Technical Note

Seismic Performance of Reinforced Concrete Frame Buildings Designed by Iranian Seismic Code

Elham Rezaei [1] ***and Ali Massumi*** [2*]

1. M.Sc. of Structure Engineering, School of Engineering, Kharazmi University, Iran
2. Associate Professor, Department of Civil Engineering, School of Engineering, Kharazmi University, Iran, * Corresponding Author; email: massumi@khu.ac.ir

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ABSTRACT

In this paper, the seismic performance of a multi-story reinforced concrete frame building has been studied. A typical reinforced concrete moment-resisting frame building was designed according to the current Iranian seismic code (IS 2800-14). Seven earthquake records were selected and scaled based on IS 2800-14 requirements. In order to assess the seismic vulnerability of the case study structure, nonlinear static (push-over) analysis and nonlinear dynamic time-history analysis have been conducted. The performance has been evaluated based on both the member and global level criteria. Comparison between push-over and nonlinear analyses results shows a relatively good consistency. The numerical results additionally show that the case-study building frames designed by IS 2800-14 satisfy the intended code requirements and meet the inter-story drift and maximum plastic rotation demands suggested by Guide 360 (Instruction for Seismic Rehabilitation of Existing Buildings).

Keywords:
Reinforced concrete frame; Seismic performance; Push-over analysis; Time-history analysis

1. Introduction

Earthquakes are well-known as the most devastating natural phenomenon causing severe damages in infrastructures. Since many buildings in Iran are exposed to the excitations induced by strong earthquakes, the common low-rise reinforced concrete buildings need to be designed based on their performance during earthquakes.

In the linear static procedure, recommended by IS 2800-14 [1] (the current Iranian code), the inelastic behaviour of structures subjected to design-level earthquakes is considered through a response modification factor. However, an inelastic analysis procedure, gives detailed information on seismic performance of structures, globally and locally, during an excitation.

In nonlinear analysis approaches, the main parameter of the structural behaviour is an estimate of the lateral force resisting system's ultimate displacement capacity. In this method, structural characteristics, such as effective stiffness, are the determining factors on which the expected displacement demand is based. Moreover, the results are relatively straightforward and consistent with design solutions.

Early studies of static nonlinear (push-over) analysis used simple procedure in which the first mode was considered in computation of the capacity curves. The works of Freeman et al [2], Saiidi and Sozen [3], and Fajfar and Fischinger [4] are amongst the first studies. Over the last decades, more guidance codes and documents have been written based on inelastic displacement rather than elastic forces [5]. There are also research studies, investigating the available procedures for seismic assessment of typical RC frames according to the Iranian code [6].

The objective of this paper is to investigate the seismic behaviour of an RC frame building, designed according to Iranian codes, using both nonlinear static and dynamic analysis methods. The structural response is assessed to determine the overall safety of the structure under seismic demands in terms of inter-story drift, base-shear versus building drift, and element plastic rotation.

2. Case Study Building Description

The five-story RC building has three and five bays in the north-south (N-S) and east-west (E-W) direction, respectively. Interior and exterior frames are intermediate concrete moment frames as the lateral resistance system. As Figure (1) shows, the building is regular in plan and height, and it is considered to be located in Tehran, where the seismic hazard is very high [1]. The soil type II was chosen as the location's soil type that accords with a rock or stiff soil the thickness of which is equal or greater than 30 m, and can be considered as equivalent to type B in the USGS classification [7].

