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Computational analysis of three dimensional steel frame structures through different stiffening members

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Abstract. Ground motion records are commonly used for fragility curves (FCs) developing utilized in seismic loss estimating analysis for earthquake prone zones. These records could be 'real', say the recorded acceleration time series or 'simulated' records consistent with the regional seismicity and produced by use of alternative simulation methods. This study has focused on fragility curves developing for masonry buildings through computational 'simulated' ground motion records while evaluating the properness of these fragilities compared to the curves generated by the use of 'real' records. Assessing the dynamic responses of structures, nonlinear computational time history analyses through the equivalent single degree of freedom systems have been implemented on OpenSees platform. Accordingly, computational structural analyses of multi-story 3D frame structures with different stiffening members considering soil interaction have been carried out with finite element software according to (1992) Earthquake East-West component. The obtained results have been compared to each frame regarding soil interaction. Conclusion and recommendations with the discuss of obtaining findings are presented.

Keywords: ground motion; masonry buildings; soil Interaction; 3D frame structure; finite element

1. Introduction

The prediction of structure damage under future earthquakes is essential in structural design (Park *et al.* 1985). The main purpose of structural engineering is to design economic and safe structures toward any earthquake (Carreras *et al.* 2011, Aghaee *et al.* 2014, Mohammadhassani *et al.* 2014b, Toghroli Ali *et al.* 2014, Ying *et al.* 2015, Asgarian *et al.* 2016, Shahabi *et al.* 2016a, Khorramian *et al.* 2017a, Qiu *et al.* 2018, Shariati *et al.* 2018). A certain degree of damage is normally expected, unless the design would be a high cost. There are many places as a seismically active area where located in a complex zone of collision (Safa *et al.* 2016c, Mansouri *et al.* 2017, Akinpelu *et al.* 2018, Mahdi *et al.* 2018). A recent study by (Jalali *et al.* 2012) has lately studied the seismic performance of structures with pre-bent strips as a damper in some regions such as Many countries by use of simulated ground motion records through one approach to derive FCs. Purposefully, it is for developing FCs for masonry building stock into a specific area by use of various approaches using regionally 'simulated' ground motion data-sets, also for comparing these fragilities with the curves obtained by use of global 'real' records (Ataei *et al.* 2015, Zahrai 2015, Chu T.H.V. *et al.* 2016, Olivier *et al.* 2017). Numerous cases of failure or damage of buildings have been induced by more failure or displacement across recent earthquakes. To prevent this, buildings need to be accurately designed for the dynamic loading caused by earthquakes (Goodarzi *et al.* 2009, Safa *et al.* 2016a, Abedini *et al.* 2017). One the most risk areas with sparse ground motion networks is selected for this study. In order to gain FCs through various methods, local building data-set obtained from a walk-down survey are used. Due to the masonry classes of many building structures in this location (57%), fragility is gained, then compared to this type of buildings. In this study, FCs have been gained due to the high alignment between nonlinear responses and PGA of masonry building stock (Shariati *et al.* 2020a). In earthquake resistant structural design, structural form choosing is crucial. Such a design characteristic should be evaluated as a) simplicity and symmetry, b) regularity and continuity, c) plan and cross sectional shape, d) stiffness, e) ductility and f) soil conditions. There are many references in which parameters affecting earthquake behavior on many structural types are discussed (Arabnejad Khanouki *et al.* 2010, Khorami *et al.* 2017d, Zandi *et al.* 2018). In this study, the responses of buildings for different distributions of stiffness and strength have been evaluated. Several researchers have studied the increasing of rigidity to decrease the horizontal

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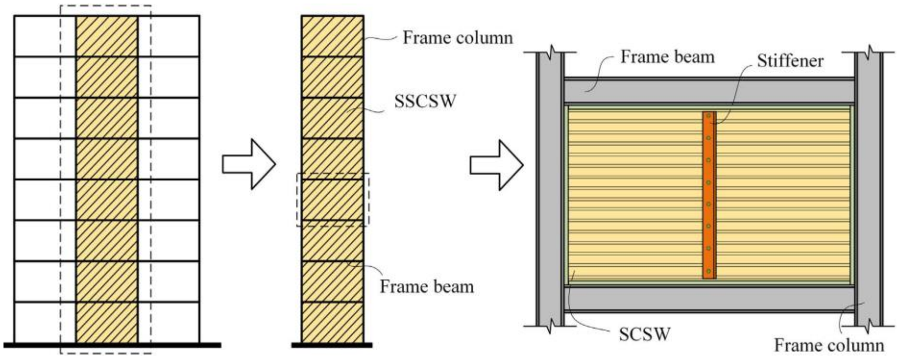

