



Seismic performance-based assessment of tunnel form building subjected to near- and far-fault ground motions

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Abstract

Existence of pulse at the beginning of the near-fault ground motions with forward directivity produces substantial amount of energy in a short period of time, which may cause brittle behavior in structures affected by a sudden shock. Despite increasing mass constructions of tunnel form buildings (TFBs) all over the world, which may be located in near-fault regions, the lack of comprehensive and cogent investigation related to their seismic behavior is evident. This study is devoted to investigate the seismic performance of 5- and 10-story TFBs subjected to near- and far-fault ground motions through performing incremental dynamic analysis. In addition, their seismic responses considering two design hazard levels under both near- and far-fault ground motions are discussed. Results illustrate that with increasing the construction height and seismic intensity, the influence of directivity on the structural responses including story shear force, drift and damage get more intensified. In comparison with far-fault ground motions, the probability of reaching structural elements to preliminary damage levels is increased up to 20%. It appears in regions with high seismicity, existence of pulse with forward directivity in ground motions decreases structural reliability in high-rise TFBs at predefined performance objectives.

Keywords Tunnel form buildings · Near-fault ground motions · Forward directivity · Fragility curves

Introduction

Recent studies on accelerograms derived from near-fault ground motions in regions prone to high seismicity revealed that in these areas, in addition to the distance of accelerograph station from the fault, its position relative to surface fault rupture is an influential feature for observing the characteristics of ground movements. When the surface rupture extends to the site and slippage of the fault is also in the same direction (forward directivity), proximity of fault rupture velocity to shear wave velocity constitutes a significant energy accumulation in front of the surface rupture which will result in arriving wave front in form of a sudden ground shock motion to the site. The shock causes

an enormous pulse at the beginning of the accelerogram which in regard to quiddity of shear movements is stronger on the perpendicular direction of the fault rather than other directions. Short duration, small wave length and pulse with long period at the beginning of the accelerograms and velocity are the most important hallmarks of near-fault ground motions with forward directivity (Fig. 1). Increasing influence of higher modes on building's responses, increasing in ductility demand and thus increasing the probability of brittle failure in these non-ductile lateral load resisting elements under these types of ground motions are highly imminent (Liao et al. 2000, 2001).

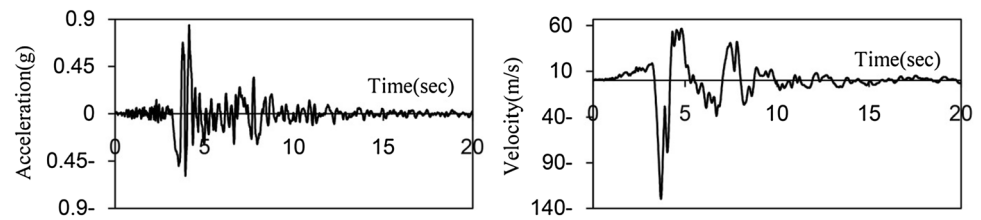
To distinguish the near-fault ground motions containing long-period pulse-like velocity from far-fault ones, personal judgment of seismologists based on visual observation of velocity component of records was utilized as indicator. To this end, recently Backer (2007) and Zhai et al. (2013) presented quantitative methods to distinguish them from each other. Backer followed: (1) apply wavelet transform on original velocity signal, (2) calculate pulse indicator, (3) calculate pulse late arrival time and (4) check

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Fig. 1 Horizontal component of acceleration and velocity for Northridge earthquake, 1994



PGV of original record. In later study, an energy-based approach was proposed to identify those ground motions with dominant pulses observed in the velocity time series; for this purpose, the potential velocity pulse was first extracted with a pulse model. The starting and ending time points as well as period of the velocity pulse were subsequently determined by the peak-point method. Records with peak ground velocities above 30 cm/s^2 and whose dominant velocity pulses hold relative energy values of greater than 0.3 can be satisfactorily classified as pulse-like.

Numerous studies have been carried out in order to evaluate the seismic performance of different systems subjected to ground motions with near-fault ground motions that are noted as follows. Researches by Mortezaei (2012) discovered that mid- and high-rise RC buildings with shear wall lateral load carrying systems under far-fault ground motions are vulnerable to more damages. Moreover, this study revealed that inter-story drift ratio is a fundamental and consequential parameter for these buildings that are subjected to near-fault ground motions with forward directivity.

Investigations of Alavi and Krawinkler (2000) on various buildings illustrated that higher stories of buildings with long period subjected to near-fault ground motions, reach the yielding point faster rather than other stories. Furthermore, during this study, it was determined that increase in strength of building does not solely lead into performance improvements in near-fault areas for all cases. Anderson et al. (1999) studied the effect of pulse-like accelerograms on response of high-rise buildings with dual system in near-fault areas. They concluded that the location of maximum deformations and damage in high-rise buildings depends on the stiffness and strength variations; in such a way that by increasing the rigidity of the structures, position of maximum deformations displaces from higher stories to lower ones. They also concluded that increasing the resistance led to get converse results. Their studies for mid-rise steel moment resisting frames have shown that during near-fault ground motions, seismic responses of buildings such as lateral displacement, story drift ratio and plastic rotation intensified dramatically. Tehranizadeh and Mashkooch Adini (2005) demonstrated that strong pulses with long durations in high-rise buildings with fundamental period higher than 0.8 s that were subjected to near-fault

ground motions could have damaging effect. Tehranizadeh and Labbafzadeh (2005) compared the dynamic response of three-dimensional steel structures using Iranian seismic code for near- and far-fault ground motions. It was presented that structural responses such as displacement, velocity and acceleration of stories, and also base shear and overturning moment for near-fault ground motions were higher than far-fault ground motions. Based on the studies by Ivan (1997), multiplying the base shear coefficient of seismic codes by a coefficient as a factor representing the near-fault effect would not take into account of this phenomenon. Hall et al. (1995) suggested that destructive effects of near-fault ground motions directly depend on the ground movements due to pulse velocity. A comparison was carried out between near- and far-fault ground motions by Huang and Zhu (2003). They demonstrated that the near-fault ground motions cause higher inelastic response in buildings than those of far-fault ground motions. In addition, they agreed with Ivan (1997) that to consider the near-fault effects, modification of design response spectra in seismic design codes is insufficient. They proposed that this process is not capable of taking into account of the increase in inelastic responses. With respect to Baez and Miranda's (2000) studies, maximum inelastic displacement to elastic displacement ratio with constant strength subjected to near-fault ground motions is higher than ordinary earthquakes with period of 0.1–1.3 s. They emphasized that maximum ground velocity is one of the influential parameters impacting this ratio. Kalkan and Kunnath (2007) pointed out to the fact that the seismic demand is a function of pulse period to fundamental period ratio of the building.

