Application of Endurance Time method in Nonlinear Seismic Analysis of Infilled Steel Frames

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SUMMARY:
Nonlinear analysis of infilled frames using conventional analysis methods, such as incremental dynamic analysis (IDA), requires high levels of modeling complexity and computational effort; therefore the infill panel is often neglected. Introducing more simple and straightforward methods of analysis, can be a solution to this problem. The Endurance Time (ET) method is an innovative method for dynamic loading and analysis of structures. In this paper, application of the ET procedure in nonlinear dynamic analysis of infilled steel frames is investigated. For this purpose, 2D moment resisting steel frame models with unreinforced masonry clay brick walls as infills, infilled in three different cases, is considered. Results of ET and IDA nonlinear analyses for different infill cases of the models are discussed and compared to observe the behavior of difference infill cases and moreover, the ET method's potential advantages and drawbacks in seismic evaluation of this category of frames is investigated.

Keywords: Infilled steel frame, Incremental Dynamic Analysis, Endurance Time method

1. INTRODUCTION

In many countries situated in seismic regions, reinforced concrete and steel frames are infilled fully or partially by brick masonry panels with or without openings, to serve as interior and exterior walls. Traditionally, such infill walls are specified by architects in such a way that they do not contribute to the vertical gravity load-bearing capacity of the structure the resulting system is referred to as an infilled frame, which has high in-plane stiffness and strength. Although the infill walls are mostly considered as non-structural elements, it is known that they have important impact on the behavior of the frames and such neglect may lead to a substantial inaccuracy in predicting the lateral stiffness, strength, ductility, and energy dissipation capacity of the frame.

Since 1940, extensive studies have been conducted on lateral load behavior of masonry-infilled frames, both experimentally and analytically. As one of the first and pioneering researchers on infilled frames, Polyakov (Polyakov, 1956) conducted comprehensive experiments on large scale steel infilled frames and some of midrise buildings in Moscow in late 1940's. They concluded that the presence of infills increases the stiffness of these 14-story buildings by a factor of 10–20. Stafford Smith (Stafford Smith, Lateral Stifness of Infilled Frames, 1962) conducted a series of tests on laterally loaded square mild steel frame models infilled with micro concrete. Monitoring the model deformations during the tests showed that the frame separated from the infill over three quarters of the length of each member. These observations led to the conclusion that, the wall could be replaced by an equivalent diagonal strut connecting the loaded corners. Flanagan and Bennett (Flanagan & Bennett, 1999) based on experimental results of 21 steel frames with clay tile infill walls, used a piecewise-linear equivalent strut to model infill and proposed an analytical procedure to calculate the strength of the infill. As one of the latest analytical researches, Mohebkhah et al. (Mohebkhah, Tasnimi, & Moghadam, 2008) employed a discrete element method (DEM) to simulate the nonlinear behavior of masonry-infilled steel frames. In the discrete element method, large displacements and rotations between blocks, including sliding between blocks, the opening of the cracks and even the complete detachment of the blocks, and automatic detection of new contacts as the calculations' process were allowed.

Despite of the rapid improvements in computer technology in recent years, because of the high complexity and demanding scientific background of infilled frames, practicing engineers often neglect the infill in the analyses of such frames. Such negligence may lead to significant error in predicting the behavior of these frames and consequently an unreliable or noneconomic design. Introducing more simple and straightforward methods of analysis and design for these frames, can be a solution to this problem. The Endurance Time (ET) method is an innovative and straightforward method for dynamic loading and analysis of structures, apprehensible for standard level of seismic engineering knowledge. The basic idea of ET method was originally introduced by Estekanchi et al (Estekanchi, Vafai, & Sadeghazar, 2004). The concept of ET method is similar to the exercise test used by cardiologists for assessing the condition of cardiovascular system of patients. In this novel procedure, an intensifying artificial accelerogram, termed as Endurance Time Acceleration function or ETA, is applied to the structure and its various structural responses monitored. Since different times in ET acceleration functions corresponds to different seismic intensities, a single ET time history analysis provides structural response information at various intensities, thus significantly reducing time and computational cost compared to IDA.