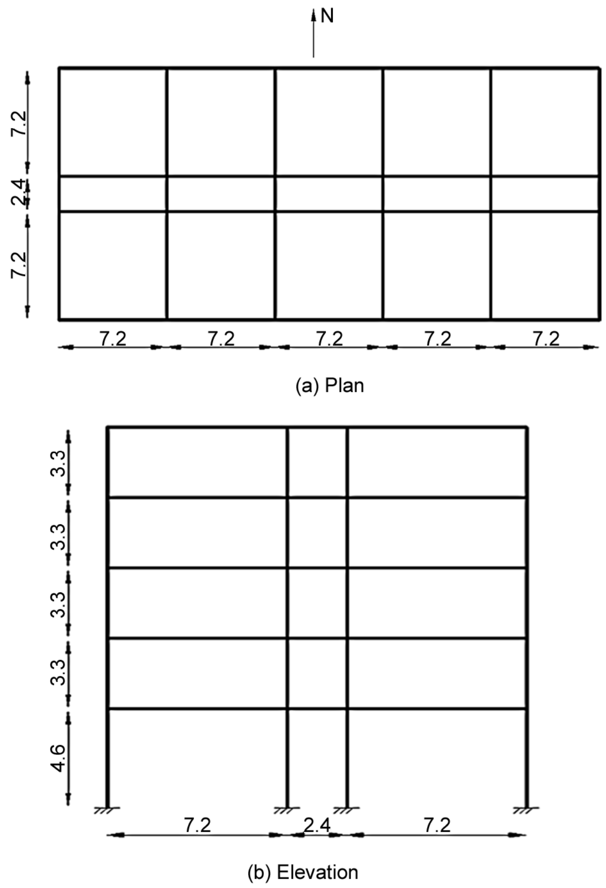

Figure 1. Plan and elevation of the case study building (unit: m)

The building frames were designed according to Iranian concrete code (ABA) [8]. The size of beams are 0.3×0.6 m in cross-section and longer exterior bays, but 0.3×0.4 m in beams with shorter bays. The dimensions of columns are 0.4×0.4 and 0.5×0.5 m, which consist of 20 (20 mm in diameter) bars. Figure (2) depicts the details of reinforcement in both beams and columns.

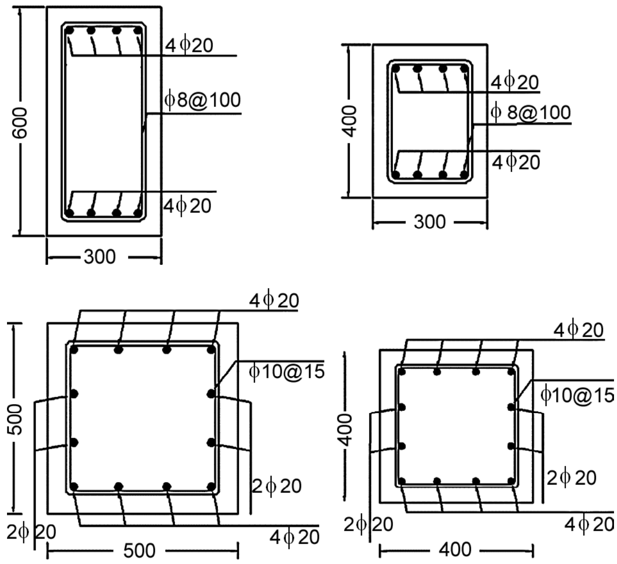

Figure 2. Member cross sections (dimensions in mm, @ is used between stirrup size and spacing).

3. Modelling and Evaluation Criteria

3.1. Analytical Model

The building frame models have been analysed and designed in SAP2000 software, employed for common linear analysis practice of reinforced concrete buildings [9]. All the details of the frame building were regarded as such buildings are typically constructed [8, 10]. After designing the models based on Iranian code, they have been analysed for nonlinear characteristics in PERFORM 3D [11] software, which is a highly focused nonlinear tool for seismic analysis whether it is static or dynamic.

Due to the symmetry in plan, two-dimensional mathematical models of each frame were used to maintain computational efficiency. The diaphragms were considered to demonstrate rigid behaviour, since the building floor plan is rectangular and has the aspect ratio of less than 3:1 [12].

Two main concerns in modelling a beam or a column element are: 1) sufficiently accurate relationship between force and corresponding deformation (F-D relationship); and 2) reasonably accurate deformation capacities, to assess performance in member level. Beams and columns were simulated in accordance with the force-deformation (F-D relationship) curve in Guide 360 [13], representing the relationship between force and the corresponding deformation in a member [11].

It is assumed that the frame members primarily bear bending moment, and the F-D relationship characteristics are controlled by flexural elasticity and plasticity. Therefore, the F-D relationships for beam elements have been designated based on their moment curvature diagrams. In defining the F-D relationship for columns, the effect of axial load was considered in the definition of moment-rotation relationship for each column.