Reviewing technical literatures and experience of recent earthquakes suggest increase in seismic demand and the probability of brittle failure mechanisms in lateral load resisting elements in structures subjected to near-fault ground motions. RC slab-wall systems utilizing tunnel form techniques have been implemented for mass construction in large scale throughout the world. The main components of a tunnel form system are composed of relatively thinner shear walls and flat slabs, higher elements stiffness and weight compared to those of traditional RC buildings. In these structures, due to their significant stiffness and resistance, higher weight, smaller lateral load carrying element thickness and participation of higher

modes in structural responses compared to those of traditional RC buildings, it appears that differences between characteristics of performance and damage under near and far-fault ground motions with forward directivity is substantial. With consideration of recent studies [Anudai et al. (2016), Klasanović et al. (2014)], it is evident that only the seismic performance of tunnel form buildings (TFBs) subjected to the far-fault ground motions have been the focus of investigations and their behavioral characteristics subjected to near-fault movements has not yet been discussed. To introduce this type of RC buildings, it is of utmost importance to note that TFBs are categorized as an innovative and modern system which is distinctive from other RC structures due to lack of need for beam and column elements. This type of building is known to merely utilize wall-slab elements as the vertical and lateral load resisting systems which are simultaneously constructed in each floor. Based on the studies carried out by Balkaya and Kalkan (2003, 2004), governing behavioral parameter in these structural systems is characterized as a membrane behavior and the compression–tension performance of all the walls in three-dimensional format plays a key role on load carrying capacity of these structures.

Unfortunately, despite extensive application, existing studies of TFBs are limited and calls for further surveys. Due to insufficient information and knowledge and little experience regarding behavior of structures constructed with tunnel form systems under past earthquakes and the presence of vast number of these constructions in regions with near-fault effects, performance assessment of buildings subjected to ground motions with forward directivity in a more accurate scale is imperative. The present research aims to study and compare the behavior of two TFBs with symmetrical and regular plan by performing incremental dynamic analysis (IDA) subjected to both far- and near-fault ground motions with forward directivity. To assess the seismic performance of these buildings under variations of ground motions' characteristics in a reliability framework evaluation, fragility curves for different damage states are also presented.

Description of prototype buildings

In this study, a prototype plan of TFBs as shown in Fig. 2 is used (Mohsenian et al. 2016). In accordance with the figure, the model buildings have regular and symmetrical plan in both directions. Inner dashed lines are defined as the coupling beams (spandrels) between adjacent walls at openings with length and height of 1 and 0.7 m, respectively. To evaluate the effect of height on the behavior of structural components, 5- and 10-story buildings were selected. The model buildings were assumed as residential

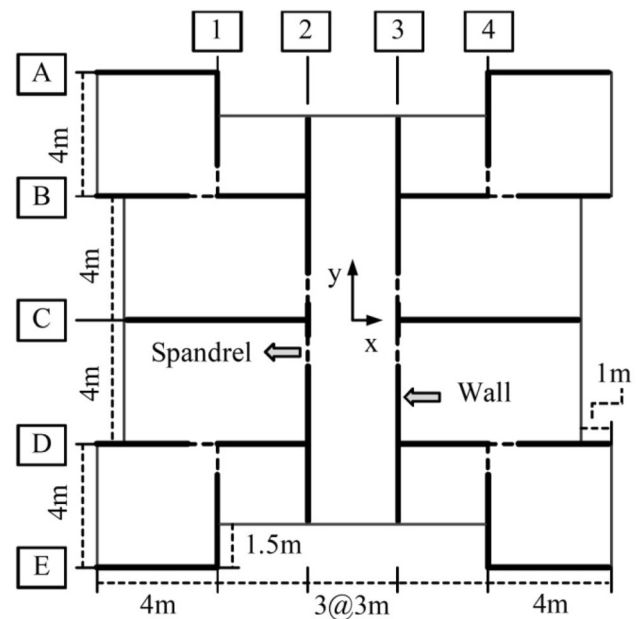


Fig. 2 Plan view of prototype buildings

buildings located in Tehran region and on soil type B [$375 \text{ (m/s)} \leq V_s \leq 750 \text{ (m/s)}$] in a high seismicity area using ISIRI-2800 (2005) provision with story heights of 3 m for each floors.

Gravity loads include dead load which is equal to 6.4 kN/m^2 for all stories, and live load which is equal to 2 kN/m^2 for all stories and 1.5 kN/m^2 for the roof. The other assumptions of this study are the out-of-plane elastic behavior of the walls, the rigid diaphragm for the floors, the fixed base of the walls in the first story, ignoring probable uplift of footings, and disregarding the sliding of the reinforcement bars in concrete.

Since the TFB is a new structural system, there is not much information about its seismic performance against past earthquakes. Currently, this system is commonly known as a “RC bearing wall system” and the behavior factor is considered as $R = 5$ for ordinary reinforced concrete shear walls, based on ISIRI-2800. According to ISIRI-2800, seismic lateral base force for regular buildings up to 50-m height by the equivalent static analysis method is obtained from the following formula:

$$V_b = CW; C = \frac{ABI}{R}, \quad (1)$$

where V_b is the shear force at base level, C is the seismic coefficient, W is the total seismic weight of building, A is the design base acceleration ratio (0.35 for Tehran), B is the building response factor determined from the design response spectrum, I is the importance factor (1 for residential buildings), and R is the building behavior factor (5 for ordinary reinforced concrete shear walls).

The fundamental period of vibration, T , shall be determined from the following formula:

$$T = 0.05H^{3/4}. \quad (2)$$

The suggested formula for the lateral load pattern specified is defined as

$$F_i = \frac{W_i H_i^k}{\sum_{j=1}^N W_j H_j^k} V_b, \quad (3)$$

where F_i is the lateral force of level i , W_i is the seismic weight of level i , H_j is the height of the story of level j , and N is the total number of stories above the base level

For the case study tunnel form structure with a natural period of vibration T equal or less than 0.5 s the value of k is equal to 1.

The prototype buildings were designed primarily in accordance with ACI318-14 (2014) using structural design software ETABS [Computers and Structures Inc (CSI) (2008)] satisfying all requirements demanded by the reference. Slabs having 150 mm thickness and 200 mm thickness were assigned to walls with two layers $\phi 8@200$ reinforcements in both vertical and longitudinal directions (for the first four stories of taller building $\phi 12$ were used for the longitudinal reinforcements). The spandrels were also assigned with diagonal reinforcements to provide ductility and enhancement in shear strength (ACI 318–14 2014).