2. INFILL ANALYTICAL MODELING

In order to study the behavior of actual masonry-infilled frames and represent their influence on the rest of the structure by a simplified model, macro-modeling strategy (versus micro- modeling) is usually addressed. In this method, the masonry infill wall is replaced by an equivalent system requiring less computational time and effort, thus suitable for analysis of multistory-multibay buildings. Various Macro models have been proposed by researchers [(Chrysostomou, Gergely, & Abel, 2002), (Kadysiewski & Mosalam, 2008(102))]. The simplest model in this category was proposed by Stafford-Smith (Stafford-Smith, 1996). According to this model, an equivalent pin-jointed diagonal strut is substituted for the infill panel. The equivalent width of the strut depends on the relative infill-frame stiffness.

In this study, the nonlinear behavior of infilled frames has been characterized by idealizing the infill as two diagonal equivalent compression struts ('Figure 2.1') that braces the frame, increasing its lateral stiffness and strength. This widely accepted approach is commonly known as the 'equivalent-strut approach'.

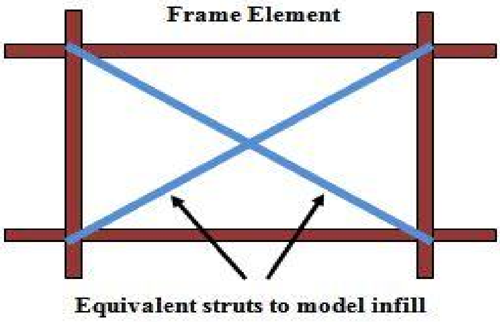


Figure 2.1. Equivalent compression strut idealization of infills

Procedures have been proposed for computing the dimensions and properties of this equivalent strut. In this regard, methods based on experimental research appear more suitable than those based on analytical research, probably because of the inherent material and geometric variability associated with infills. After a series of experimental investigations on small-scale moment frames with mortar infilling, Stafford Smith (Stafford Smith & Carter, A method of analysis for infilled frames, 1969) proposed that the width of the equivalent strut, w_{strut}, would depend on a relative stiffness parameter, λ (Eqns. 2.1 & 2.2):

$$w_{strut} = \frac{\pi}{C\ \lambda_{strut}\ cos\theta} \tag{2.1}$$

Where,

$$\lambda_{strut} = \sqrt[4]{\frac{E_m\ t_{infill} \sin 2\theta}{4\ E_{col}\ I_{col}\ h_{infill}}} \tag{2.2}$$

In these equations, h_{infill} and t_{infill} are the infill's height and thickness; E_{col} and I_{col} are the elastic modulus and moment of inertia of confining columns, and E_m, C and θ are the elastic modulus of masonry infill material, an empirical constant and the angle between the diagonal of the infill panel and the horizontal , respectively. In this approach, each equivalent strut element is assigned an appropriate hysteretic force-deformation relationship, 'backbone curve', generally including a descending post-peak strength, in-cycle degradation, and pinching. Conventional clay masonry is considered as the infill material in this study. For this infill type, the Ibarra-Krawinkler (Ibarra, Medina, & Krawinkler, 2005) hysteretic model with pinched hysteretic rules, incorporated in a backbone curve ('Figure 2.2'), shows better agreement with past experimental results by Flanagan and Bennett (Flanagan & Bennett, 1999) and thus is used for this purpose in this research.

The parameters of the backbone curve, i.e. strength at yield, F_y, maximum strength (at capping point), F_c, plastic deformation capacity, δ_p, the post-capping tangent stiffness, K_{pc}, and the residual strength, F_r, are determined based on published experimental data by Flanagan [(Flanagan & Bennett, 1999) ,(Flanagan & Bennett, 2001)], for conventional clay masonry infills.