Guide 360 suggests that when using nonlinear dynamic analysis method, the complete cyclic behaviour (hysteretic behaviour) of every element should be modelled based on experimental results or a known cyclic behaviour model. The overall F-D relationship, illustrated in Figure (3), can be used as the backbone of the cyclic behaviour, showing the sever reduction in stiffness and strength in elements [13].

In order to define backbone, based on the best-fit models for reinforced concrete members [14], initial and post-yielding stiffness have been calculated. The values of initial and post-yielding stiffness are then directly entered in PERFORM-3D program. Guide 360 suggests that the post-yielding stiffness should be 0-10 percent of the initial stiffness [13]. Given the structural characteristics, 10 percent has been selected [14].

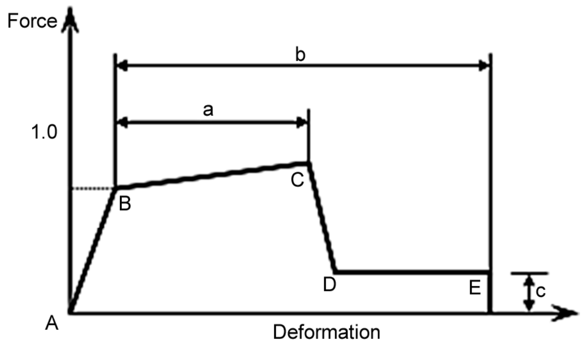

Figure 3. RC members F-D relationship.

In RC members, high shear force leads to pinching in load-member displacement curves, which is mainly caused by bond slippage and closing cracks, developed in plastic hinge zone. Since there were no considerable shear forces and the bars were assumed to have no sliding along, the F-D relationships were modelled without any pinching effect [15].

Moreover, the rotational responses of RC members in this study did not show demands beyond the C point in Figure (3), thus there was no need to regard any degradation in strength, when defining the F-D relationship [14]. Although the structure was analysed in case of strength loss occurrence, and as expected, no sign of noticeable changes in seismic demands was observed.

Capacity values, adopted to define deformation capacities, were based on Guide 360 [13]. In Guide 360, as in FEMA 356, the end rotation capacities for concrete members are given as plastic rotations. Moreover, columns were fixed at base and considered to have rigid end zone type of connection at joints with beams.

In modelling phase, the basic components of a member are: 1) stiff end zones at the two ends of elements, and 2) inelastic segment defined based on Guide 360 specifications. In PERFORM 3D, FEMA concrete member type was chosen for the inelastic segment, since the model specifications of Guide 360, for beams and columns, are very similar to those of FEMA 356.

Figure (4) illustrates a PERFORM-3D frame compound component for the chord rotation model. The crucial parts of this model are the FEMA member components. The model has two of these components, which are of finite length with nonlinear properties, to account for two different strengths at the member ends. In PERFORM-3D, the FEMA member type component, implement the chord rotation model for concrete beam or column [11]. Using a chord rotation model as shown in Figure (4), it is then converted to the model shown in Figure (5).

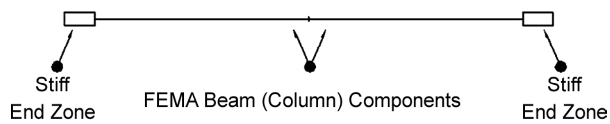

Figure 4. Basic components for chord rotation model.

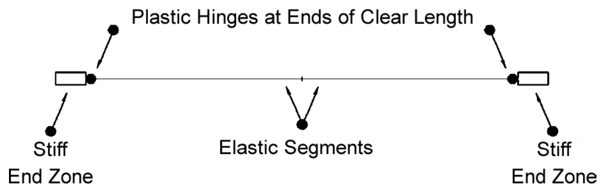

Figure 5. Implementation of chord rotation model.

3.2. Failure Criteria

The failure criteria that were used to assess the probable occurrence of a specific performance level included global and member-level limit states at each story. In local level evaluation, three performance criteria, Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP), set the plastic rotation limits and accordingly, provide a detailed estimation of member behaviour during seismic excitations. In global-level evaluation, comparison between the inter-story drift ratios of standard and those of the structure establishes the assessment procedure.