Analytical modeling

The finite element-based program PERFORM-3D (Computers and Structures Inc. (CSI) 2007) was utilized for non-linear analysis and shear walls elements from the library of the software were assigned for the walls and spandrels in the analytical modeling. The shell elements have four nodes and 24 DOFs. The section properties of wall elements were modeled with fiber technique. The concrete and steel rebars were discretized into longitudinal fibers with predefined one-dimensional stress–strain constitutive law. The concrete in boundary zones at two ends of each wall where confined by stirrup ties was modeled with confined model and elsewhere was modeled with unconfined behavior. The concrete and steel rebar behavior was modeled using Mander et al. (1988) and Esmaily and Xiao (2005) stress–strain relationship, respectively. The tensile strength of concrete was ignored. When a fiber cross-section cracks, it propagates across the cross-section, perpendicular to axial axis of the wall and the neutral axis shifts. These fibers are stiff in compression, and after cracking have no stiffness in tension.

The concrete and reinforcement properties in finite element model are shown in Table 1. The ratio of height to length has a major influence on flexure and shear dominant behavior of the walls. For the walls with aspect ratios more than 3, the governing parameter is flexural and those with ratios less than 2, is shear (Mac Gregor 2002). Satisfying all requirements of supplementary guidelines such as Building and Housing Research Center of Iran and also Turkish standard in providing minimum ratio of walls area to total floor area of 3% in both X - and Y -direction requires to provide long-length walls in the design of proposed plan. Thus, shear in most of the walls (except for a few walls where the governing criteria are bending) and in all coupling beam controls their inelastic behaviors. In these elements, bending behaves linearly. The walls and spandrels that were governed by shear, relative lateral deformation and chord rotation were selected as the damage criteria. In the present study, to model the inelastic shear behavior of the elements based on ASCE41-13 (2014), nominal shear strength of cross sections of the elements were considered as ultimate strength. It should be noted that to estimate the nominal shear strength of spandrels, acceptance criteria and non-linear parameters of primary type element for spandrels were utilized. It should further be noted that elastic out-of-plan behavior of walls, floor rigid diaphragm, fixed base connections for first story walls, ignoring soil–structure interaction and reinforcements slippage were other assumptions of this research.

Non-linear responses to earthquake excitation

Examining the wall percentage in plans with respect to the data presented in Table 2 related to the translational modal properties, reveals that the stiffness and strength in longitudinal direction of plan (x -direction) is greater than transverse direction (y -direction). Therefore, case study buildings were analyzed only in transverse direction of plan. Despite symmetrical nature in both primary

Table 1 Material properties

Mechanical property	Steel rebars	Concrete
Young's modulus (MPa)	200,000	26,800
Poisson's ratio	0.3	0.2
Yield stress	400 MPa	–
Ultimate stress	600 MPa	–
Tensile strength	–	0
Compressive strength (MPa)	–	25 MPa

Table 2 Mode properties of buildings

	5-story structure		10-story structure	
	T^a (s)	M^b (%)	T (s)	M (%)
First mode (torsional)	0.2235	0	0.7485	0
Second mode (transitional in transverse direction of plan)	0.1397	79.6	0.4453	75.41
Third mode (transitional in longitudinal direction of plan)	0.1342	74.02	0.3187	67.35
Fourth mode (torsional)	0.05162	0	0.1482	0
Fifth mode (transitional in longitudinal direction of plan)	0.0414	19.51	0.09576	20.87
Sixth mode (transitional in transverse direction of plan)	0.03672	14.18	0.09502	14.47

^aPeriod^bThe modal mass participation factor

directions, participation of torsional modes in structural response is substantial. An interesting observation is that the first mode for both structures is torsional, not translational. This phenomenon should be attributed to the elimination of peripheral wall areas which is due to the special implementation of this system and the need to remove the molds after setting and hardening of concrete from structure's peripheral faces.

Record selection

To perform successive response history analyses, an ensemble of 14 ground motion records compatible with the building's soil condition type B from USGS classification [$360 \text{ (m/s)} \leq V_s \leq 800 \text{ (m/s)}$], were selected from the PEER strong ground motion database (2018). Seven records were from far-fault ground motions and the rest are from near-fault ground motions with forward directivity (with pulse) based on FEMA P695 classification (FEMA 2009). After plotting and comparing the spectral response for each couple set of ground motions, primary accelerogram components were selected based on higher spectral values as the building's vibration frequency to perform IDA. It should be mentioned that minimum number of records based on seismic design codes was employed because of the time-consuming procedure of each non-linear analysis (about 10 h to complete each analysis using intel core i7-3.4 GHz). Selected ground motion records summarized in Table 3.

Incremental dynamic analysis

To incorporate the variation of the amplitude, frequency contents and duration of probable future ground motions (record-to-record variability) on seismic response of the structures, incremental dynamic analysis (IDA) was conducted. IDA has been widely used to determine the

demands and capacity of structures in multiple seismic intensities and limit states. The seismic collapse capacity of a structural system can be determined by IDA (Vamvatsikos and Cornell 2002). An IDA curve involves a series of non-linear dynamics time history analyses of the structure subjected to a ground motion of increasing intensity. In addition, an ensemble of ground motion records, each record in the ensemble being scaled to multiple levels of intensity with respect to the specific intensity measure was often utilized to apply on structure model. The scaling levels of seismic intensity are appropriately selected to force a building undergoing the entire range of behavior, from elastic to inelastic and finally to global dynamic instability in the form of large engineering demands (e.g., inter-story drift), indicating the collapse of the building. Maximum inter-story drift ratio and chord rotation among the common parameters are chosen for estimating damage measure. For intensity damage parameter, spectral acceleration $S_a(T_1, 5\%)$, at fundamental elastic natural period among other intensity measures was chosen. It was indicated that both advantages of efficiency and sufficiency in selection of S_a as an intensity measure are maintained. The IDA curves present the maximum inter-story drifts when the building is subjected to increasing levels of ground motion intensity. Based on the IDA curves, the structural collapse capacity is determined as the last point on the IDA curve that is larger than the 20% of the initial tangent slope of the IDA curve or dynamic instability occurred (Vamvatsikos and Cornell 2002).

The IDA curves were developed using successive dynamic analyses for the prototype buildings subjected to both near- and far-fault ground motions and are shown in Fig. 3. The IDA percentile curves are also shown in Fig. 4. Results illustrate that this structural system in comparison with other systems exhibit high seismic capacity.

