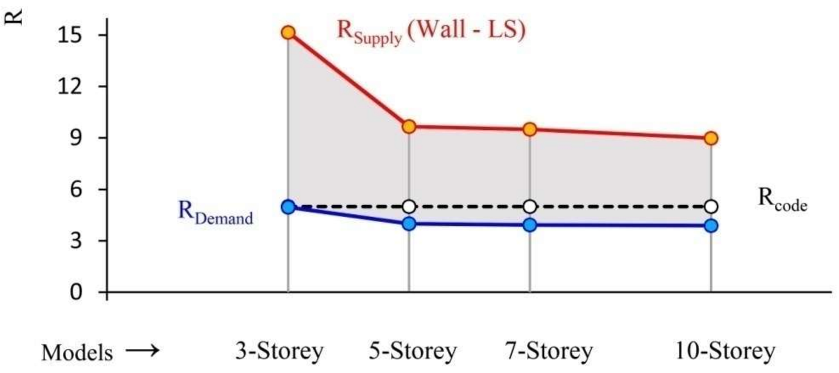


**Fig (13): Effect of building's height on the Code-Based, Demand and Supply Response Modification Factors**

## Natural Frequencies of Irregular Tunnel-Form Buildings

As mentioned before, analysis of the characteristics of the vibration modes of the irregular tunnel-form buildings in this study showed that the translational and torsional displacements in the first mode (along y direction) are coupled (see Table (1)). The results also indicated that torsional displacements in general possess a greater share compared to translation displacements.

To assess the torsional stiffness, Ω parameter is defined as the ratio of torsional to translational frequencies of the structure using the following equation:

$$\Omega = \sqrt{\frac{K_\theta}{K} \times \frac{M}{I_M}} \qquad (7)$$

In this equation, $K_\theta$, $I_M$, K and M, respectively denote the torsional stiffness, mass moment of inertia, lateral stiffness and building's mass. In this study, Ω parameter was estimated for all the horizontally irregular structures. Torsional stiffness and mass moment of inertia have been calculated at the centres of rigidity and mass, respectively (Annigeri and Mittal 1996). In this respect, Equation (7) can be rewritten as:

$$\Omega^2 = \frac{K_{\theta,CS} \times M}{I_{M,CM} \times K} = \frac{\rho_K^2}{\rho_M^2} \qquad (8)$$

where $\rho_K$ and $\rho_M$ represent the scaled stiffness and mass gyration radius about centres of rigidity and mass, which are calculated from equations (9) and (10). It is noted that "b" represents the plan's width.

$$\rho_k = \frac{1}{b}\sqrt{\frac{K_{\theta,CS}}{K}}\ , \ \rho m = \frac{1}{b}\sqrt{\frac{I_{M,CS}}{M}} \qquad (9), (10)$$

It should be mentioned that calculation of the above parameter by using Equations (9) and (10) can be a difficult task. To tackle this issue, in this study the torsional index (Δ) is employed. This index is defined as the ratio of displacements of left and right edges of storey diaphragms while structure is in elastic range of behaviour. It is obtained by conduction pushover analysis, in which loading pattern is triangular and lateral loads are applied to the mass centres. Subsequently, $\rho_K$ is calculated based on Equation (11) as suggested by Tso and Wong (1995).

$$\Delta = \frac{\delta_{\min}}{\delta_{\max}} = 1 - \left(\frac{e}{\rho_k^2}\right)\left(1 + \left(\frac{e}{\rho_k^2}\right)(0.5 + \eta)\right)^{-1} \tag{11}$$

where $\delta_{min}$ and $\delta_{max}$ are minimum and maximum displacements of the edge, respectively (displacement of stiff edge of diaphragm as shown in Fig 1); $\Delta$ represents the ratio of minimum to maximum displacements; and e and $\eta$ are the distance between centres of rigidity and mass and the distance between the centres of geometry and rigidity, respectively (both normalized to the plan's width). In this study, for each storey, $\rho_K$ is calculated based on the latter equation.

Fig (14) shows the $\Omega$ parameter calculated for each storey of the studied buildings. It is shown that $\Omega$ for all buildings is less than 1, which means the dominant behaviour of the buildings is governed by torsional displacements. Interestingly, as the number of storeys increases, the value of this parameter is reduced indicating the fact that torsion is intensified in the upper storeys. In this regard, smaller $\Omega$ values have been calculated for the taller buildings implying the higher effects of torsion developed in this building. Based on the results, employing the drift at mass centre cannot accurately represent the distribution of maximum responses developed in the storeys. Also it is shown that, due to the high torsional movements developed in the upper storeys, the centre of the roof may not be a proper choice for displacement requirements. Therefore, to assess the level of damage, it is recommended to use other response parameters such as flexible edge displacements or the maximum strains in the structural elements.