4. Push-Over Analysis

4.1. Global Performance

Push-over or nonlinear static analysis as a viable developed method of analysis [3,16, 17], has the main advantage of predicting the elastic and the inelastic behaviour of structures, offering reliable information about seismic details [18]. Push-over results are usually presented in displacement versus base-shear, depicting structure's overall response. Figure (6) shows the push-over curve results for both uniform and inverted triangular load patterns.

Inter-story drift ratio is selected as a criterion to emphasize the performance of structure. Figure (7) shows inter-story drifts for the two frames.

Inter-story drift ratio limit, according to the Iranian code, is 0.025. This value is for the buildings with up to five stories, under design earthquake [1]. This criterion is set to 2% for life safety performance level and 4% for collapse prevention performance level, based on ASCE SEI 41-06 [19]. The inter-story drift line graphs in Figure (7) show that both the distribution and the magnitude of the inter-story drifts are closely analogous. There is also a uniformity in the inter-story drift of each frame, displaying characteristics of elastic behaviour at the inter-story drift ratio below 0.5%. On the other hand, there is a significant amount of inter-story drift in the first and second stories above 0.5% inter-story drift, denoting inelastic behaviour. Moreover, between the two different push-over load patterns, the uniform pattern shapes a less conservative behaviour in both frames.

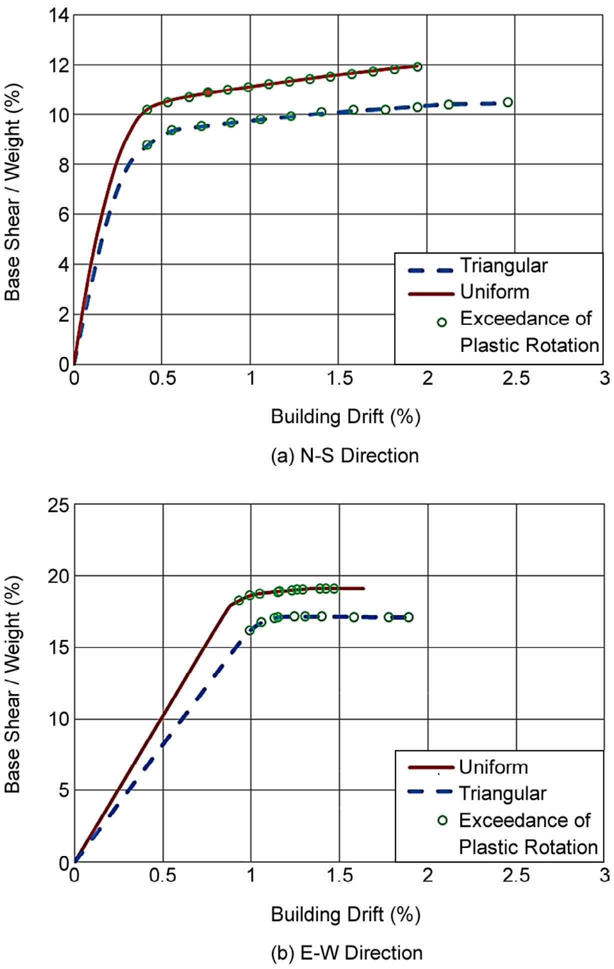

Figure 6. Push-over analysis results.

4.2. Member-Level Response

Plastic rotation limits specified by Guide 360, for life safety and collapse prevention performance levels of the case-study beams are 0.01 and 0.02, respectively. For those of the case-study columns, the limits are 0.012 and 0.016. Figure (8) shows the progress and positions of plastic hinges beyond CP limit for both uniform and inverted triangular lateral load patterns.

5. Nonlinear Time History Analysis

Nonlinear time-history analysis was conducted for the N-S direction frame in addition to push-over analysis. The reason for choosing this specific frame was that it has shallower beams in its middle span. Nonlinear time-history particular approach provides the most realistic possible structural response in comparison to push-over analysis; therefore, in this study, it has been used as verification diagnostic [10].