To compare the performance levels of different members of the structure, the roof drift ratio which is characterized as the average drift in which the relevant performance levels exceeded at the walls and spandrels are

Table 3 Properties of seismic ground motions used in the analysis

No.	Earthquake and year	Station	M_w	R^a (km)	PGA (g)
Far-fault records					
1	Cape Mendocino, 1992	Eureka—Myrtle and West	7.01	44.60	0.1782
2	Northridge, 1994	Hollywood—Willoughby Ave	6.69	25.70	0.2455
3	Northridge, 1994	Lake Hughes #4B—Camp Mend	6.69	32.30	0.0629
4	Cape Mendocino, 1992	Fortuna—Fortuna Blvd	7.01	23.60	0.1161
5	Northridge, 1994	Big Tujunga, Angeles Nat F	6.69	24.00	0.2451
6	Landers, 1992	Barstow	7.28	36.10	0.1352
7	San Fernando, 1971	Pasadena—CIT Athenaeum	6.61	31.70	0.1103
Near-fault records					
1	Cape Mendocino, 1992	Petrolia	7.01	4.500	0.6624
2	Chi—Chi, 1999	TCU102	7.6	45.60	0.2978
3	Supper station hills, 1987	PTS	6.54	16.00	0.4549
4	Kocaeli, 1999	Izmit	7.51	5.300	0.2195
5	Landers, 1992	Lucerne	7.28	44.00	0.7268
6	Loma Prieta, 1989	Saratoga—Aloha Ave	6.93	27.20	0.5125
7	Northridge, 1994	Sylmar—Olive view	6.69	9.000	0.8433

^aClosest distance to fault rupture

Fig. 3 IDA curves for both near- and far-fault ground motions; **a** 5-story building; **b** 10-story building

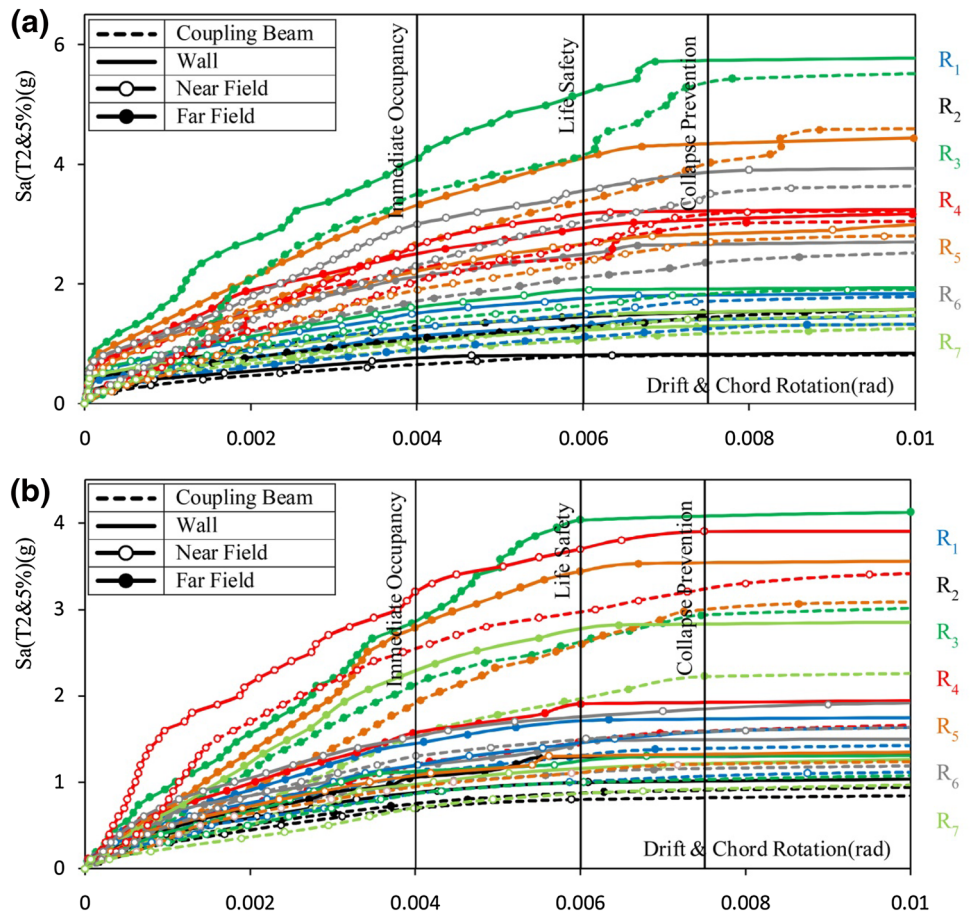
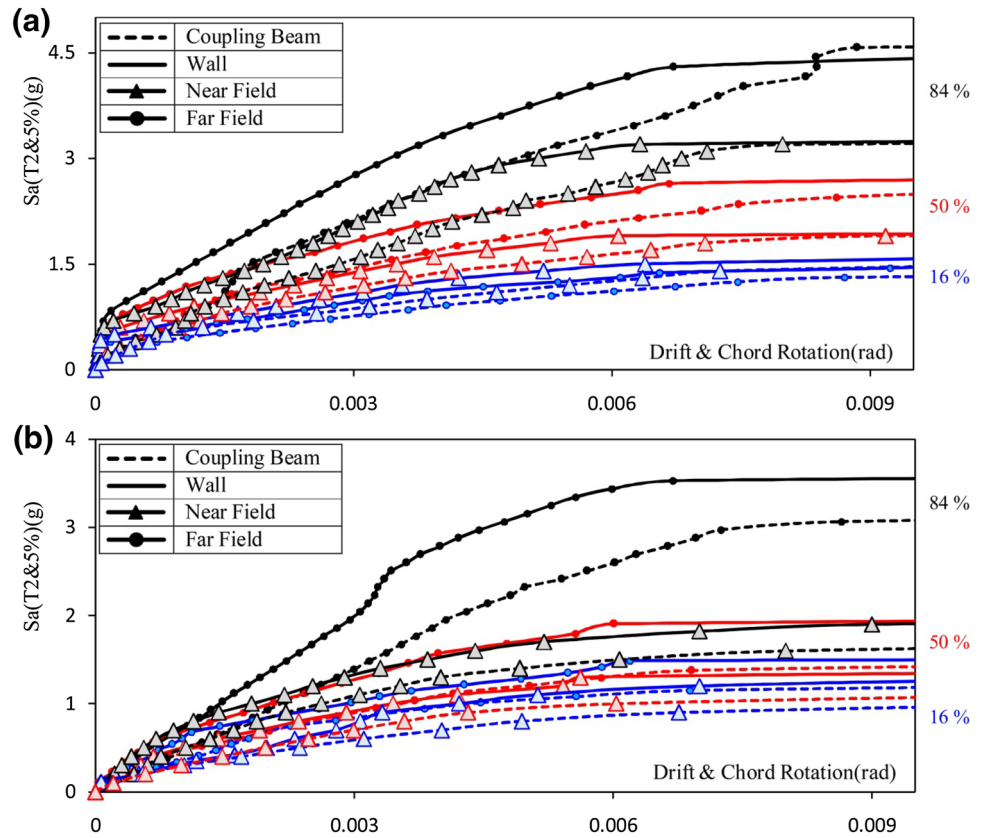