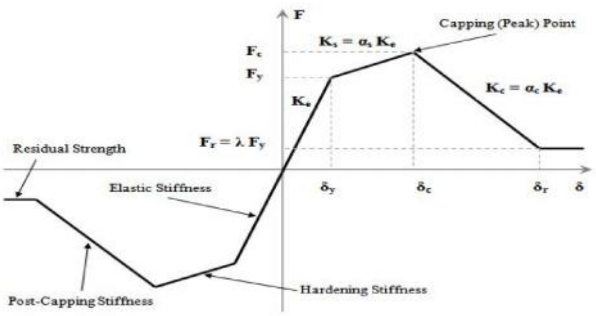


Figure 2.2. Backbone curve for Ibarra-Krawinkler hysteretic model

3. INFILLED STEEL FRAME MODELS

Three different 2D moment resisting steel frames, named F1b3f (1bay 3 story), F1b7f (1bay 7 story) & F4b3f (4bay 3 story) here, have been selected. All the frames are designed according to Iranian National Building Code (INBC) provisions (INBC, 2007), which is quite similar to the AISC-ASD design code (AISC, 1989). A significant assumption made in all of these frames is that the infill panels only withstand lateral loads, while the steel frame is designed for both gravity and lateral loads. A schematic presentation of the models is given in 'Figure 3.1'.

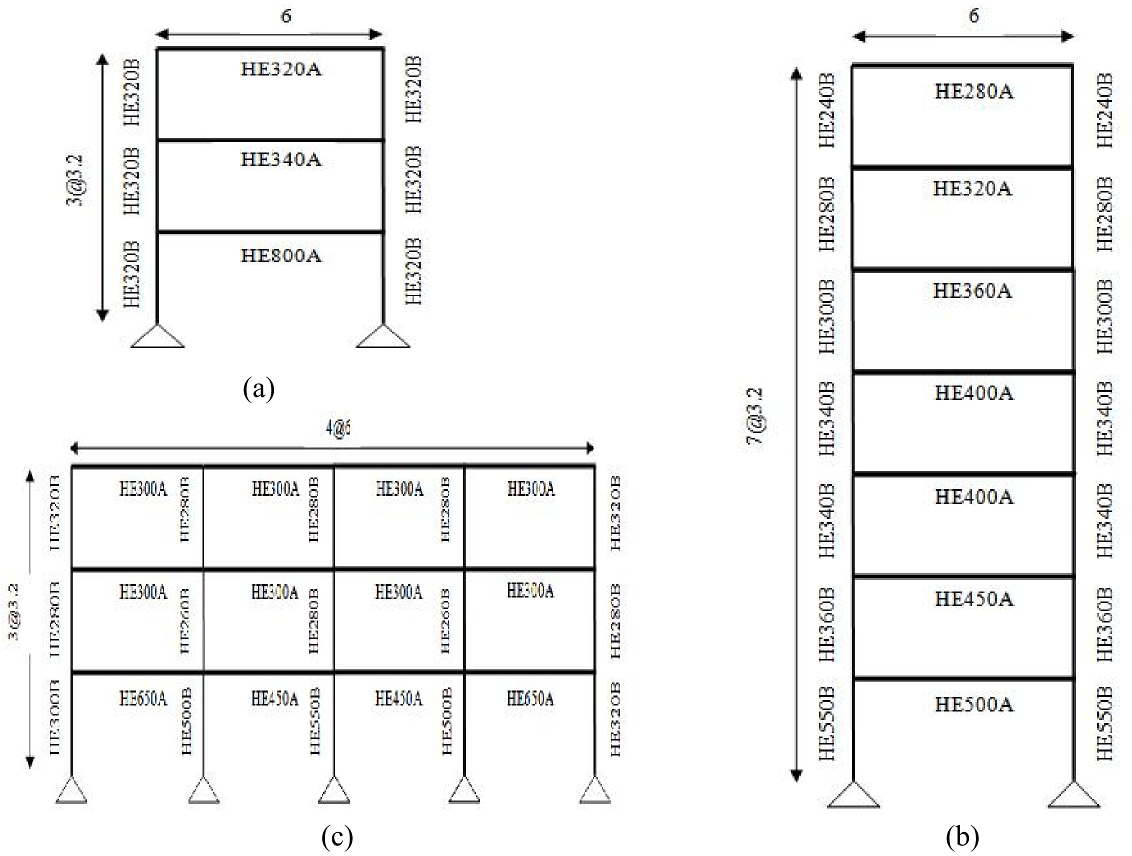


Figure 3.1. Frame models: (a) model F1b3f (b) model F1b7f (c) model F4b3f

Significant design and dynamic properties of the frames are presented in Table 1.