Fig. 1 Elements in frame structures

Fig. 2 Location of earthquake

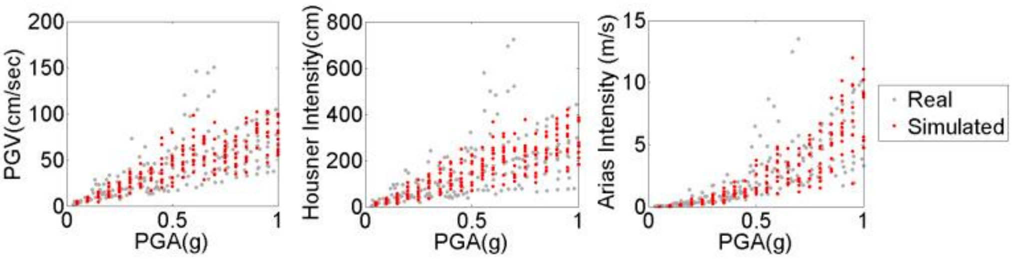

Fig. 3 Distribution of ground motion parameters for simulated and real records

displacement of structure(s) subjected to earthquake (Khorami *et al.* 2017a, Khorami *et al.* 2017c). Accordingly, the structural member is added to moment resistant frame. To solve the deficiencies for lateral displacements, some structural members and systems such as shear walls, tube structures, core assembly, braced frame and hybrid structural systems are become popular (Zahrai 2015, Mansouri *et al.* 2016, Shafaei *et al.* 2016). According to (Hamidian *et al.* 2011, Shariati *et al.* 2011a), the vertical pre-compression loads are able to be effectively released for 3D frame structure with their corrugations laid horizontally. Following the benefits of 3D frame structure, it is practically applied in the structures of building as lateral force resistance systems (Fig. 1). The shear resistance of 3D frame structure with small thickness (i.e. t = 0.5 − 1.5mm) and many numerical and empirical studies have been studied (Shariati *et al.* 2011a, Shariati *et al.* 2011b, Tahmasbi *et al.* 2016).

2. Methodology

2.1 Ground motion records

The impact of input risk in final FCs is studied by the use of two diverse (real and simulated) ground motion record sets. To provide regional specific ground motion dataset in this article, ground motion simulations are used. Considering various earthquakes with M_w=5.0, 5.5, 6.0, 6.5, 7.0, and 7.5 similar to those in Fig. 2, event in 1992 (M_w=6.6) are simulated by the use of stochastic finite-fault method (Arabnejad Khanouki *et al.* 2010, Jalali *et al.* 2012, Khorami *et al.* 2017d). To show the obtained records' features, they are baseline corrected and filtered with a 4th-order bandpass Butterworth filter (within f=0.25-25 Hz). Then, the Housner intensity (HI), Arias intensity (Ia), Peak Ground Velocity (PGV), PGA has been computed. Fig. 3 has compared the distribution of mentioned ground motion

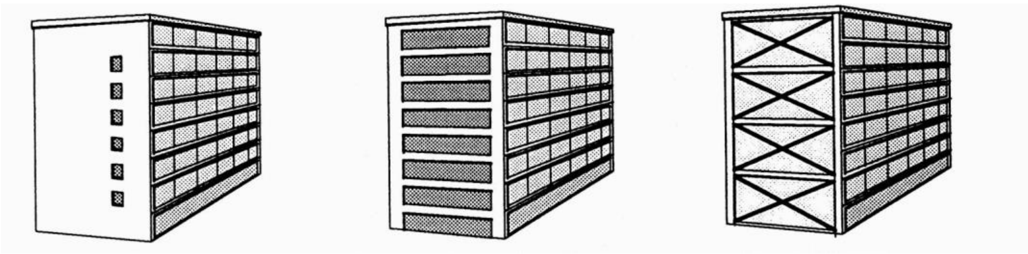

Fig. 4 Three-dimensional view of structural systems used in this study a) frame with shear walls (frame no:1), (b) moment-resisting frame (frame no:2) and (c) frame with X-bracing (frame no:3)