Fig. 4 16th, 50th and 84th percentile of IDA results; **a** 5-story building; **b** 10-story building



recorded in pushover analysis and illustrated in Fig. 5. For the performance evaluation of the case study construction with considering the limit states of immediate occupancy (IO), life safety (LS), and collapse prevention (CP), the target displacement of each construction is calculated by average values of the resulted multiple-strip dynamic analyses (ASCE 2014; Jalayer 2003) at each intensity of the design basis earthquake (DBE-return period of 475 years, PGA 0.35 g) and the maximum considered earthquake (MCE-return period of 2475 years, PGA 0.55 g) hazard levels and compared with the displacement values corresponding to the performance levels obtained from numerical acceptance criteria for non-linear procedures in RC walls and spandrels. All spandrels have aspect ratio (length/section height) less than 2. Therefore, they are characterized as deep beams and the shear deformation is dominant (against to the flexural deformation).

Figure 5 attempts to compare the target displacements and the displacements corresponding to each performance level which are illustrated as vertical lines on the base shear roof drift diagrams obtained from non-linear static analysis of each construction. Surveying the figures shows that the 5- and 10-story TFBs are in IO performance level under both types of earthquake ground motions in both walls (W) and spandrels (SP). Therefore, the results show

an appropriate seismic performance of the case study constructions.

It can be deduced that taller building show lower seismic capacity at particular performance levels (Fig. 5). These results are explained through the following discussions.

Pushover analyses are performed on two constructions (5- and 10-story) with and without spandrels. By comparing the diagrams, it is concluded that the spandrels do not have a significant effect on the ductility and the strength of the lower height case-studies in comparison with taller one. As can be seen, through increasing height, their contribution becomes more significant. Figure 6 confirms this observation. The induced shear deformation on spandrel was calculated from the inter-story drift (θ) and the associated chord rotation (β). The chord rotation was accrued from the racking deformation. Since the walls are constrained by the in-plan rigid floors, they deflect laterally by the same amount at the correspondent floor level. As the rotations of the walls are equal to the rate of change of lateral deflection with height, the walls also rotate by the same amount at the same floor level. Hence, the rotations of the two ends of a spandrel should be equal, as illustrated in Fig. 6. The shear strain γ can be estimated as

$$\gamma = \theta + \beta, \tag{4}$$

Fig. 5 Seismic demands of case study TFBs from pushover analysis corresponding to limit states; **a** 5-story building; **b** 10-story building

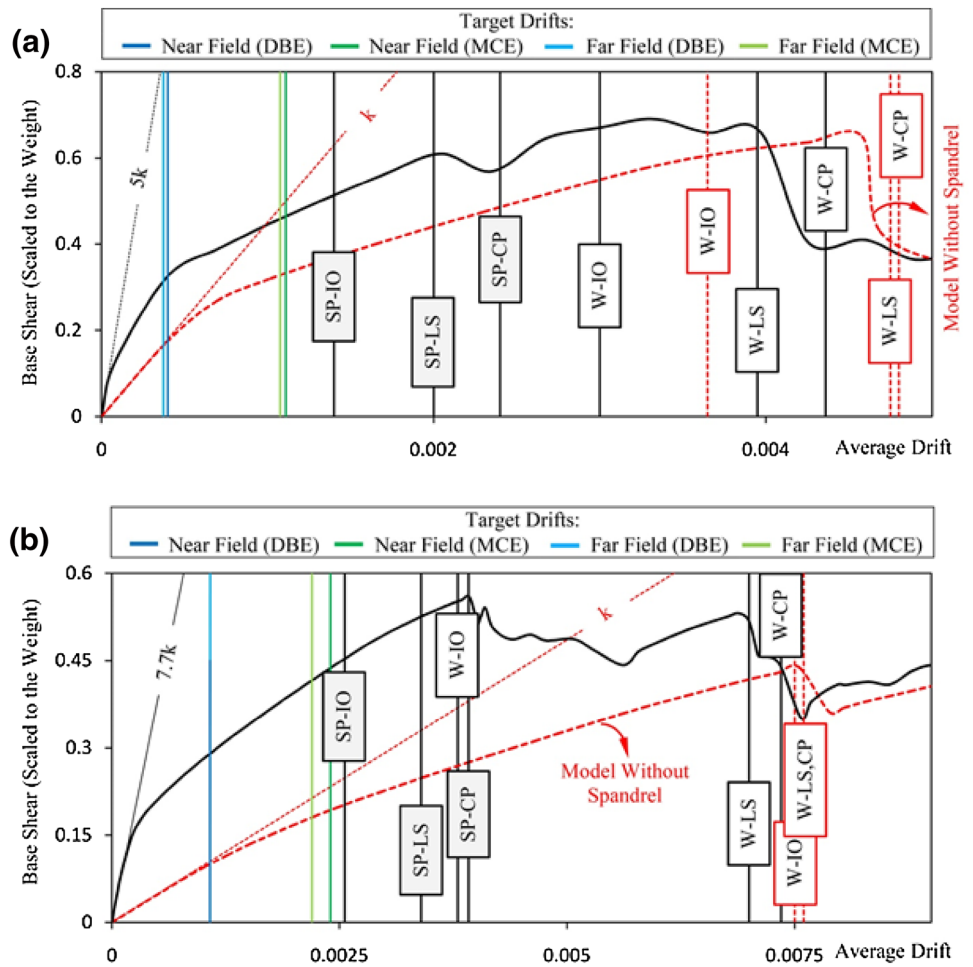
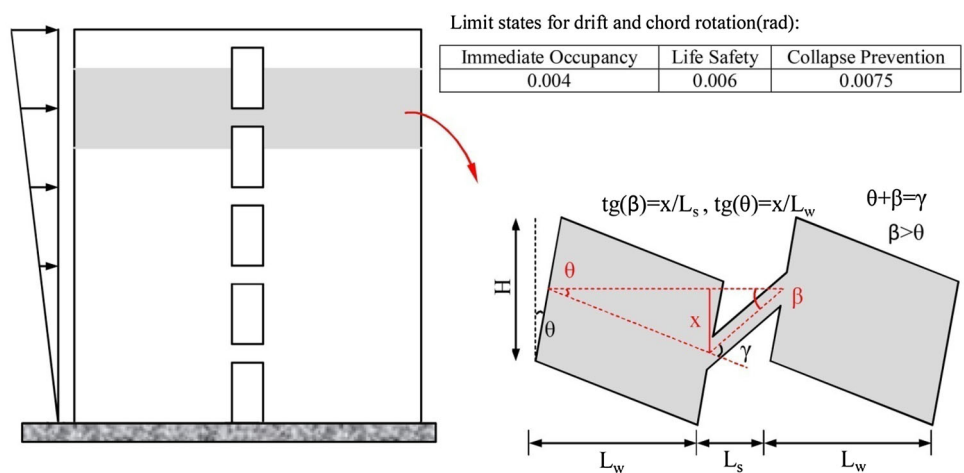