Table 1: Major design and dynamic properties of the frames

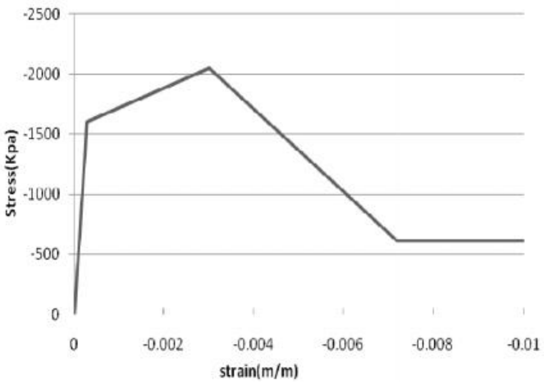
Frame	Number of Stories	Number of Bays	Natural Period(sec)	Base Shear Coefficient (C)	Equivalent Viscous Damping Ratio (ξ - %)
F1b3f	3	1	0.72	0.15	5
F1b7f	7	1	1.32	0.11	5
F4b3f	3	4	0.66	0.15	5

The bare steel frames were modeled using the Opensees software (Opensees, 2002). Material Steel01 with a yielding strength (f_y) of 2.4e8 (N/m^2) and elastic modulus equal to 2e11 (N/m^2) was used. Nonlinear Beam-Column elements with plastic hinges were used for modeling beam and column elements.

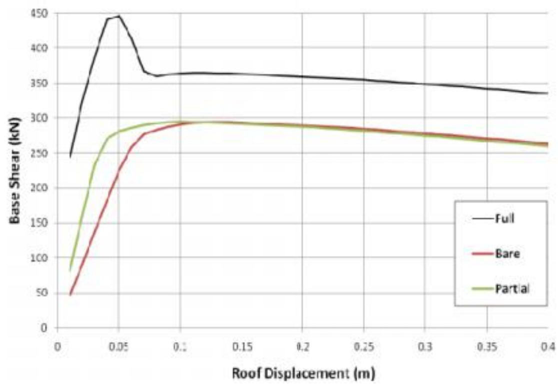
All models are infilled in three forms: (1) Bare Frames (no infill) (2) Infills in all stories (fully infilled) (3) Infills in all stories except for the first story (partially infilled).

For every frame in each model, the geometric and mechanical properties of the equivalent struts, including width (w_{strut}), length, thickness, relative stiffness parameter (λ_{strut}) and the stress-strain relationship (backbone curve) parameters, has been defined and calculated. After the above calculations, the infills were modeled in the Opensees software. The hysteretic uniaxial material was used to define the backbone behavior of the infill. In 'Figure 3.2', the backbone curve for infills located in the first story of model F1b3f and pushover curves for different infill cases of this model is illustrated. Since the geometric and mechanical characteristics of infills in different stories of model

F1b3f is very similar to each other, the backbone curve of only the first story, as a representative of all stories, is given here.



(a)



(b)

Figure 3.2. Infill panel and infill steel frame behavior of model F1b3f:
(a) Typical backbone curve of infills (b) pushover curves for different infill cases

4. ET ANALYSIS

In this study, the ETA40g (1, 2, 3) acceleration set with 40 second long excitation functions, was used for ET analysis. ETA40g01 acceleration function is depicted in 'Figure 4.1'. One horizontal component of the acceleration functions has been considered and dynamic soil-structure interaction was neglected. P-Δ effects have been considered in the analysis.

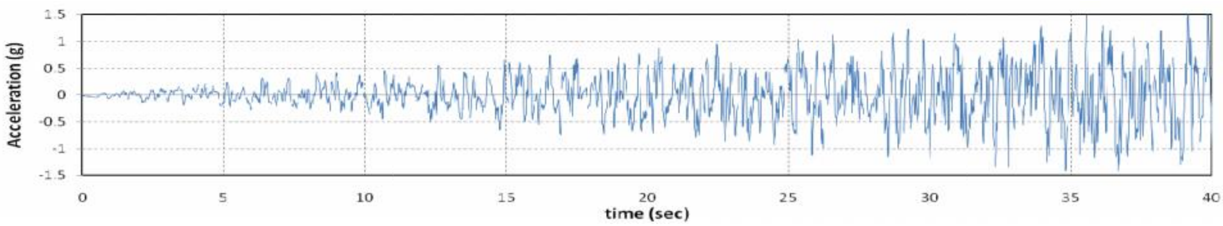


Figure 4.1. ETA40g01 accelerogram

ET analysis results are usually presented by increasing ET curves where the *y* coordinate at each time value, t, corresponds to the maximum absolute value of the required EDP in the time interval [0, t] as given in Eqn. 4.1:

$$\Omega(f(t)) \equiv Max(Abs(f(\tau)) : \tau \in [0,t]) \tag{4.1}$$

In this equation, Ω is the Max_Abs operator as defined above and f (t) is the desired response history such as base shear, interstory drift ratio, damage index or other parameter of interest. The *x* coordinate axis of an ET curve is time, which is correlated with the intensity measure (IM). As an example ET curve illustration, 'Figure 4.2' presents the drift ratio response history and corresponding ET curve of the roof (third) story of model F1b3f(Full), i.e. F1b3f model with Full infill case, subject to the ETA40g01 acceleration function.

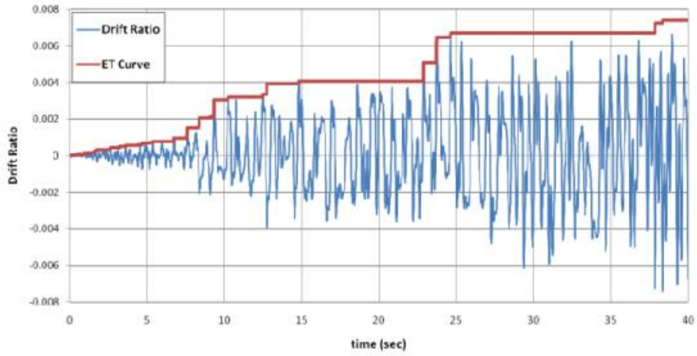


Figure 4.2. Drift Ratio response history and corresponding ET curve for F1b3f (Full) model (3rd story) subject to ETA40g01 acceleration function

As can be seen in the above figure, an ET curve is the maximum absolute value of the response parameter up to each time instant, and thus usually has a serrated and stepwise shape pattern. To attain a soft and gradual pattern, ET curves are usually smoothed using a smoothing technique.

In the following, to validate the results obtained by ET analyses, IDA is conducted and corresponding results of the two analyses for different models and infill cases are compared.

5. IDA ANALYSIS

In this study, seven ground motions were selected for IDA analysis of the models. They were chosen to be compatible, i.e. in soil conditions, fault rupture mechanism, seismic wave propagation and so on, with the seismic design spectrum of the frames. These earthquake records were selected from a set of 20 ground motions, used in the ATC-55 project (FEMA-440 report) (FEMA-440, 2005), which are recorded on stiff soil (class C) conditions. For convenience, this set of 7 ground motions is named the GM set from now on. The GM set, along with their general characteristics, are summarized in Table 2.

Table 2: Description of the GM set

No	Date	Earthquake name	Record name	Magnitude (Ms)	Station number	PGA (g)
1	01/17/94	Northridge	NRORR360	6.8	24,278	0.51
2	06/28/92	Landers	LADSP000	7.5	12,149	0.17
3	04/24/84	Morgan Hill	MHG06090	6.1	57,383	0.29
4	10/17/89	Loma Prieta	LPAND270	7.1	1,652	0.24
5	10/17/89	Loma Prieta	LPGIL067	7.1	47,006	0.36
6	10/17/89	Loma Prieta	LPLOB000	7.1	58,135	0.44
7	10/17/89	Loma Prieta	LPSTG000	7.1	58,065	0.50

In order to be consistent with seismic codes, the GM set is scaled. For this purpose, they are scaled according to ASCE-7 (ASCE, 2006) guidelines, i.e. scaled such that their 5%-damped linear spectral acceleration response is equal or greater than the ASCE-7 design spectrum (for LA with the following design parameters: S_s=1.5; S_1=0.6; F_a=1.0; F_v=1.3; T_L=8) for the period range of 0.2T_1 to 1.5T_1, where T_1 is the fundamental period of vibration of each frame modeled as a linear system. The ASCE-7 Target design spectrum is shown in 'Figure 5.1'.