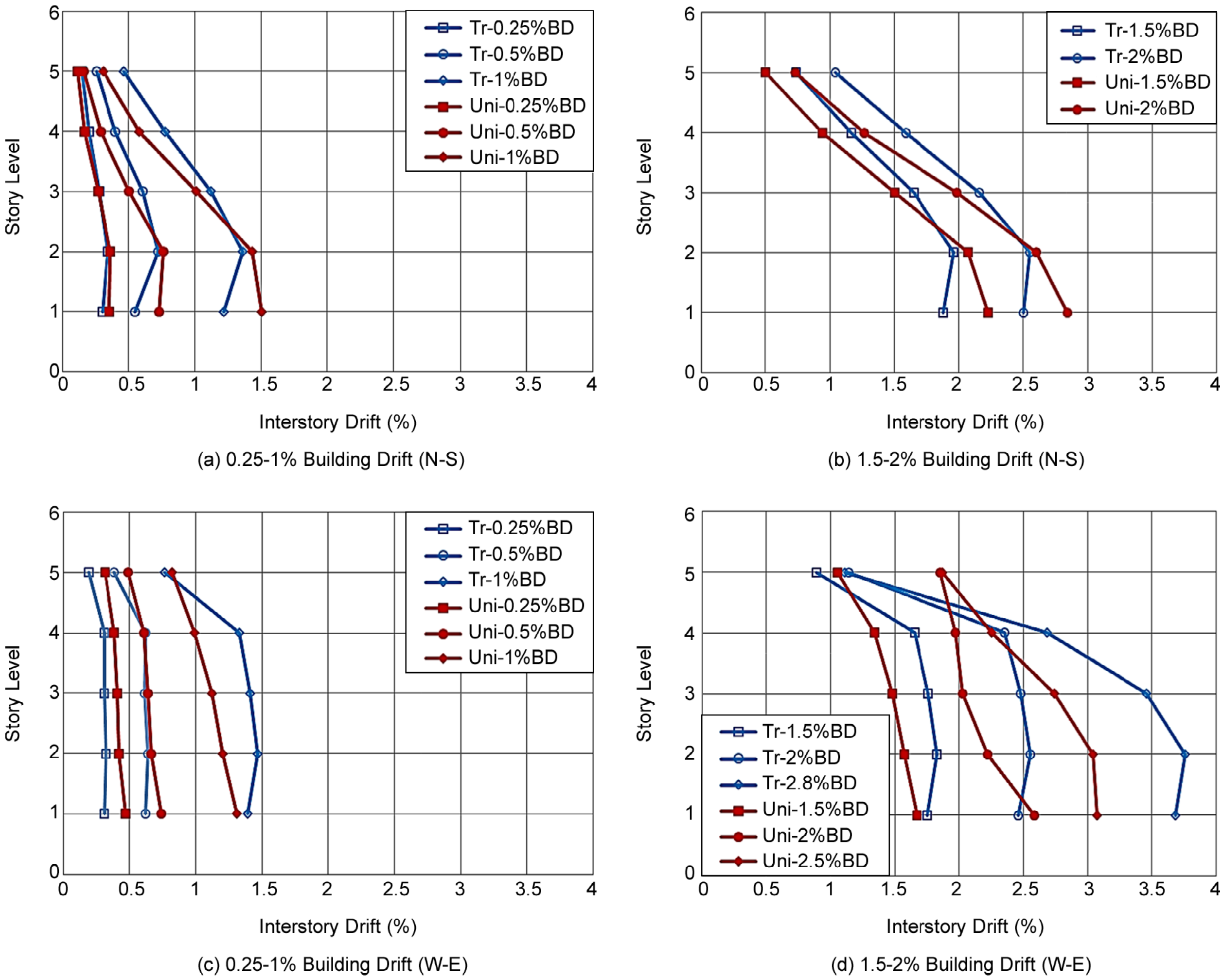

Figure 7. Inter-story drifts at different building drifts from push-over analysis (Tr., Uni. and BD indicate Triangular load pattern, uniform load pattern, and building drift, respectively).

