Application of Endurance Time method in evaluation of seismic performance of a typical sandwich panel building

Mohammad Yekrangnia*

Abstract:

This study deals with the seismic performance evaluation of a typical five-story sandwich panel building with load-bearing wall as the only lateral force-resisting structural elements. For this purpose, incremental dynamic analyses (IDA) were performed on the model making use of seven selected ground motions. As a substitute to IDA, the response of the model in terms of maximum plastic strain, base shear and the inter-story drift ratio of the first story were compared with those from Endurance Time (ET) analysis. The results show that ET can simulate the seismic response of the studies model with an acceptable accuracy. However, the discrepancy between the ET results and those of the real ground motions increases ~~with~~ by increasing the excitation intensity. In general, ET underpredicts the drift ratios in very intense excitations, whereas the error of base shear with respect to the real ground motions remains almost constant regardless of the ground motion intensity.

1. Introduction

There are several studies on seismic performance of Reinforced Concrete (RC) shear walls [1]. However, very few studies have aimed at understanding the behavior of lateral load-bearing sandwich panels as the primary structural component. The main reasons for this negligence can be attributed to the fact that; 1) the behavior of lateral load-bearing sandwich panels is often assumed to be similar to RC shear walls; therefore, the design and evaluation of these structural elements follows those related to RC shear walls and 2) sandwich panels are not supposed to contribute in lateral load-bearing mechanism; in other words, they are designed as non-structural or structural components that contribute in transferring vertical loads only or they are designed against out-of-plane loadings that have originated from wind or blast loads. Benayoune et al. [2] performed experimental and numerical studies on axial capacity of composite sandwich panels. Based on the results of their studies, they proposed the limitation for slenderness ratio of these panels as 25.

* Corresponding Author: Assistant Professor, Department of Civil Engineering, Shahid Rajaee Teacher Training University, Tehran, Iran, E-mail: yekrangnia@sru.ac.ir

They proposed a semi-empirical relation to determine the capacity of sandwich panels under axial loading which was an enhanced from of the previously existing relations for solid RC walls. The ICC legacy report on 1997 Uniform Building Code [3] proposes several prescriptive limitations in the design of sandwich panels. Based on this revision, the axial and out-of-plane strength of sandwich panels are determined based on the proposed relation. However, for determination of in-plane capacity of these panels, the formulas used for determination of flexural capacity of RC walls based on ACI 318 [4] are proposed. The same procedure has been embraced by the Iranian code of practice for design specification, manufacturing and construction of 3D panel structures [5].

Mourtaja et al. [6] studied static cyclic behavior of a typical half-scale sandwich panel building by experimentations. They found the strength of the building can reach up to 10 times of the designed capacity according to the Turkish code. Walker and Smith [7] proposed a procedure for the best material combination and optimally designed sandwich panels. Their procedure was based on determination of the most economical material layer combination. The results of the optimization led to considerably cheaper sandwich panels compared to those available. Speaking of out-of-plane behavior of sandwich

panels, Benayoune et al. [8] investigated the effects of different parameters including steel shear connector's stiffness numerically and experimentally. They found out there is similarity between the out-of-plane behavior of sandwich panels and that in RC solid slabs since different layers in the former act as a composite section. Moreover, the ultimate strength and the degree of the desired composite action considerably depend on the steel shear connector's stiffness. Static cyclic testing was performed on seven sandwich panels by Voon and Ingham [9]. The results of their experimental study indicate that the ductility factor of sandwich panels based on their proposed details can reach up to 4.0. Moreover, they suggest that the strength capacity of these structural elements can be predicted by conventional flexural theory including a typical magnitude of strength reduction factor. Pavese and Bournas [10] performed several shaking table tests on full-scale sandwich panels and a two-story H-shaped specimen with the aim of evaluating the effects of the opening and roof boundary conditions. The results of their study proved strong flexural and shear coupling in the response of the specimens. However, due to high reinforcement of the considered walls, no shear failure associating with the sudden strength and stiffness degradation were observed. They claimed that the design procedure of sandwich panels can be regarded as the lightly reinforced shear wall based on Eurocode 8 [11]. Also, they proposed a relation for prediction of sandwich panels' deformation capacity. Rezaifar et al. [12] investigated seismic performance of a typical full-scale sandwich panel building by shaking table facility at Sharif University of Technology. Consequently, they studied the construction of each panel wall using calibration FEM. Their findings show the overall displacement ductility ratio of 4.5 and the over strength coefficient of approximately 6.0 of the studied specimens.

In this paper, the seismic performance of a typical sandwich panel building is evaluated by using nonlinear time-history analysis. The structural models are introduced first, followed by numerical simulation approach. Then, the load protocol involving a brief description of the endurance time method proceeded by the finite element modeling results is presented.