Fig. 6 Distortion demand on the coupling beams (schematic)



As can be seen in Eq. (4), for low-rise construction, the low shear deformation demands are applied due to low chord rotation and with increasing height the chord rotation, and hence shear deformation demand becoming greater. It can be concluded that shear deformation demand and its effect on overall stiffness and ductility of

construction is increased in taller ones. Therefore, for taller constructions, regarding more seismic demand, the spandrels under taken more shear distortion demand and when having proper design, they may be used as the structural energy-dissipating fuses.

Fig. 7 The mean maximum inter-story drifts under DBE and MCE ground motions

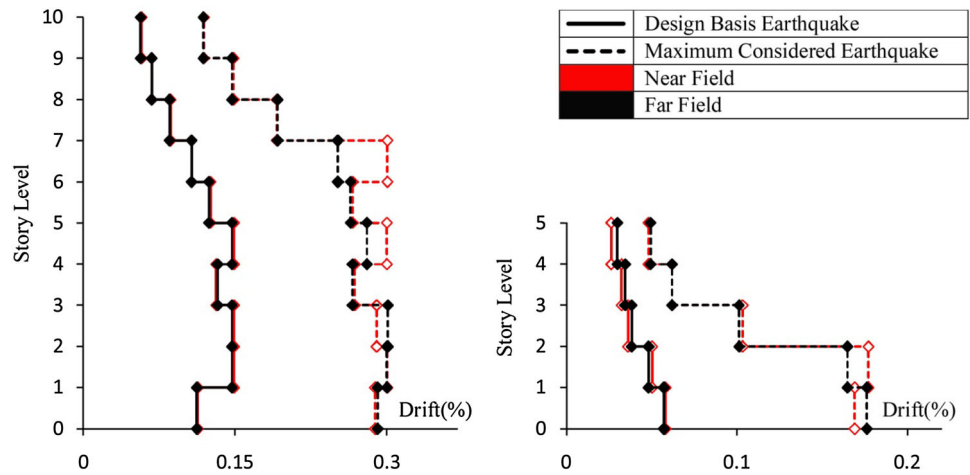
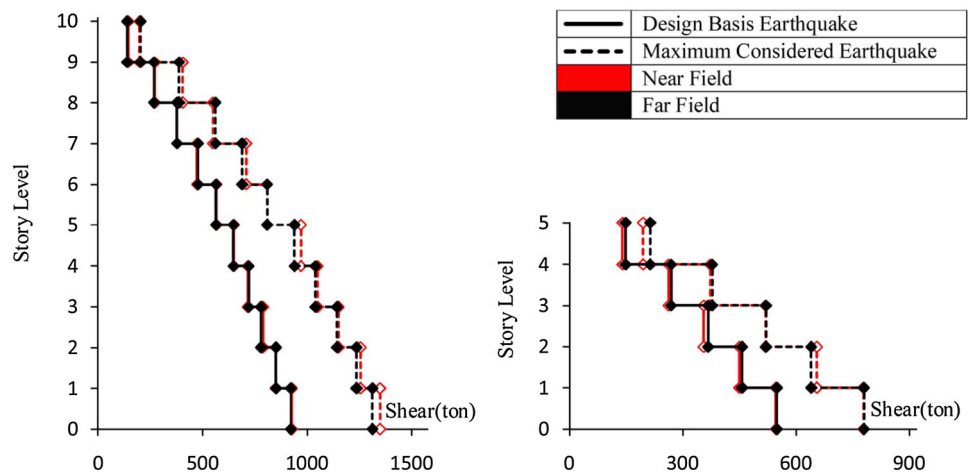


Fig. 8 The mean maximum story shear force under DBE and MCE ground motions



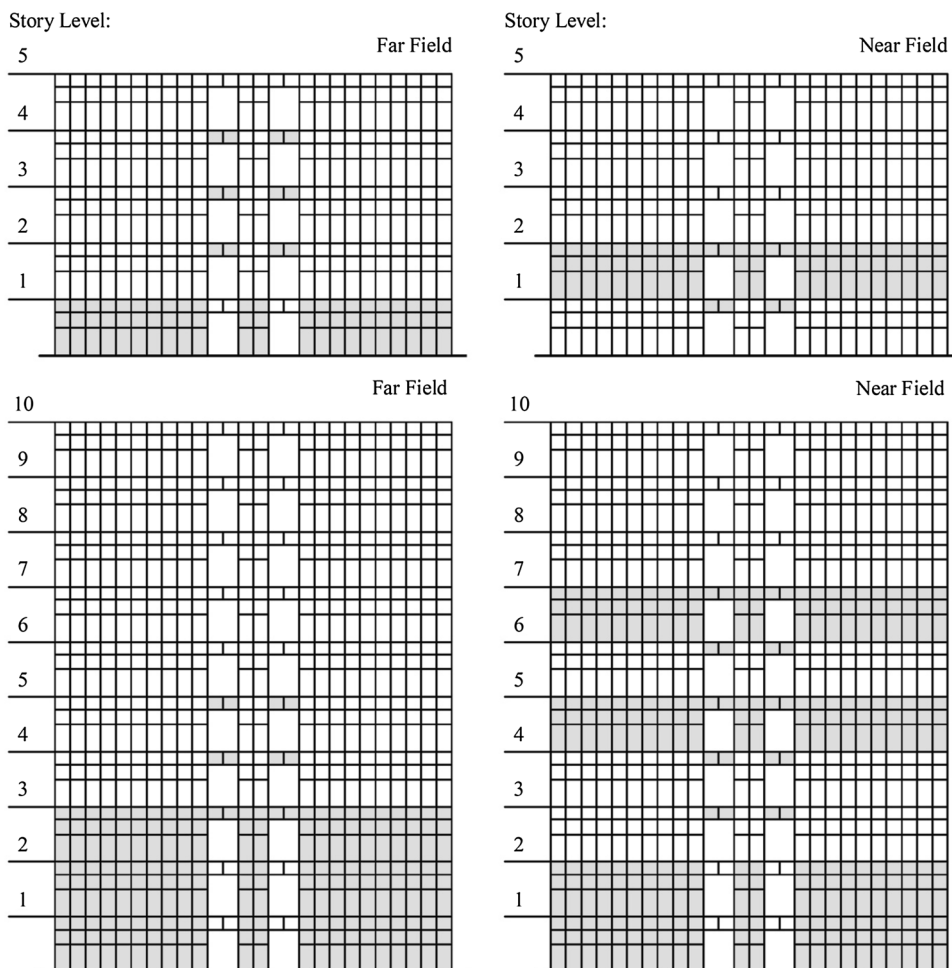
The mean values of maximum inter-story drift ratio demands and maximum story shear forces for both near- and far-fault ground motions were obtained at above-mentioned intensities and compared in Figs. 7 and 8. Results reveal that at DBE intensity, differences in story shear forces and inter-story drift ratios for near- and far-fault ground motions are intangible to type of ground motions. Under the MCE intensity level, except in couple stories (second story of the 5-story building, fifth and seventh story of the 10-story building), seismic response curves overlap in both near- and far-fault ground motions. At this seismic intensity level, the mean value of maximum inter-story drift ratio of the 5- and 10-story buildings under near-fault ground motions are 7 and 19% greater than values of far-fault ones, respectively. This result can be attributed to the propagation of pulse-type wave inside the structure (similar bullwhip effect) and the influence of higher modes. In case of buildings exposed to pulse-type ground movements, bullwhip effect could lead into local or large inter-story deformation. It can be seen that the stiffness and strength of this type of buildings are high to some