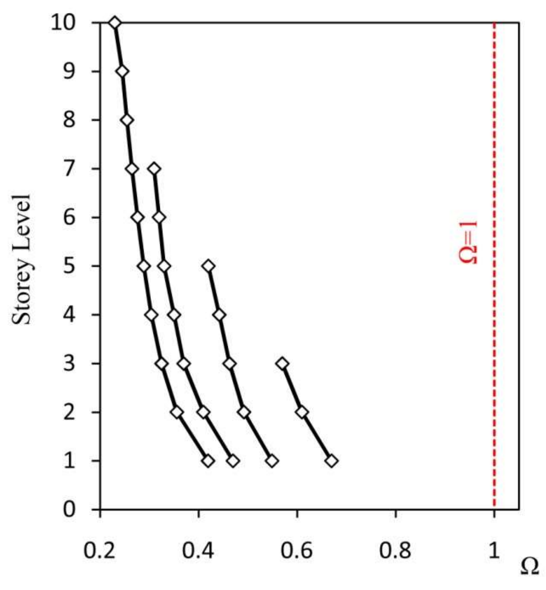


**Fig (14): Uncoupled frequency ratios for 3, 5, 7 and 10-storey buildings**

For better insight, Equation (11) can be rewritten in the following form:

$$\rho_k^2 = \left( \frac{0.5(1+\Delta)}{1-\Delta} - \eta \right) e \qquad (12)$$

Fig (15) shows the scaled torsional stiffness ($\rho_K$) as a function of minimum to maximum displacement ratio ($\Delta$) for the tunnel-form buildings used in this study. In general, it is shown that increasing $\Delta$ results in an increase in $\rho_K$. When the minimum and maximum displacements of the edge are equal and in the same direction (i.e. $\Delta$=1), $\rho_K$ tends to infinity indicating a complete translation displacement. On the contrary, for the case where the minimum and maximum displacements of the edge are equal but in the opposite direction (i.e. $\Delta$=-1), $\rho_K$ tends to zero representing a dominant torsional behaviour.

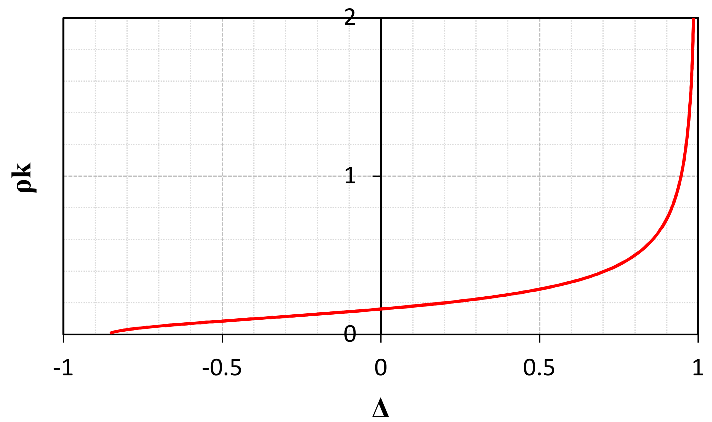


**Fig (15): Scaled torsional stiffness ($\rho_K$) as a function of minimum to maximum displacement ratio ($\Delta$), e= 0.056 and $\eta$= 0.039**

## Conclusions

With reference to the models studied herein and the assumptions made, the results indicate that the tunnel-form structural system is capable to exhibit acceptable seismic performance despite the presence of horizontal geometric irregularity. Based on the results obtained, the requirement of being horizontally regular for tunnel-form buildings seems to be too conservative at least for the buildings studied herein.

1. The earthquake intensity required for the walls and coupling beams to reach various performance levels was estimated to be several times greater than that of DBE hazard level. Therefore, it is reasonable to expect an elastic behaviour from these structures even under strong ground motions.

2. Based on the probabilistic investigations on 3, 5, 7 and 10-storey tunnel-form irregular buildings, the probability for the coupling beams to reach the performance level of immediate occupancy (IO) is less than 2 and 19% under DBE and MCE hazard levels, respectively. Likewise, the probability of reaching the same performance level for the walls is approximately 0 and 2%, respectively. This indicates that the studied buildings can practically satisfy IO performance level under both hazard levels.
3. Due to the larger seismic demands of coupling beams compared to those of the walls, these elements can act as a seismic fuse in tunnel-form buildings to absorb and dissipate the earthquake input energy, especially in lower seismic intensities
4. For a specific level of intensity, the seismic reliability of tunnel-form buildings is generally reduced as the height (i.e. number of storeys) increases. This trend is especially evident in the case of coupling beams.
5. The governing behaviour of the horizontally irregular tunnel-form buildings studied herein is a flexible torsional mode, in which the torsional response is intensified by increasing in the building's height. Besides, it was found that, in general, the diaphragm rotational displacements increase from the bottom to the top of the structures. Irregularity-induced torsions also intensify the displacement demands in the perimeter parts of the buildings and thus, damages are initiated from those parts.
6. With respect to the greater values of displacement raised by torsion compared to the translational movements, it appears that using the drift at storey mass centre as damage measure (DM) is not appropriate for irregular tunnel-form buildings. In this respect, other damage measures such as flexible edge drift or local damage measures for beams and walls are recommended.
7. Response modification factor of the studied buildings based on the selected hazard levels is smaller than the values estimated for the supply modification factor when the walls reach the life safety performance level. This highlights the fact that such structures exhibit sufficient strength and safety under intense hazard levels. It was shown that considering the code-based response modification factor of 5 for preliminary design of irregular tunnel-form buildings can ensure the structural safety and stability of the buildings under both DBE and MCE hazard scenarios.

8. Parametric analysis on the demand and supply response modification factors indicates that increasing the building's height results in an increase and a decrease in the modification factors originated by ductility and over-strength, respectively. Increasing the building's height, can also transform the shear-dominant behaviour to the membrane and flexural type response in tunnel-form structural systems.

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