2. Structural models

In this study, seismic performance of a typical sandwich panel building is evaluated making use of nonlinear time-history analyses. The geometrical characteristics of the model building are shown in Figure 1. The 5-story building is supposed to be located on soil class C based on ASCE 7 [13] with very high site seismicity. The sandwich panels are the only load-bearing system in the considered building. The vertical loads on the two-way sandwich panel

slabs are assumed to be 370 kg/m2. The concrete thickness on load-bearing, non-load-bearing and slabs panels are 4, 5 and 5cm, respectively. All panels are reinforced with two-sided 50×50mm steel mesh consisting of 3mm bars.


a) Model section (East view)

b) Model plan view

c) Numerical model (North-West view)
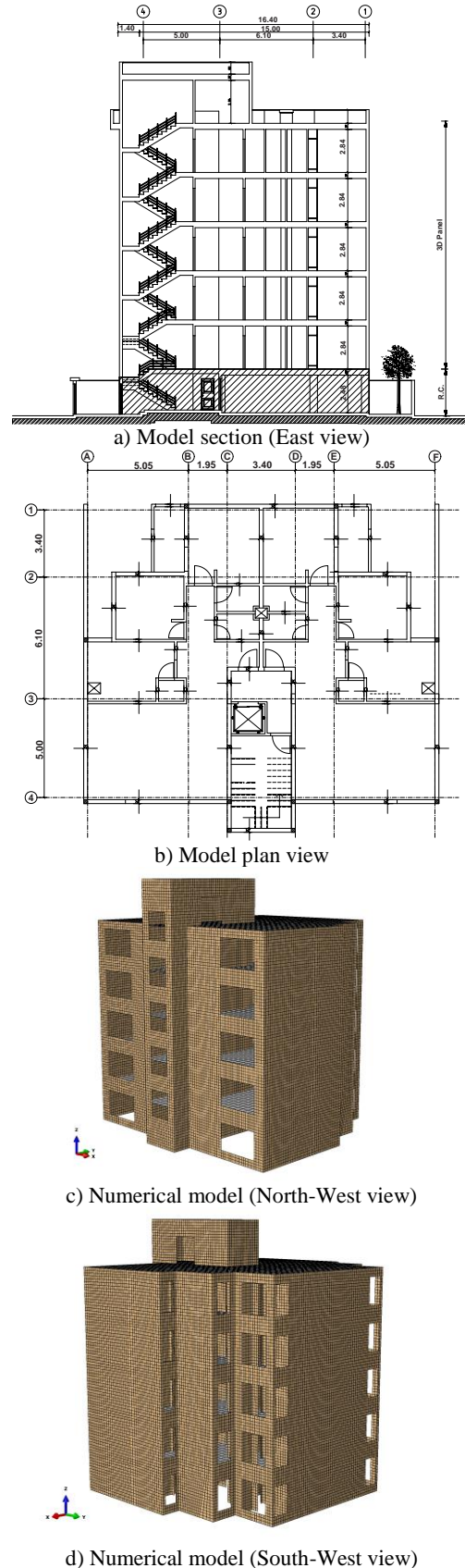
d) Numerical model (South-West view)
**Fig. 1:** Model geometrical characteristics (dimensions in m)

3. Numerical simulation approach

In this study, all analyses were performed making use of dynamic explicit numerical procedure in the commercial software ABAQUS [14]. The walls and roof of the structural models containing concrete, polystyrene and steel materials were modeled as composite shell sections with different thicknesses by 1st-order, and reduced integration three-dimensional shell elements (S4R). Mesh sensitivity analysis was performed and the mesh with element size of about 15cm was selected. The concrete damage plasticity material model (seeFigure 2a) was assigned to the concrete part of the shell elements in order to capture cracking and crushing of concrete. Drucker-Prager's plastic flow with Lubliner et. al. [15] yield function, as well as the modifications proposed by Lee and Fenves [16] was utilized for the concrete. Kent and Park [17] model was used for stress-strain behavior of concrete in the compression regime. Moreover, compressive and tensile damage–was considered for the concrete according to the Eqs. (1) and (2). This damage influences the slope of the unloading branch of the stress-strain curve of concrete. It is noteworthy that the tensile damage of concrete has no effect on the compressive behavior while compressive damage of concrete has full influence on concrete in the tensile phase. For reinforcing steel, Mises yield function was assumed with bilinear stress-strain behavior possessing strain hardening based on Figure 2b. The mechanical properties of materials are presented in Table 1. In this table, E and ν are modulus of elasticity and Poisson's ratio, respectively; f_c and f_t are the compressive and tensile strength of concrete; G_f^I is the fracture energy in Phase I for unit surface area; f_u and ε_u are the ultimate strength and ultimate strain of steel, respectively. It is noted that the utilized modeling procedure for sandwich panels have already been verified in similar studies [18].