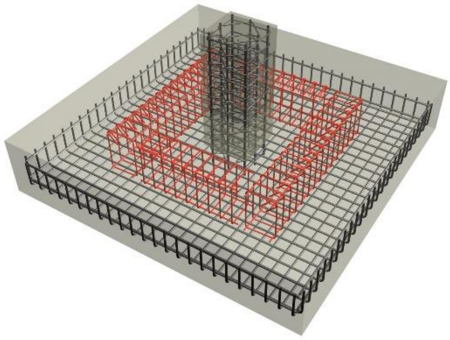

Fig. 5 A view of Approximate Boundaries for Finite-Element Models of Static Soil–Foundation Interaction (Ismail *et al*. 2018)

Table 1 The cross-sectional dimensions of structural elements

Station	Date	Components	Magnitude, Ms	Peak ground accel./(g)	Peak ground velocity (cm/s)	Peak ground disp. (cm)
Yarimca	17.080.1999	Ns	7.8	0.328	88.7	15.5

Table 2 Properties of earthquake

Storey number	Cross-section dimensions of the columns (cm)		Cross-section dimensions of the beams (cm)	Cross-section dimensions of the bracing components (cm)	Thickness of the shear walls (cm)
	Side Columns	Inner Columns			
1st storey	80x80	70x70	45x70	40x40	35
2nd storey	40x40	60x60	45x70	40x40	35
3rd storey	30x30	50x50	45x70	40x40	35
4th storey	60x60	60x60	45x70	40x40	35
5th storey	30x30	40x40	45x70	40x40	35

parameters for simulated and real record sets. The outcomes of real and simulated records as scattered data have demonstrated that even for a specified PGA, regional variability in the selection process is regarded. Also, regarding HI, Ia, and PGV, a close match is gained for the sets (Shao *et al.* 2015, Shao *et al.* 2018, Shao *et al.* 2019, Shi *et al.* 2019a, Shi *et al.* 2019b). Bearing structural systems used in this research are depicted in Fig 4. Based on Fig 4, one of the frames is moment-resisting frames (frame no: 1), another one is a frame with X-bracing (frame no: 2) and the last one is a frame with shear walls (frame no: 3). Also, all frames consist of four-bay and five stories as soil interaction. The height of the first story is 3.5 m and other stories are 3m. The span lengths of external bays are 4 m and internal bays are 3.5 m. The cross-section dimensions of each element are given in Table 1.

2.2 Construction details and elastic buckling of 3D frame structure

It should be noted that the cross-sectional dimensions of frame, beams, bracing components and thickness of shear walls considered frames are constant in all stories. In structural solutions as earthquake structures, the effects of structure