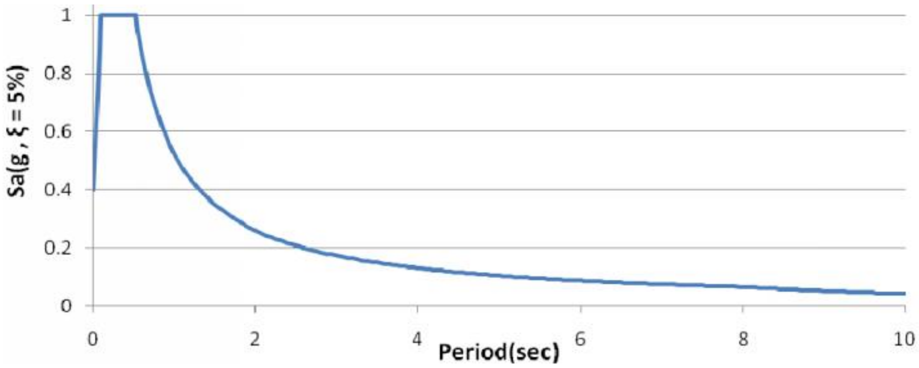


Figure 5.1. ASCE-7 design spectrum

Since the fundamental period of vibration for each frame differs for different infill cases (Bare frame, Partially infilled frame and fully infilled frame), to reduce complexity, the scale factor for each frame is obtained such that it meets the above stated criteria for all infill cases, i.e. instead of using 3 different scale factors (corresponding to 3 different infill cases) for each frame, we use one.

After selecting and scaling the ground motions (GM set) for each frame, Incremental dynamic analysis (IDA) was conducted using the Opensees software. A 5% equivalent viscous damping coefficient, typical of frame structures, was considered in all frames and infill cases. Each frame with different infill cases was subjected to increasing intensity levels of the GM set and various responses such as interstory drift, displacement, base shear and element forces were recorded. One horizontal component of the ground motions has been considered and dynamic soil-structure interaction was neglected. P-Δ effects have been considered in the analysis.

6. COMPARATIVE STUDY

An important question is how the results of ET and IDA analyses can be compared. Results of ET analysis are obtained through time and as mentioned before in this method, the time is correlated with intensity measure (IM). To establish a relation between the results of the ET method and any other method, the IM value of the other method should be found in the ET analysis. Therefore, a procedure should be defined to find an equivalent time in the ET analysis in which the IM values of the two methods are equal. Scaling over a periodic range, with scale factors given in Table 3, is useful when comparing the relative performance of structures with different periods of free vibration as applied in ET analysis. However, in order to further refine the results for individual frames and infill cases, and to compare ET and IDA results, a second scaling factor, S_2, has been applied to the time scale in order to obtain the approximate time corresponding to the actual spectral acceleration response. In this way, Eqn. 6.1 is used to correlate the scaling factor of the records with the endurance time of the specific structure via an equivalent time:

$$t_{ET} = t_{Target} \times S_1 \times S_2 \tag{6.1}$$

Where,

t_{ET} is the equivalent time in ET analysis which corresponds to an IDA analysis with ground motions scaled by the S1 scale factor, and t_{Target} is the time at which the ET acceleration functions response spectrum matches the target spectrum (ASCE-7 design spectrum with the aforementioned parameters), i.e. t_{Target} = 10 sec. S_1 is the scaling factor used to achieve different levels of acceleration response spectrum in IDA, and S_2 is the correction factor defined as the ratio between the acceleration response spectrum of a specific record($S_{aS}(T)$) and the target spectrum($S_{aC}(T)$) at the fundamental period of vibration(T_1). This factor is calculated using Eqn. 6.2:

$$S_2 = \frac{S_{aS}(T_1)}{S_{aC}(T_1)} \tag{6.2}$$

Tables 3 to 5 compare different response parameters obtained by IDA and ET analyses for different models and infill cases. These results are the average values of the GM ground motion set and series g ET acceleration functions for IDA and ET analyses, respectively. In table 3, N.A. fields refer to values that are unavailable due to numerical inconvergence or structural collapse.