$$D_c = 1 - \frac{\sigma_c}{f_c'} \quad \text{and} \quad D_t = 1 - \frac{\sigma_t}{f_t} \tag{1,2}$$
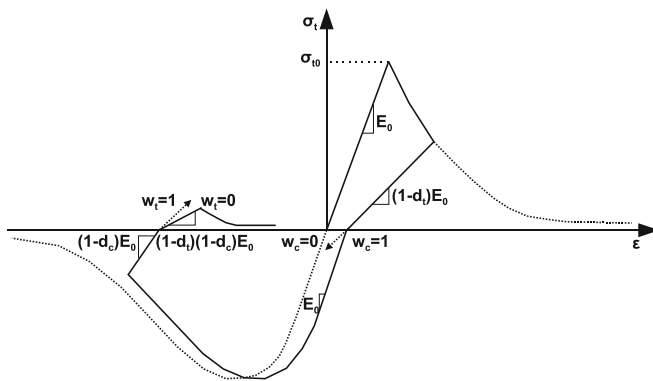
a) Concrete damage material model [13]
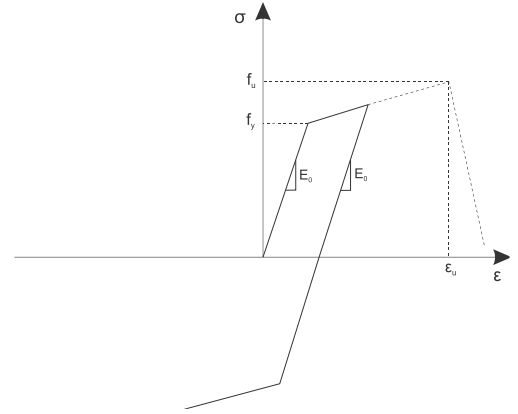
b) Steel yielding material model

Fig. 2: Material model under uniaxial tension and compression

**Table 1:** Mechanical properties of materials

|	Elastic		Compressive			Tensile		
	E (GPa)	ν	f_c (MPa)	f_u (MPa)	ε_u	f_t (MPa)	G_f^I (N/mm)	f_u (MPa)
Concrete	27	0.2	28	--	--	2.8	1.1	--
Polystyrene	0.003	0.0	--	--	--	--	--	--
Steel	210	0.3	415	530	0.08	415	--	530

4. Load protocol

For determination of Engineering Demand Parameter (EDP) of each building against any given seismic event with a specific Intensity Measure (IM), it is necessary to perform Incremental Dynamic Analysis (IDA). This analysis can be very time consuming for some structures including the model in this study, which makes it practically unpopular. For solving this issue, an innovative dynamic pushover procedure called Endurance Time (ET) proposed by Estekanchi et al. [19] with major recent achievements is utilized [20,21]. A comprehensive review of the ET was made by Estekanchi et al. [22].

These intensifying acceleration functions have been produced making use of numerical and optimization techniques. The main advantage of ET records is that their response spectrum at any time is proportional to the response spectrum at a target time ($t_{target} = 10sec$) which is summarized in Eqs. (3) and (4) and presented in Fig. 3. As such, the time in these records is a critical factor determining the intensity of the excitation. The more a typical structure withstands ET records, the more favorable is its seismic performance. This is schematically shown

in Fig. 4 which indicates that Design A cannot meet the code requirements because the building collapses before reaching the target time. Also, as can be seen in Fig. 5, up to a specific time, the maximum value of each EDP is critical, and time-history of the maximum of absolute results of ET records are plotted besides the maximum EDP from real earthquakes.
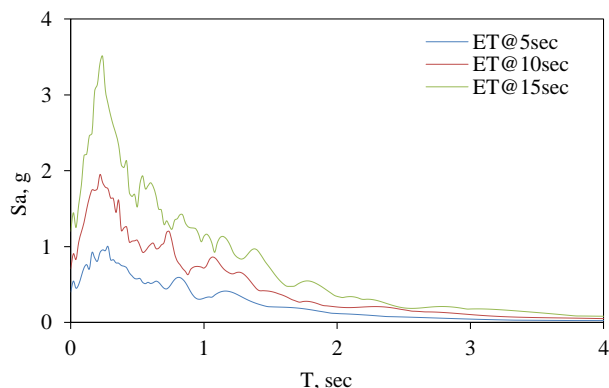
Fig. 3: Proportional intensification of spectrum intensity and ET time
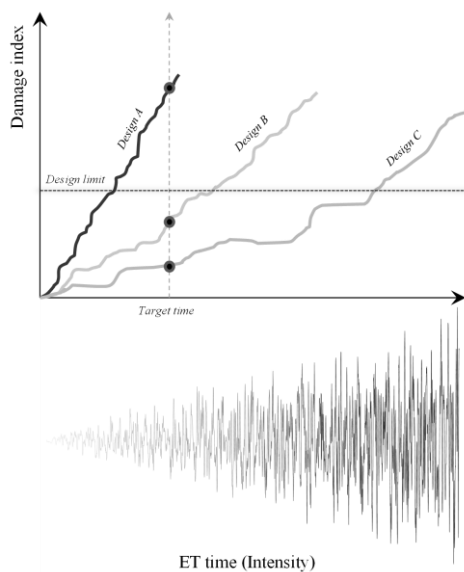
Fig. 4: Schematic representation of ET records and EDP in different design levels
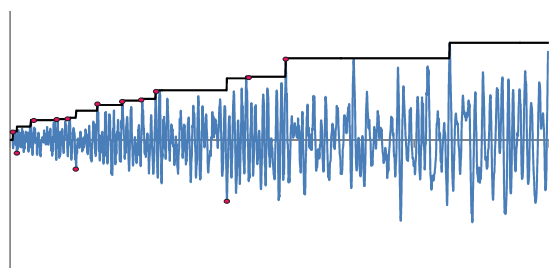
Fig. 5: Method of plotting increasing curves of ET results