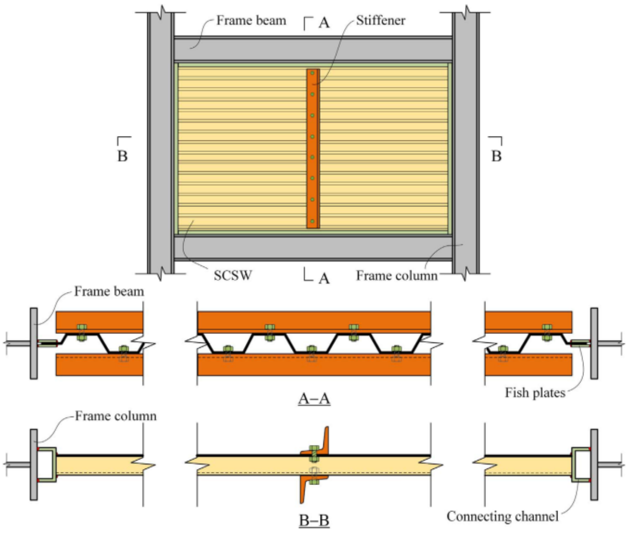

Fig. 6 Connection details of 3D frame structure

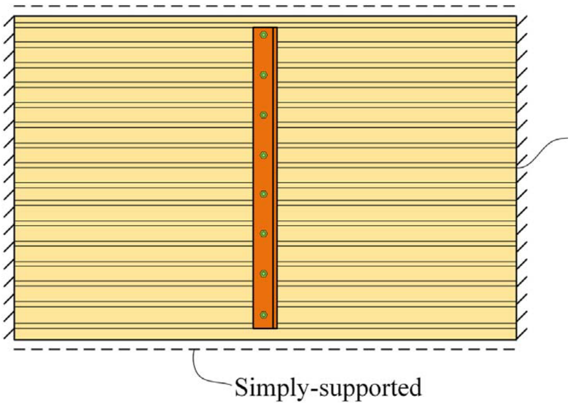

Fig. 7 Boundary conditions of 3D frame structure

behaviors on soil are discarded, even if the reality is not such. Because structure and soil generally behave differently during an earthquake. Indeed, soil and structure affect the behavior of each other because soft soils expand the amplitude of earthquake waves opposite to rock soils. In such soils, the more layer thickness is particularly increased, the more soil dominant period is increased (Khorami *et al.* 2017b, Katebi *et al.* 2019, Safa *et al.* 2019, Shariati *et al.* 2019e, Suhatril *et al.* 2019, Trung *et al.* 2019a, Trung *et al.* 2019b, Xie *et al.* 2019, Armaghani *et al.* 2020, Naghipour *et al.* 2020, Safa *et al.* 2020, Shariati *et al.* 2020a, Shariati *et al.* 2020b, Shariati *et al.* 2020c, Shariati *et al.* 2020d, Shariati *et al.* 2020e, Shariati *et al.* 2020f). The finite element mesh of these frames type used in analysis is carried out with LUSAS by considering soil interaction related to the properties of foundation soil (Fig. 3). It is accepted that the depth of foundation soil is 1.5H fold of frames height and the length of foundation soil is three fold (3H) of frame height from each side of frame (Khorramian *et al.* 2015, Khorami *et al.* 2017d, Khorramian *et al.* 2017b, Li *et al.* 2019, Luo *et al.* 2019, Mansouri *et al.* 2019, Milovancevic *et al.* 2019). Also, the vertical boundaries only make motion in vertical direction and the frame is supported rigidly from the foundation base. A summary of properties of foundation soil and frames and also the location and properties of earthquake applied in example are given in Table 1-3. Finite element analysis (FEM) is carried out with LUSAS V13.7. Damping ratio in all analyses is taken as 5%.

The corrugated plate of 3D frame structure is jointed with surrounding frame beams and columns with connection transition components (Fig. 5): the corrugated plate is joined with the frame beams including fish plates (A-A sectional view in Fig. 6), while it is joined with the frame columns including connecting steel channels (BB sectional view in Fig. 6). Connecting transition components comprising steel channels and fish plates are welded onto the edges of 3D frame structure while manufactured and across the construction procedure on site. Later, these components are welded with surrounding frame members (Mohammadhassani *et al.* 2013a, Mohammadhassani *et al.* 2013b, Mohammadhassani *et al.* 2014a, Mohammadhassani *et al.* 2014c, Nasrollahi *et al.* 2018, Nosrati *et al.* 2018, Paknahad *et al.* 2018, Naghipour *et al.* 2020). By adopting the connecting transition components, the out-of-plane rigidity of 3D frame structure would be essentially developed since these components behave like edge stiffeners, mightily to control the occurrence of large out of

Table 3 Properties of frames and foundation soil

Material	**E (kN/m²)**	**υ**	**γ (kN/m³)**
Frames (concrete)	$4.95x10^7$	0.8	25
Foundation soil (Loose Sand)	$30x10^3$	0.55	20

plane deflections of 3D frame structure across its erection and transportation (Safa *et al.* 2016b, Sadeghipour Chahnasir *et al.* 2018, Safa *et al.* 2019, Sajedi *et al.* 2019, Safa *et al.* 2020). Since bending moment could be transmitted via connecting steel channels, it could not be transmitted via fish plates, so the boundary conditions of 3D frame structure could be taken to be 2-side simply supported and 2-side fixed (Fig. 7).