Table 3: Comparison of different ET and IDA results for model F1b3f

Infill Case	*IDA scaling factor (S1)*	*Equivalent Time (sec)*	*IM [Sa/g]*	*maxIDR*		*Roof Displacement (m)*		*Base Shear (kN)*	
				ET	IDA	ET	IDA	ET	IDA
Bare	0.2	3.08	0.2217	0.0061	0.0059	0.0315	0.0361	153.7	166.1
	0.6	9.24	0.6651	0.0303	0.0257	0.0882	0.1147	296.6	292.4
	1	15.40	1.1085	0.0732	0.0674	0.1443	0.2421	310.5	318.0
	1.4	21.56	1.5518	0.1442	0.1397	0.3562	0.3682	338.8	330.1
	1.8	27.72	1.9952	0.1584	N.A.	0.5825	N.A.	392.6	N.A.
Partial	0.2	2.96	0.2505	0.0054	0.0060	0.0273	0.0246	225.6	199.9
	0.6	8.88	0.7514	0.0135	0.0215	0.0621	0.0858	296.7	295.7
	1.0	14.80	1.2523	0.0285	0.0456	0.1291	0.1919	313.5	315.4
	1.4	20.72	1.7532	0.0686	0.0891	0.1758	0.4007	323.3	340.9
	1.8	26.64	2.2541	0.0852	N.A.	0.3484	N.A.	339.8	N.A.
Full	0.2	5.14	0.5121	0.0012	0.0007	0.0113	0.0251	263.4	342.6
	0.6	15.42	1.5363	0.0021	0.0028	0.0903	0.0736	481.9	466.7
	1	25.70	2.5606	0.0049	0.0060	0.2004	0.1414	485.1	482.7
	1.4	35.98	3.5848	0.0093	0.0115	0.3705	0.2259	489.2	487.9

Table 4: Comparison of different ET and IDA results for model F1b7f

Infill Case	*IDA scaling factor (S1)*	*Equivalent Time (sec)*	*IM [Sa/g]*	*maxIDR*		*Roof displacement (m)*		*Base Shear (kN)*	
				ET	IDA	ET	IDA	ET	IDA
Bare	0.2	3.26	0.2549	0.0033	0.0037	0.0376	0.0361	504.1	476.7
	0.6	9.78	0.7648	0.0086	0.0116	0.0990	0.1053	1108.0	1090.2
	1.0	16.30	1.2747	0.0164	0.0206	0.1629	0.1677	1275.9	1265.9
	1.4	22.82	1.7846	0.0245	0.0311	0.2161	0.2501	1343.8	1328.8
	1.8	29.34	2.2945	0.0560	0.0450	0.3917	0.3534	1391.8	1357.6
	2.2	35.86	2.8044	0.0685	0.0653	0.4547	0.4804	1409.9	1415.1
Partial	0.2	3.66	0.2549	0.0050	0.0044	0.0208	0.0187	840.5	738.6
	0.6	10.98	0.7648	0.0109	0.0101	0.0589	0.0589	1338.3	1287.1
	1.0	18.30	1.8224	0.0202	0.0157	0.1434	0.1025	1388.1	1361.2
	1.4	25.62	1.7846	0.0320	0.0235	0.2343	0.1759	1403.9	1404.7
Full	0.2	4.64	0.2549	0.0005	0.0005	0.0021	0.0037	1395.5	663.6
	0.6	13.92	0.7648	0.0019	0.0027	0.0131	0.0268	1952.4	1391.9
	1.0	23.20	2.3087	0.0072	0.0057	0.0722	0.0669	2047.6	1968.7
	1.4	32.48	1.7846	0.0223	0.0127	0.1234	0.1139	2056.4	2059.8