$$S_{aR}(T, t) = \frac{t}{t_{Target}} S_{aR}(T) \qquad (3)$$

$$S_{uR}(T, t) = \frac{t}{t_{Target}} S_{aR}(T) \times \frac{T^2}{4\pi^2} \qquad (4)$$

where T is the natural period of the structure and S_{aR} (T) is the template spectrum, S_{aR} (T, t) is the target spectrum to be approached at time t of ET function and S_{uR} (T, t) is the target displacement spectrum to be induced at time t by ET function. In order to evaluate accuracy of ET records in capturing EDP of the real earthquake records, seven seismic events among those recommended by FEMA440 [23] listed in Table 2, were selected. It is noteworthy that these records were selected based on the code requirements of ASCE 7 [13] and Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard 2800) [24]. In this study, the excitation records were scaled based on Standard 2800 procedure which is similar to ASCE 7 with the exception of applying different scale factors to record components in order to assign equal intensity to them. Another difference is the factor of 1.4 compared to 1.3 of ASCE7-05, which originated from $\sqrt{1^2 + 1^2}$, indicating the concept of equal intensity of both components, whereas, the acceptable relative intensity of 0.83 has been considered by ASCE7-05 from 1.3 amplification factor. Similar to ASCE7-05, Standard 2800 states that acceleration spectrum of average of SRSS of the selected records should not be lower than codified spectrum in $0.2T_1 \sim 1.5T_1$ where T_1 is the period of the first vibration mode equal to 0.125 sec in this study. As previously stated, this period increases in the first seconds of even moderate excitations and based on the results, reaches to about 1.0 sec. As a result, it is more conservative to take T_1 as 1.0 sec. This assumption can be justified by Bommer's findings [25] which suggest scaling records with the upper bound of $3.0T_1$ for reducing scaling-dependent scattering of the results. According to Fig. 6, Standard 2800 spectrum gives higher values in long periods and this assumption yields high scale factors. Therefore, in the aforementioned range of period, the records spectrum was scaled in order to have equal area with the codified spectrum instead of applying the code method. The scaling of ground motions was based on the spectral acceleration of Standard 2800 corresponding to different endurance times of 5, 10, and 15 sec. It is worth mentioning that PEER Database selects and scales records based on mean square error from a given base spectrum [26]; however, combining two components using Geometric Mean (GM) method by this database may result in high scale factors. Imagine a record with 100% and 50% matching of the components to ASCE7-05 spectrum. Based on this code, the scale factor equal to 1.16 is produced as a result of using SRSS method compared to 1.3 times the base

spectrum. On the other hand, using GM method and scaling these components to the base spectrum, results in a scale factor of 1.41 that is considerably larger than the former one. Comparison of spectral acceleration of ET, average of 7 records and codified spectra are shown in Fig. 6. In this study, the excitations were applied to the model in three different directions simultaneously. The selected Endurance Time Excitation Function (ETEF) was ETA20in_xyz.

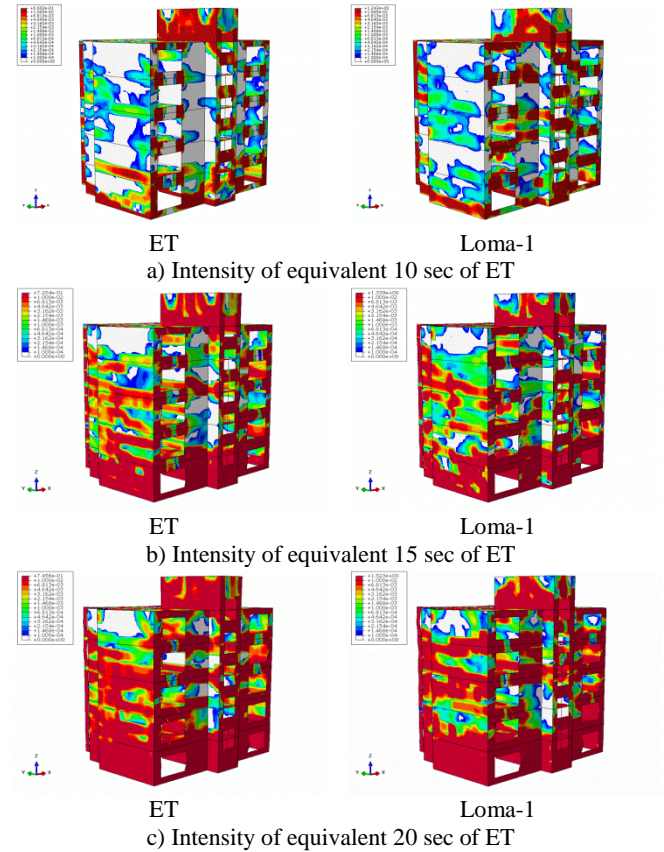**Table 2:** Selected earthquake records for comparative study of ET