2.3 Damage index

Considering structural engineering, structural damage has a physical interpretation while losing the resistance capability of external forces and becoming unstable. Damage control is complex in a structure due to few response parameters able to be instrumental in determining the damage stage occurred to a damage indices have been suggested for years with the aim of quantifying the structural damage in structures subjected to seismic excitations (Shah *et al.* 2015, Shah *et al.* 2016a, Shah *et al.* 2016b, Shah *et al.* 2016c, Shahabi *et al.* 2016a, Shahabi *et al.* 2016b, Nosrati *et al.* 2018, Katebi *et al.* 2019). One of these indices in previous studies came from (Park *et al.* 1985) over the consistent with dynamic behavior. They have explicated seismic structural damage as a linear combination of damage made by more deformations, when damage is caused by repeated cyclic loading effect. Park–Ang damage index is applied in different forms over last 30 years based on specific requirements. Important modifications of this index came by Shao *et* al. (Shao and Vesel 2015, Shao *et al.* 2019) as Eq. (1).

$$DI = \frac{dm-dy}{du-dy} + \frac{B}{vydu}\int dE \tag{1}$$

d_m = maximum deformation (demand) in dynamic loading
d_u = final deformation (capacity) in monotonic static loading
d_y = yield displacement
dE_h = increasing hysteretic energy (demand)
V_y = yield strength
β = a positive constant weighting the effect of cyclic loading on structural damage

In this equation, if the ultimate strength υ_u is smaller than υy , υy is replaced by υ_u. When the dynamic responses of a large structure population adjusted to ground shakings are of concern, applying idealized and simplified structural methods is more favorable than the complicated 3D frame structure models. Therefore, each building sub-class is idealized into an Equivalent Single Degree of Freedom system in this research. For each model, a well-known hysteresis method offered by (Hosseinpour *et al.* 2018) and named as “Modified Ibarra-Medina-Krawinkler Deterioration Model with peak-oriented hysteretic response” has been identified (Fig. 8). Moreover, three basic structural parameters comprising ductility factor (μ), period (T) and strength ratio (η) are identified for all building sub-classes. Total 20 samples are simulated for each subclass by use of Latin Hypercube Sampling method. Structural parameters corresponded to the simulated buildings could be seen in (Heydari *et al.* 2018).

3. Results and discussion

Shear buckling curves have been compared with FE numerical outcomes (Fig. 10). The hollow symbols show the models whose rigidity rates are lower than the transition variable, however, the solid symbols show the models whose rigidity rates are more than the transition variable that provide Eqs. (2)-(4).

$$\varphi = \begin{cases} 1 - 0.148\lambda_n^2\lambda \\ \varphi\alpha - \sqrt{\alpha^1}\Sigma\lambda_n^2 \end{cases} \tag{2}$$

$$1 - 0.137\lambda_n^2 \tag{3}$$

$$1 - 0.73/\lambda_n^2 \tag{4}$$

Conservatively predicting the shear resistance of most important ones for design are understood better the effect of stiffening components and foundation soil interaction. 5%-damped inelastic response spectra of scaled Loma Prieta earthquake records and scales the surrogate record for ductility ratio of 0, 2, 4, and 6 (Fig. 5). The mean value of errors in estimating the inelastic response spectra of original earthquake record using the surrogate record is 0.0489 that is calculated by numerical measure. Although the effect of higher modes might be significant on the nonlinear time-history response of 3D frame structure, considerable overlaps between the obtained inelastic response spectra (Fig. 9) suggest that the surrogate record could reasonably predict the actual roof displacement response-history of 3D frame structure.