extent, so that for lower intensity levels, ground motions with directivity are not able to instigate higher modes. It is apparent that the structure responses are also intensified by increasing height and ground motion intensity subjected to near-fault pulse-type ground motions. In a comparative evaluation, it can be noted that story shear forces with respect to inter-story drift ratios possess lower sensibility to near-fault ground motions having directivity effect.

Influence of the near-fault with directivity pulses on onset of failures in the walls and coupling beams of understudied buildings in comparison with far-fault ground motions was evaluated. The first failure of structural members (gray elements) subjected two types of ground motions were determined and shown in Fig. 9. It can be seen that the failure mode and damage initiation of constructions are affected by existence of pulses in applied accelerograms in buildings.

In both buildings and for both near- and far-fault ground motions, initial failures occur in walls depicted at axis 2 and 3 of plan, and the coupling beams perform as failure fuses in buildings. The altitude location of failure onset in

Fig. 9 The wall elements experience maximum damages under MCE ground motions (axis 2 and 3 of plan)



elements for these axes for both types of ground motions apparently contrasts. In both buildings under near-fault in comparison with the far-fault ground motions, due to increasing drift ratio in upper stories regarding existence of rigorous pulses, failures in walls, which are the primary load carrying elements, shift to upper stories. This phenomenon is even more pronounced in taller construction owing to higher mode effect.

Fragility curve development

One of the primary challenges for engineers has been, and still is, the ability to reliably predict intrinsic uncertainty in the properties of loads and structural component characteristics subjected to seismic excitation and to incorporate these properties into analysis tools. In these circumstances, identifying the building's performance in terms of statistics and probability to consider uncertainty is of great importance. After the structural collapse, the capacity is determined by the IDA curves, a lognormal distribution is fitted

to generate the collapse fragility curve using S_a as a random variable

If R , LS_i , IM and im_i stand for seismic responses of structure, limit state corresponding to R , earthquake intensity measure and given intensity, respectively, thus the seismic fragility, which is a conditional probability distribution function, can be shown as (Cimellaro et al. 2006):

$$\text{Fragility} = P[R > LS_i | IM = im_i]. \quad (5)$$

Developing fragility curves demand a specific probability analysis and based on desired accuracy, numerous methods are available (Khalvati and Hosseini 2008).

Due to inherent high stiffness and strength of TFBs and remaining a significant portion of structure in elastic region, record-to-record variability is the main source of uncertainty in comparison with those of other uncertainties relevant to analytical modeling (Alamilla and Esteva 2006). Hence, in this study, incremental dynamic analysis (IDA) utilizing a couple of seismic ground motion records consistent with seismic hazard level of the specified site is employed. This analytical method considers the uncertainty

Fig. 10 Fragility curves for various performance levels in elements (5-story building). **a** Coupling beams. **b** Walls

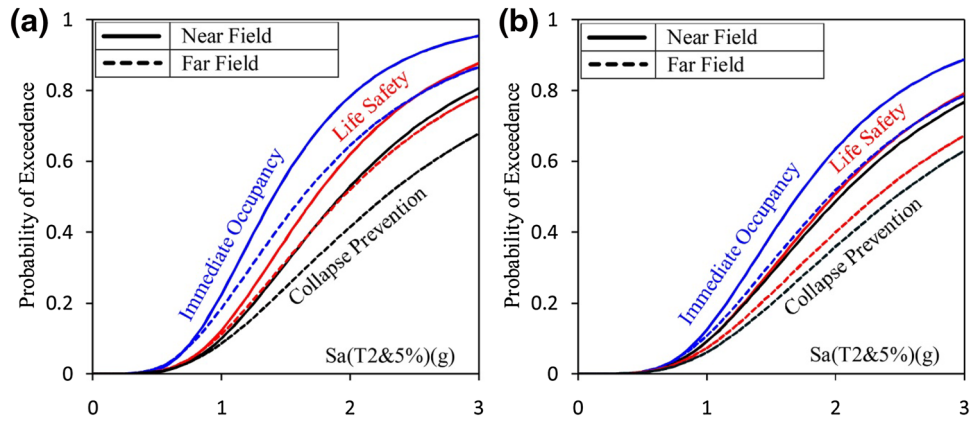
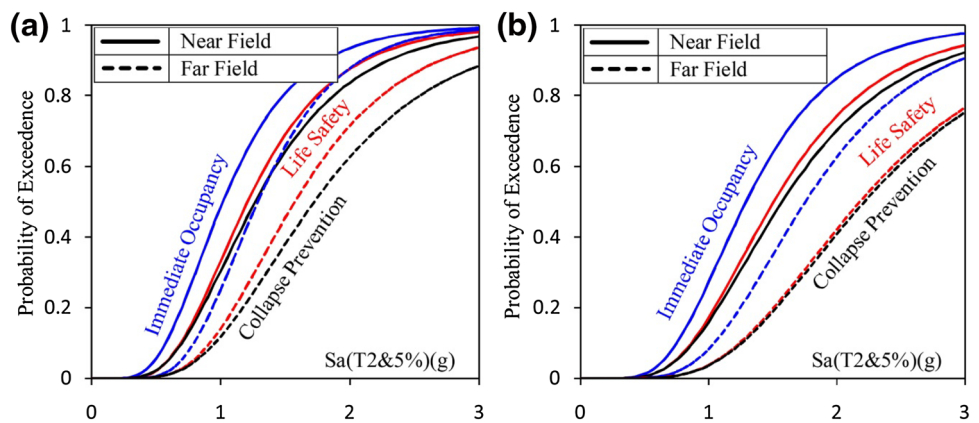


Fig. 11 Fragility curves for various performance levels in elements (10-story building). **a** Coupling beams. **b** Walls



pertaining to characteristic of ground movements including variability in amplitudes, frequency contents and durations.