Earthquake name	Scale factor	Magnitude (M_s)	Station Number	Referred as
Landers	1.80	7.5	12149	Landers
Loma Prieta	1.28	7.1	58065	Loma-1
Loma Prieta	0.95	7.1	47006	Loma-2
Loma Prieta	0.75	7.1	58135	Loma-3
Loma Prieta	0.92	7.1	1652	Loma-4
Morgan Hill	1.39	6.1	57383	Morgan
Northridge	0.54	6.8	24278	North
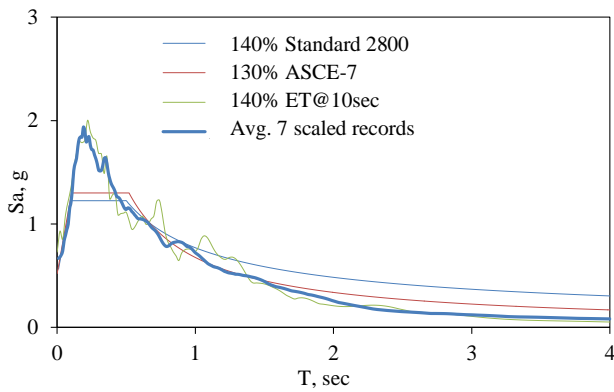
Fig. 6: Comparison of spectral acceleration of ET, average of 7 records and codified spectra (The first mode period of vibration of the model is 0.37 s).

5. Finite element modeling results

The results of the ET analyses and an example of one of the real ground motion results in terms of maximum principal plastic strain in different endurance times are shown in Fig. 7. As can be seen, the results of ET and the Loma-1 in terms of damage severity and propagation are in close match. Also, the tensile plasticity which is approximated by concrete cracking becomes more intense and propagates by increasing the endurance time. The first story and the ridge are the most vulnerable parts of the building. Comparison of the contours in different endurance times and the time history (TH) analysis with the corresponding intensities indicates that acceptable agreement between the two sets of results exists. The results of the second set were not reported here for the sake of brevity.

ET   Loma-1
a) Intensity of equivalent 10 sec of ET

ET   Loma-1
b) Intensity of equivalent 15 sec of ET

ET   Loma-1
c) Intensity of equivalent 20 sec of ET
Fig. 7: Contours of maximum principal plastic strain

Comparison of the results of ET analyses and TH analyses for the base shear of the model building and the inter-story drift ratio of the first story versus endurance time in two horizontal directions of the building is made in Fig. 8. An acceptable match between the results of ET and TH is observed that proves the ability of ET to predict the response of the model in different excitation intensities with acceptable accuracy. Generally, the accuracy of the ET in predicting the base shear is more acceptable compared to inter-story drift ratio. This can be attributed to the fact that by increasing the excitation intensity, the model building reaches the maximum force capacity; however, the inter-story drift ratio that follows the increasing trend cannot be precisely captured by ET in excitations with large intensity. This justifies the significant discrepancies between the two sets of results of ET and TH in higher endurance times. It can be noted from Fig. 8 that for longitudinal direction, the base shear in ET analysis is higher than that of the average ground motions, while inter-story drift that was predicted by ET is less than that of the other set. This can be explained by the contribution of some ground motion parameters such as strong duration and the number of cycles especially in higher intensities that affect the maximum inter-story drifts, although they marginally influence the maximum base shear of the model. This can lead to gradually increasing the discrepancy of the results of ET and the corresponding ground motions. Incidentally, the considerable differences

49

between the ET and TH are associated with ground motion excitations levels with very large return period (2475 years and 8000 years for endurance time of 15 s and 20 s, respectively), and hence, ET can acceptably predict the response of the model building as a substitute for design level earthquakes that involve a majority of case studies in practice. As a suggestion for the acceptance criteria for sandwich panels, those proposed by ASCE 41 for RC shear walls (RC1 in this code) can be applied here [27]. The acceptable drift ratio for Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) based on this code is 0.4, 0.6, 0.75, respectively. Based on this assumption, the results show that the model building behaves acceptably at the target time of endurance time.