It is seen that in case of not considering foundation soil interaction, variations of bending moment, shearing force and axial forces obtained throughout earthquake, frame no: 2 give generally smaller values than the others. As seen from Fig. 4, in case of not considering foundation soil interaction, absolute maximum bending moment of frame no: 2 and 3 are 539 kN (in the 4.38 second) and 538 kNm (in the 8.86 second), respectively. From the bending moment point of view, frame no: 3 are better than the others. As seen from Fig. 5, absolute maximum shear forces of frame no: 2 and 3 are 503 kN (in the 3.8 second) and 484 kN (in the 9.08 second), respectively. Also, from the shearing forces point of view, frame no: 3 are better than

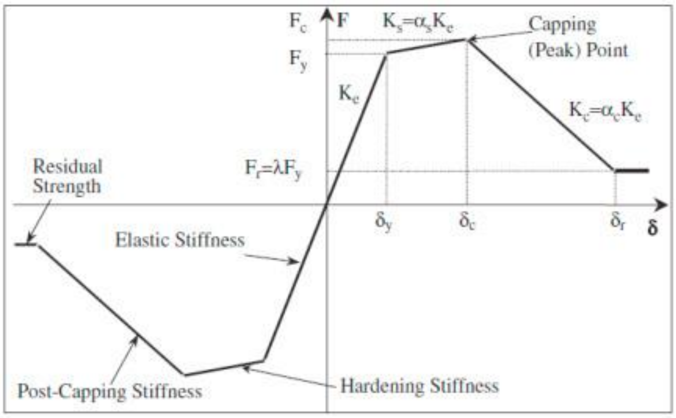

Fig. 8 Backbone curve for hysteresis method

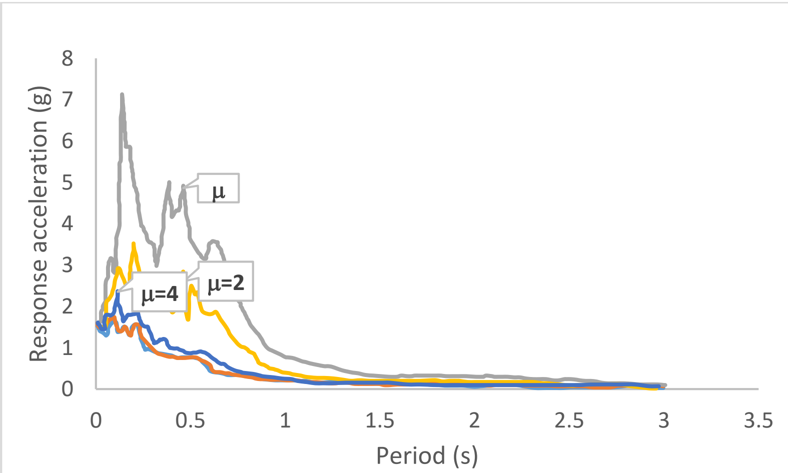

Fig. 9 5%-damped inelastic response spectra of scaled Loma Prieta earthquake record and scaled surrogate record

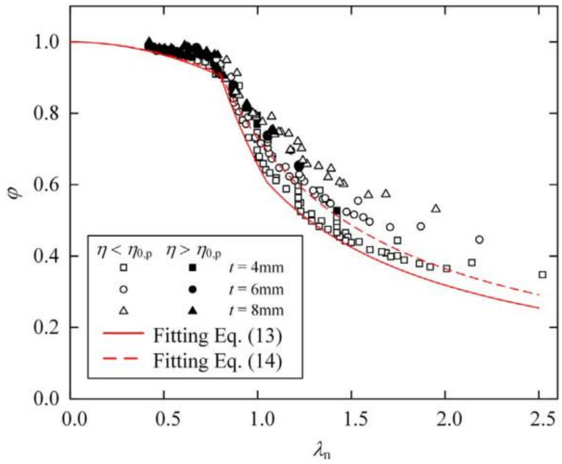

Fig. 10 The time histories of bending moments at 209 node point

the others. Based on Fig. 6, absolute maximum axial forces of frame no: 2 and 3 are 141 kN (in the 4.38 second) and 147 kN (in the 8.88 second), respectively. These show that, from the axial forces point of view, frame no: 2 are better than the others. Also, in Fig. 4 in case of considering foundation soil interaction, variations of bending moment obtained for three different frames are considerably decreased. In Fig. 11, in case of considering foundation soil interaction, variations of shearing force obtained for three different frames are considerably increased. In Fig. 12, in case of considering foundation soil interaction, variations of axial forces obtained for three different frames are