Table 5: Comparison of different ET and IDA results for model F4b3f

Infill Case	*IDA scaling factor (S1)*	*Equivalent Time (sec)*	*IM [Sa/g]*	*maxIDR*		*Roof Displacement (m)*		*Base Shear (kN)*	
				ET	IDA	ET	IDA	ET	IDA
Bare	0.2	3.74	0.1467	0.0092	0.0102	0.0853	0.0859	270.9	248.4
	0.6	11.22	0.4401	0.0233	0.0274	0.2598	0.2090	461.6	439.9
	1.0	18.70	0.7335	0.0392	0.0466	0.3481	0.2787	509.7	487.7
	1.4	26.18	1.0269	0.0921	0.0883	0.5337	0.3576	535.2	515.8
	1.8	33.66	1.3202	0.1344	0.1296	1.0904	0.4866	576.6	557.1
Partial	0.2	3.02	0.1467	0.0042	0.0068	0.0321	0.0428	254.3	294.9
	0.6	9.06	0.4401	0.0187	0.0227	0.1187	0.1553	503.7	505.2
	1.0	15.10	1.0284	0.0233	0.0361	0.2038	0.2360	580.9	567.0
	1.4	21.14	1.0269	0.0582	0.0577	0.2685	0.3126	605.5	593.2
	1.8	27.18	1.3202	0.1276	0.0938	0.5601	0.4331	615.5	611.3
	2.2	33.22	1.6136	0.2981	0.1014	0.9632	0.5748	625.7	622.2
	2.6	39.26	1.9070	0.3365	0.1468	1.9752	0.9530	765.2	657.5
Full	0.2	3.36	0.1467	0.0016	0.0017	0.0326	0.0312	339.1	299.5
	0.6	10.08	0.4401	0.0071	0.0079	0.1193	0.1348	549.7	560.5
	1.0	16.80	1.6676	0.0117	0.0169	0.2010	0.2373	626.1	654.8
	1.4	23.52	1.0269	0.0322	0.0245	0.3410	0.2916	638.5	675.3
	1.8	30.24	1.3202	0.0419	0.0340	0.4944	0.3842	653.4	678.4
	2.2	36.96	1.6136	0.1070	0.0436	0.9209	0.4963	660.4	694.7

7. SUMMARY AND CONCLUSIONS

By investigating various response parameters of different models and infill cases, the following general conclusions can be made:

1) In models F1b3f (3 story 1 bay frame) and F1b7f (7 story 1 bay frame), for low levels of seismic intensity (IM) where the frames are still in their linear range, in all infill cases, the maximum interstory drift ratio (maxIDR) obtained by IDA is generally larger than those of ET; while for higher levels of IM, ET predicts higher demands. In the bare cases, this difference is small while in the full infill cases, this variation is much greater (up to 40% in very high IM's). As for roof displacements, in all infill cases, for small IM's, ET and IDA results are in good agreement with each other but for higher IM's, ET shows slightly higher values (an average 15%). In all infill cases and intensity levels, the base shear results are quite close to each other.

2) In model F4b3f (3 story 4 bay), for low levels of IM, in all infill cases, the maximum interstory drift ratio (maxIDR) obtained by the two methods are comparable; while in higher levels of IM, ET predicts higher demands. In the bare case of model F4b3f, as like models F13f and F1b7f, this variation is quite small while in the full case, it is much larger (around 40% in some cases). By looking at values of roof displacements, it can be seen that for all intensity levels, in the bare case, ET and IDA results are close to each other, while in the partial and full infill cases, ET predicts higher values of roof displacements (approximately 40% in average). This higher prediction is salient in the full infill case. In the Bare and Partial infill cases, the base shear amounts of the two methods are in good agreement with each other while in the full case, ET produces higher values than IDA.

These differences in results, especially in the bare and partial infill cases (because of sudden formation of soft story mechanism in the first story), are mainly due to P-Δ effects and its influence on amplifying deterioration of strength and stiffness in structural elements of the frames (Ibarra & Krawinkler, 2005(6)). Another important probable cause of variation in results can be the scaling method of ground motions (Riahi & Estekanchi, 2010). Even though the acceleration spectrum used for scaling of the GM set ground motions, and also development of the ET series g acceleration functions, are the same, i.e. ASCE-7 design spectrum, but it should be noted that the ET series g acceleration functions have been produced in such a way to completely match the target spectrum in the target time as far as possible in both intensity (Sa) and general shape; but the earthquake records have been scaled up to approximately match only the intensity of the target spectrum. Thus the general shape of response spectrums of these ground motions differs vastly in shape with each other and with the target spectrum. This difference in the shape of response spectrums with the target spectrum, and moreover different duration of ground motions, leads to considerable variability in results among different records while this variability is much lower among ET acceleration functions because of their matching criteria stated.

In summary, for lower levels of seismic intensity which the frames are generally in their linear range, ET predicts acceptably close structural responses to IDA for all infill cases. As intensity levels increase and the frames exhibit nonlinear behavior, discrepancy grows among the results of ET and IDA. This variation is greater in the full case of all models and is more salient in model F4b3f.

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