a) Base shear
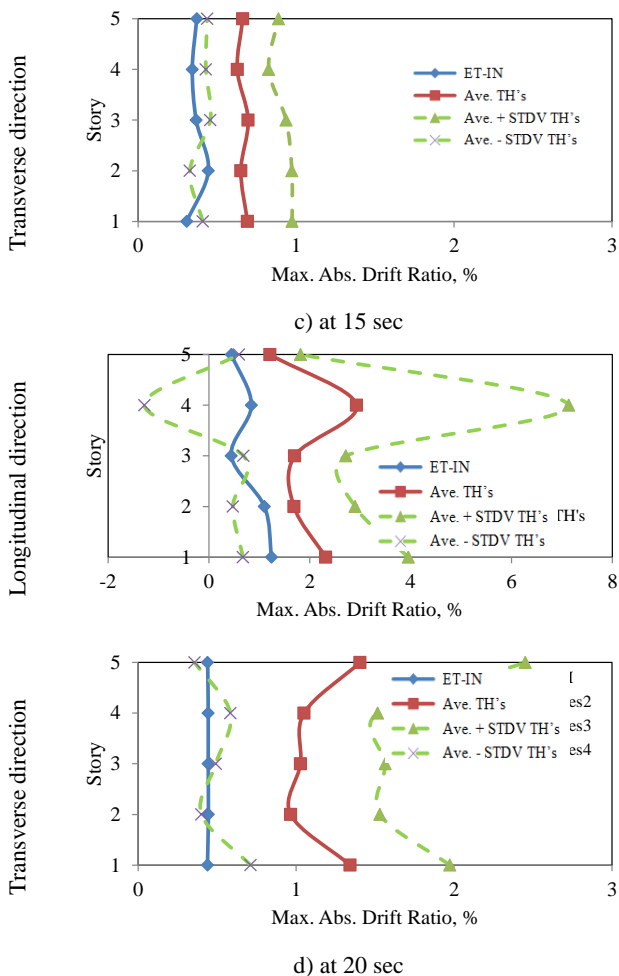

b) Inter-story drift ratio of the first story
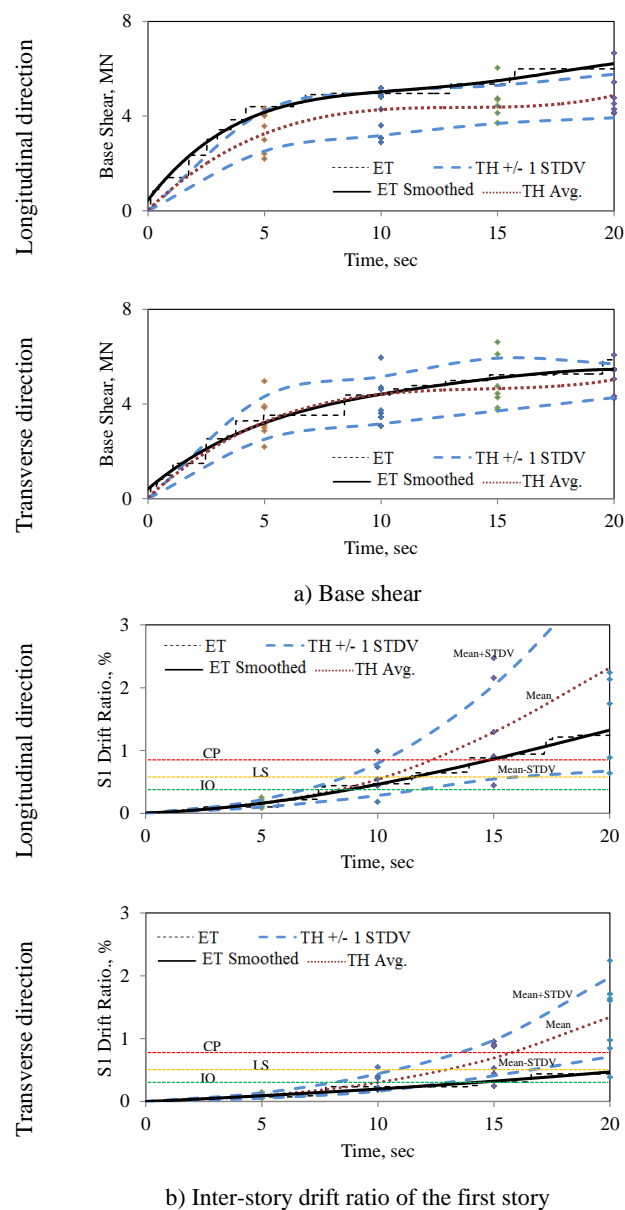
Fig. 8: Comparison of the base shear and inter-story drift ratio of the first story of ET and time history analyses

Fig. 9 shows the maximum absolute inter-story drift ratio of the first story of ET and time history analyses. As can be seen in this figure, the results of ET and TH are in close

match in lower excitation intensities. In all cases, the predicted inter-story drift ratio by ET is less than that of the TH. This difference increases with increasing the excitation level and can be attributed to the nonlinear behavior of the model under high intensity excitation that is influenced by several parameters including excitation duration, number of cycles, etc. More information about this dependency can be found elsewhere [28-31].