Chord rotation and inter-story drift ratio are considered as seismic demand and capacity at each performance level for the coupling beams and walls, respectively, according to the corresponding values in ASCE 41-13 (2014). The fragility curves for case study constructions are presented in Figs. 10 and 11 based on computing the probability of exceeding triple limit states, i.e., immediate occupancy (IO), life safety (LS) and collapse prevention (CP) for far- and near-fault ground motions. Higher probability of exceedance at the entire performance levels for near-field ground motions in comparison with far-fault ones are <http://www.thesaurus.com/browse/conspicuous> evident. Although the difference between the probability of exceedance of the coupling beams at two LS and CP is obvious, but the probability of exceedance for the walls at these performance objectives, especially for taller archetypes, are closer in values.

The probability of exceedance at two seismic hazard levels, i.e., design-based earthquake (DBE 10%/50 years) and maximum considered earthquake (MCE 2%/50 years) for two types of earthquakes are shown in Tables 4 and 5. Comparing the probabilities in these tables indicates that

Table 4 Fragility curve quantities (5-story)

Performance level	IO	LS	CP
Near-fault			
Coupling beam			
DBE	16.0	8.29	6.82
MCE	43.0	27.2	22.0
Wall			
DBE	8.41	6.28	6.16
MCE	27.9	20.5	20.0
Far-fault			
Coupling beam			
DBE	13.8	7.96	6.10
MCE	33.8	23.1	17.5
Wall			
DBE	7.56	5.10	4.20
MCE	22.5	15.9	13.6

All values are in percentage

the coupling beams reach to each limit state much sooner than walls. This phenomenon is more obvious for the taller building. This conclusion is discussed through Eq. (4)

Table 5 Fragility curve quantities (10-story)

Performance level	IO	LS	CP
Near-fault			
Coupling beam			
DBE	39.2	25.0	23.0
MCE	70.8	56.6	52.1
Wall			
DBE	19.7	11.8	11.0
MCE	50.6	36.7	33.8
Far-fault			
Coupling beam			
DBE	17.2	9.23	7.86
MCE	50.9	32.5	26.9
Wall			
DBE	5.02	2.19	2.08
Maximum probable earthquake	23.0	11.9	11.4

All values are in percentage

Table 6 The probability of exceedance ratio for near- and far-fault ground motions

Performance level	IO	LS	CP
5-story building			
Coupling beam			
DBE	1.16	1.04	1.12
MCE	1.27	1.18	1.25
Wall			
DBE	1.11	1.23	1.46
MCE	1.23	1.29	1.47
10-story building			
Coupling beam			
DBE	2.28	2.71	2.93
MCE	1.39	1.75	1.94
Wall			
DBE	3.92	5.39	5.29
MCE	2.20	3.07	2.97

where chord rotation increased as the building's height increases.

Table 6 compares the probability of exceedance ratio for near- and far-field ground motions at triple limit states for two archetypes. According to Table 6, the main conclusions are the following.

At DBE hazard level and under near-fault ground motions, the probability of exceedance for the walls and coupling beams of the 5-story building for IO performance level are 11 and 16% greater than far-fault ground motions,

respectively. In the same case, at MCE hazard level, these percentages are modified to 23 and 27%, respectively.

At DBE hazard level and under near-fault ground motions, the probability of exceedance for the walls and coupling beams of the 10-story building for IO performance level are 292 and 128%, greater than far-fault ground motions, respectively. In the same way at MCE hazard level, these percentages are modified to 120 and 38.8%, respectively.

As a general conclusion, under both near- and far-fault ground motions, the buildings stand for IO performance level in DBE hazard level. In MCE, the low-rise building exhibits the same performance level for both types of ground motions, but the taller building shows different performance level. In this building, under forward directivity near-fault ground motions, the performance displays lower level of life safety.

Conclusions

The present work explains the effect of near- and far-fault ground motions with forward directivity (pulse) on seismic performance of tunnel form buildings as a part of comprehensive investigation to give insights of the important parameters affecting their response by an analytical study. It focuses on studying the existence of pulse-type effect on ground motions imposed to TFBs and whether the presence of forward directivity increase peak (transient) and (residual) permanent drift demands in frame models with different number of stories. For that purpose, a set of 14 earthquake ground motions recorded in far-field and near-fault accelerographic stations were considered in this study. The following conclusions are drawn from this investigation:

- Tunnel form system presents high capacity and seismic reliability subjected to both far and forward directivity near-fault ground motions. Constructing high percentage of structural walls in plan provides sufficient seismic capacity under wide range of high seismic intensities.
- In DBE hazard level, the seismic response trend of buildings under far- and near-fault ground motions are similar. The difference between responses and accumulated damages in archetypes subjected to far-fault and forward directivity near-fault ground motions appears to be significant at higher intensity levels. Amplifying powerful velocity pulses incites higher modes in taller building and propagating deformation such as wave pulses through construction height induces significant inter-story drifts.

- With increasing height of TFBs, the membrane elongation of walls is intensified, which will lead into inducing larger shear distortion demand on spandrels between them. Therefore, these elements would be able to dissipate significant amount of earthquake energy if designed in ductile manner. As a consequence, for taller construction the special design of spandrels as the structural fuses for seismic loading is noteworthy.
- TFBs are more vulnerable when subjected to near-fault ground motions rather than far-fault ground motions, especially for taller constructions. Lack of desired ductility capacity as well as high stiffness and strength of this type of construction system may be considered as reason for this different behavior.
- It should be noted that although the coupling beams are more vulnerable than the walls, at lower performance level (LS and CP levels) for 5-story and at all performance levels for 10-story walls, the destructive effects of near-fault ground motions in respect to far-fault ones in walls, are more significant than in coupling beams.

Finally, it must be acknowledged that special attention should be devoted to the design of RC structures with tunnel form system, and near-fault zone subjected to intensive ground motions with forward directivity.

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