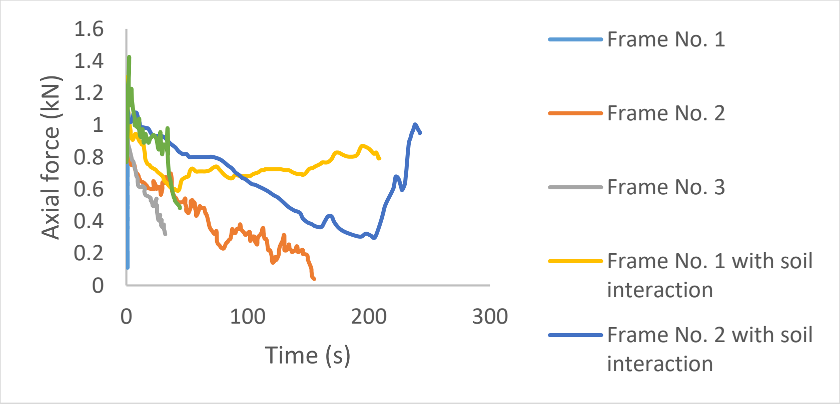

Fig. 11 The time histories of the shear forces at 209 node point

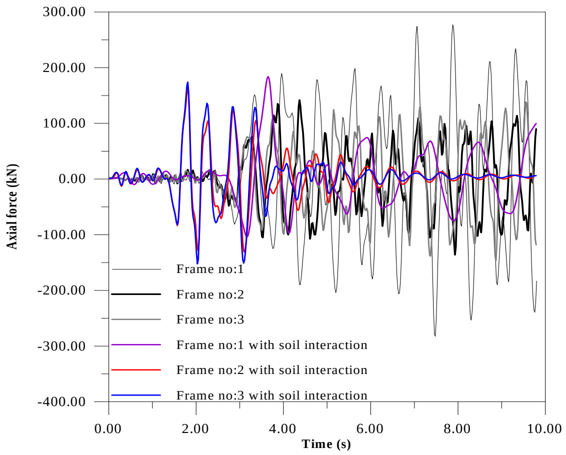

Fig. 12 The time histories of axial forces at 209 node point

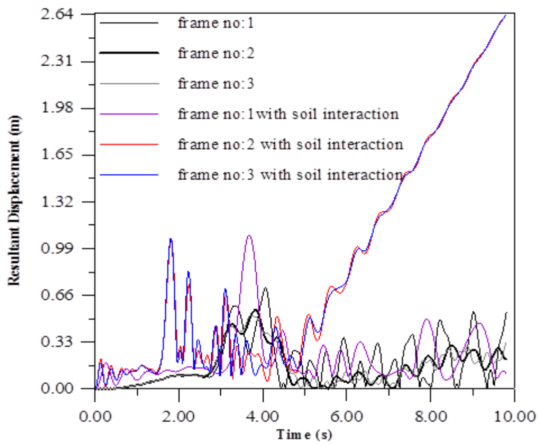

Fig. 13 Resultant displacements obtained node point 543 soil interactions for earthquake

considerably increased to certain times and later decreased according to the all considered frames type. This finding reveals the importance of soil interaction for building design (Armaghani *et al.* 2020, Naghipour *et al.* 2020, Shariati *et al.* 2020e, Shariati *et al.* 2020f). In the case of considering (or not considering) the foundation soil interaction, resultant displacements obtained throughout the earthquake process of 543 node point (Fig. 12) for the considered three types of structural systems are given in Fig. 13.

Based on Fig. 14, in case of not considering foundation soil interaction, the largest maximum displacement occurs in moment resisting frame. Its maximum value and variation during earthquake are significantly different from others. Regarding maximum displacement (moment resisting frame), any decrement in maximum displacements are 27%