a) at 5 sec

b) at 10 sec

c) at 15 sec

d) at 20 sec
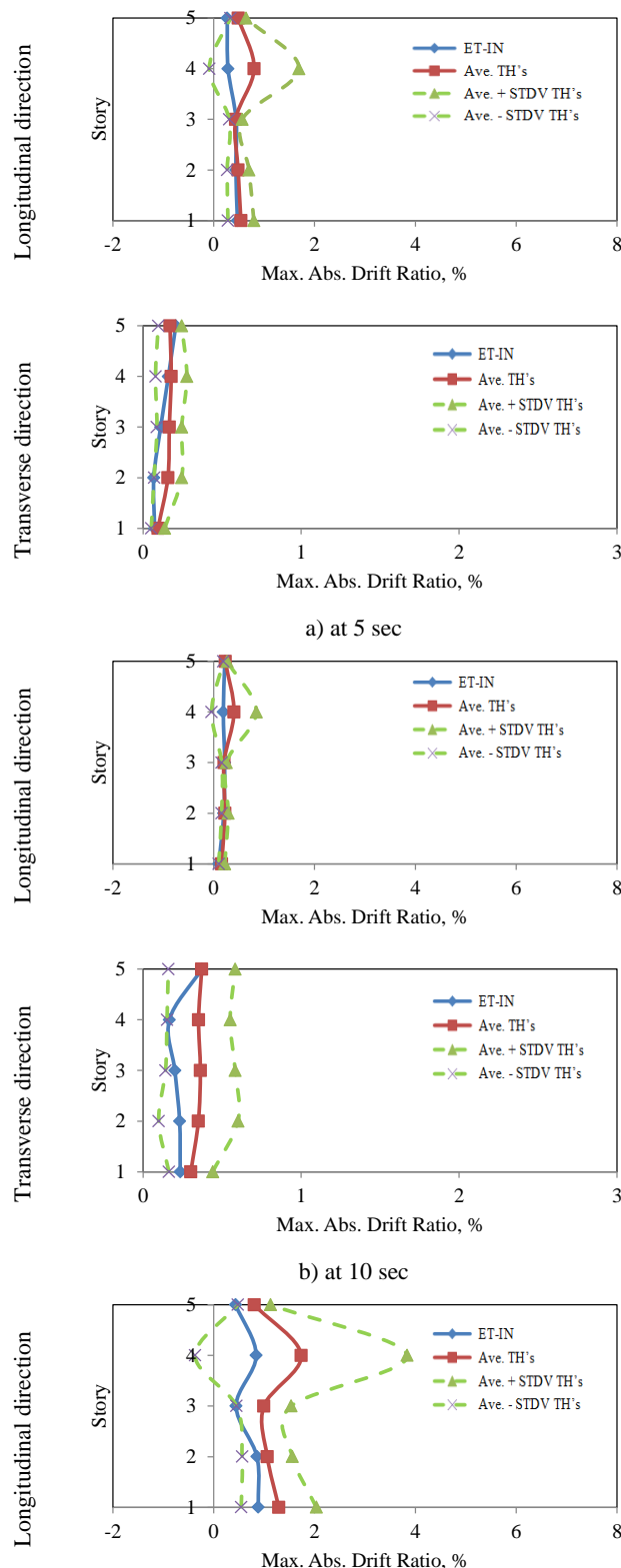
Fig. 9: Comparison of the maximum absolute inter-story drift ratio of the first story of ET and time history analyses

6. Conclusions

In this paper, the seismic performance of a typical sandwich panel building was evaluated using nonlinear time-history analysis. For comparison purposes, both endurance time analyses and time-history analyses consisting of seven selected ground motion excitations with different intensities were applied to the model building and the results in terms of maximum principal plastic strains, base shear and inter-story drift ratios were derived. The results indicate that in low to moderate levels of excitation, the endurance time can accurately predict the response of the model building compared to that from the real earthquake records. However, the difference between the endurance time analyses and the ones related to real ground motions increases in more intense excitation levels. The differences in inter-story drift ratios were larger than that in the base shear. The endurance time can, in some cases, underpredict the inter-story drift ratios in large intensities because some parameters such as duration and number of ground motions play a more important role in the determination of structural response. It is important to note that the observed underestimation from ET analysis may

depend on the properties of the ET excitations used and the method used in matching ET analysis time with equivalent GM intensity. This difference may also be case sensitive and hence, cannot be generalized.

References:

[1] Estekanchi H., Harati M., Mashayekhi M. (2018). An Investigation on the interaction of RC shear walls and moment resisting frames in RC dual system using endurance time (ET) method, The Structural Design of Tall and Special Buildings, e1489, 27(12), 1-16.

[2] Benayoune A., Abdul Samad A., Trikha D.N., Abang Ali A.A., Ellinna S.H.M. (2008). Flexural behaviour of pre-cast concrete sandwich composite panel– Experimental and theoretical investigations, Construction and Building Materials, 22, 580–592.

[3] ICC evaluation service, Legacy report ER-5618, Tridipanel 3D/EVG panels, March 2006.

[4] ACI Committee. (2002). Building code requirements for structural concrete:(ACI 318-02) and commentary (ACI 318R-02). American Concrete Institute.

[5] The Islamic Republic of Iran Vice Presidency for Strategic Planning and Supervision, The code of practice for design specification, manufacturing and construction of 3D panel structures, (First Revision), No. 385, Office of Deputy for Strategic Supervision Department of Technical Affairs, 2013.

[6] Mourtaja W., Karadog F., Yuksel E., Alper I. and Balci A.A. (2000). 3D behavior of shotcreted lightweight panel buildings, 12 WCEE, Auckland, New Zealand.

[7] Walker M. and Smith R. (2002). A computational methodology to select the best material combinations and optimally design composite sandwich panels for minimum cost, Computers and Structures 80, 1457–1460.

[8] Benayoune A., Samad A.A.A., Abang Ali A.A., Trikha D.N. (2007). Response of pre-cast reinforced composite sandwich panels to axial loading, Construction and Building Materials, 21, 677–685.

[9] Voon K.C. and Ingham J. (2000). In-plane testing of 3-D wall panels, Uniservices No. 7654.02, The University of Auckland.

[10] Pavese A. and Bournas D.A., Experimental assessment of the seismic performance of a prefabricated concrete structural wall system, Engineering Structures 33 (2011), 2049–2062.

[11] Eurocode 8: Design of structures for earthquake resistance-part 1: general rules, seismic actions and rules for buildings. Brussels: European Committee for Standardization (2005).

[12] Rezaifar O, Kabir M.Z., Taribakhsh M. and Tehranian A. (2008). Dynamic behaviour of 3D-panel single-storey system using shaking table testing, Engineering Structures 30, 318–337.

[13] ASCE. (2005). Minimum design loads for buildings and other structures. ASCE/SEI 7-05 including Supplement No.1, Reston, VA.

[14] Abaqus, User's manual version 6.9, Hibbitt, Karlsson and Sorensen Inc., Pawtucket (RI, USA), 2005.

[15] Lubliner J., Oliver J., Oller S., and Oˇnate E. (1989). A plastic-damage model for concrete, International Journal of Solids and Structures, 25(3), 299-329.

[16] Lee L., and Fenves G. (1998). Plastic-damage model for cyclic loading of concrete structures, Journal of Engineering Mechanics, 10.1061/(ASCE)0733-9399(1998)124:8(892), 892-900.

[17] Kent D.C., and Park R. (1971). Flexural members with confined concrete, Journal of Structural Division, 97(7), 1969-1990.

[18] Kabir, M. Z., Shadan, P., & Kabir, H. (2018). A numerical and experimental study on the dynamical behavior of 3D-Panel Wall on Piloti RC Frame. International Journal of Structural Integrity.

[19] Estekanchi H.E., Vafai A. and Sadeghazar M. (2004). Endurance Time method for seismic analysis and design of structures, Scientia Iranica, 11(4), 361-370.

[20] Mashayekhi M., Estekanchi H., Vafai A., Ahmadi G. (2019). An evolutionary optimization-based approach for simulation of endurance time load functions, Engineering Optimization, 52(12), 2069-2088.

[21] Mashayekhi M., Harati M., Estekanchi H. (2019). Development of an alternative PSO-based algorithm for simulation of endurance time excitation functions, Engineering Reports, (3), 1-15.

[22] Estekanchi H., Mashayekhi M., Vafai A., Ahmadi G., Mirfarhadi S.A., Harati M. (2020). A state-of-knowledge review on the Endurance Time Method, Journal of Structures, 27, 2288-2299.

[23] Applied Technology Council (ATC) (2005). "Improvement of nonlinear static seismic analysis procedures." Rep. No. FEMA-440, Washington, D.C.

[24] Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard 2800) (2013), Fourth Edition. Building and Housing Research Center, Iran (in Persian).

[25] Bommer J.J., Magenes G., Hancock J., Penazzo P. (2004). The influence of strong-motion duration on the seismic response of masonry structures. Bulletin of Earthquake Engineering, 2(1), 1-26.

[26] Ancheta T.D., Darragh R.B., Stewart J.P., Seyhan E., Silva W.J., Chiou B.S., ... and Donahue J.L. (2013). PEER NGA-West2 database.

[27] ASCE 41-06 (2006). Seismic Rehabilitation of Existing Buildings, American Society of Civil Engineers, Reston, Virginia.

[28] Harati M., Mashayekhi M., Ashouri M., Estekanchi H. (2019). Influence of ground motion duration on the structural response at multiple seismic intensity levels, Numerical Methods in Civil Engineering, 3(4), 10-23.

[29] Harati M, Mashayekhi M, Estekanchi H. (2019), "Estimating the duration effects in structural responses by a new energy-cycle based parameter", 8th International Conference on Seismology and Earthquake Engineering (SEE8), IIEES, Nov 11-13, Tehran.

[30] Mashayekhi M, Harati M, Ashouri M, Estekanchi H. (2019), "Introducing a response-based duration metric and its correlation with structural damages", Bulletin of Earthquake Engineering, 17, 5987–6008.

[31] Mashayekhi M, Harati M, Darzi A, Estekanchi H. (2020), "Incorporation of strong motion duration in incremental-based seismic assessments", Journal of Engineering Structures, 223.