and 37% for the frames stiffened with X-bracing and shear wall, respectively. Based on Fig. 13, maximum displacements are occurred around the time interval on which maximum ground acceleration is occurred. From Fig. 13, regarding the displacements of 543 node point in case of taking foundation soil interaction, all considered frames are greater than the one when interaction is not considered. Especially, variations of resultant displacement obtained for frame no: 2 and frame no: 3 are considerably increased. This finding is seen in the cases of considering soil interaction, then the vibration period of considered frames are increased (Shariati *et al.* 2020c, Shariati *et al.* 2020d, Shariati *et al.* 2020e, Shariati *et al.* 2020f). Comparing the results derived from the real and simulated records have shown a vast seismic responses' range for all masonry sub-classes because they are typically non-engineering structures with no standards in terms of construction technique and material quality (Li *et al.* 2019, Luo *et al.* 2019, Milovancevic *et al.* 2019, Safa *et al.* 2019, Sajedi and Shariati 2019, Shariati *et al.* 2019a). Simulated records offer rather higher exceedance possibilities than in real records for all subclasses and limit states except MU3C-LS3 case. A negligible gap is found between the curves for all subclasses when the curves are compared for LS1. Adding that this finding is highly pronounced for low-story buildings with higher quality. Whereas, while concerning the results for LS2 and LS3, the gap becomes visible particularly for more PGA levels (Shariati *et al.* 2019f, Shariati *et al.* 2019g, Suhatril *et al.* 2019, Trung *et al.* 2019a, Trung *et al.* 2019b, Xie *et al.* 2019, Safa *et al.* 2020, Shariati *et al.* 2020a, Shariati *et al.* 2020b, Shariati *et al.* 2020c, Shariati *et al.* 2020d). Maximum gap related to the exceedance possibility reaches up to 0.10, in which the simulated record-based FCs approximately show greater variables than in real records. This might be due to the potential nonlinear responses of structural methods at higher intensity (PGA) levels in which the responses are more complex because of yielding. Fig. 14 represents FA-based fragilities by use of simulated records for a sample case (MU2B) analysing the curves' sensitivity for ground motion variability by use of 200- versus 20-record sets. Comparing of Curves shows that the outcome derived from 200-record set almost lie into one standard deviation of mean curves based on 20- record sets. Variations among individual 20-record sets and 200-record set observed for all sub-classes have represents that ground motion variability has an essential influence on ultimate results. Therefore, considering ground motion variability regarding source to site distance, magnitude and soil condition for a specific intensity range in derivation of FCs seems reasonable. Comparing FCs according to 2 alternative assumptions for the possibility of exceedance could be essential. In case of comparing the sample subclass' curves (MU2B) from 2 alternative approach, it could be found that the exceedance possibilities computed based on ND-based model are more than in FA-based model (Fig. 4(b)). This observation could have validity for all sub-classes, in which the maximum gap in related to the exceedance possibility is around 0.35 (Shariati *et al.* 2019b, Shariati *et al.* 2019c, Shariati *et al.* 2019d, Shariati *et al.* 2019e, Armaghani *et al.* 2020).

4. Conclusions

The impact of different stiffness features in terms of foundation soil interaction on the seismic response of structures systems subjected to earthquake has been investigated by using finite element method. Thus, the main conclusions and recommendations are given as follow:

1. More methodology evaluation is needed. All outcomes of this research could be later verified by comparing the estimated damage rates from the offered FCs toward the observed damage data gained after earthquake in1992.
2. Accordingly, because of the ground motion records' variability and their dynamic features as ground motion amplitudes, frequency content, energy contents and final FCs have been affected by use of alternative ground motion set. The effect ratio is quantified herein.
3. From the analyses carried out for different structural systems, resultant displacements, bending moments, shearing forces and axial forces are significantly changed by vertical bearing system types and foundation soil interaction.
4. In case of not considering foundation soil interaction, frame with X bracing (frame no: 2) is reduced to the resultant displacements without making building heavy.
5. Resultant displacement values obtained from linear analysis is carried out regarding the foundation soil interaction with finite element method in which all frames are greater than the ones obtained from linear analysis in case of not considering foundation soil interaction by this method. This situation needs to take subsoil consideration in building design.
5. To generalize the results of different stiffness type while considering foundation soil interaction, solutions must be obtained using many earthquake inputs and foundation soil models. Results obtained from different inputs and models must be evaluated against one another. It is recommended that it may be useful for engineering community to consider nonlinear behaviors and different subsoil conditions. Also, the authors of this study have suggested that in terms of building design safety in earthquake regions, nonlinear analyses together by subsoil interaction should be performed.

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