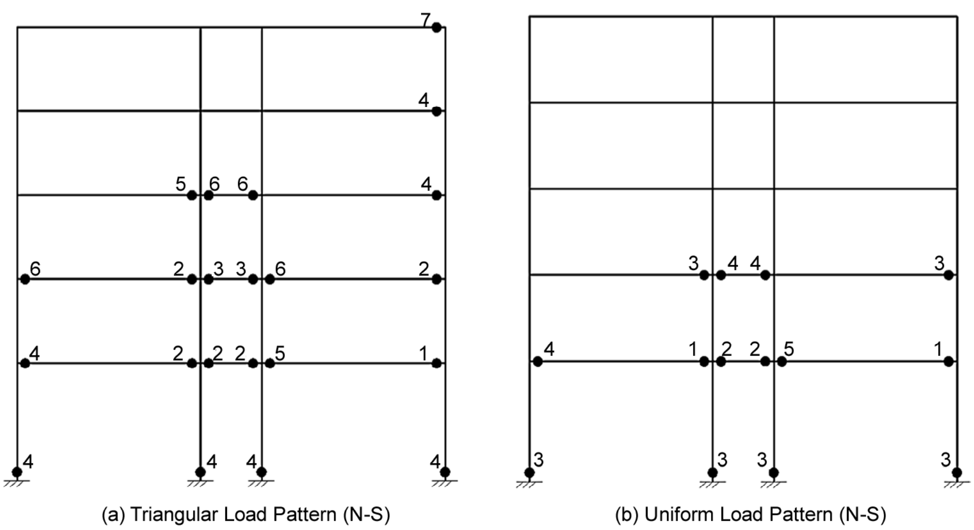

Figure 8. Sequence of exceedance of Guide 360 plastic rotation limits during push-over analysis.

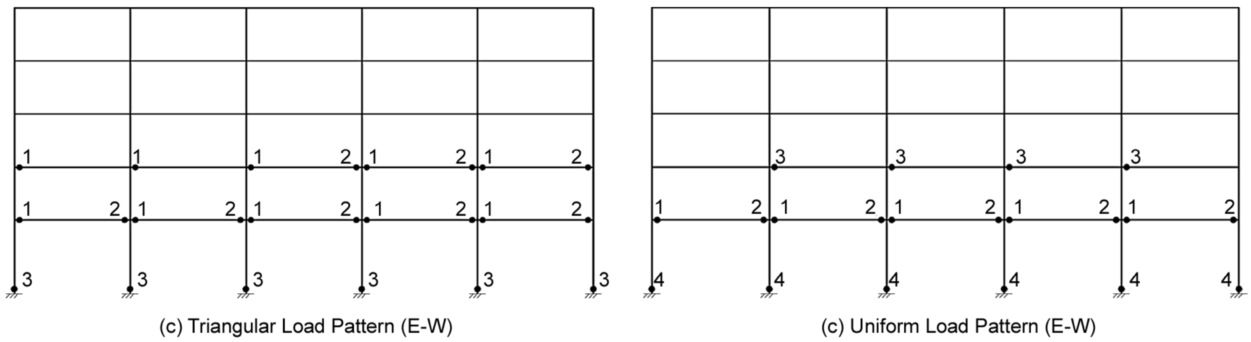

Figure 8. Continue.

5.1. Acceleration Time Histories

Since there are no acceleration records available for the location of the case-study building, the process of obtaining reliable data can be demanding. The criteria on which the selection of the earthquake records is based are: 1) the status of geophysics of the location, and 2) the specifications of the ground motion [1]. In other words, the selected earthquake records should belong to the same type as the case-study location soil; besides, the records should have response spectra in agreement with the target design spectrum. To do so, seven earthquake acceleration records were selected based on the aforementioned criteria with regarding the following three specific items: 1) a minimum magnitude of 6 in Richter scale, 2) type II for soil profile type, according to IS 2800-14, which is rock or stiff soil with more than 30 cm thickness; and 3) peak ground acceleration (PGA) of more than 0.2g. The basic characteristics of the selected earthquake records are provided in Table (1). The scaling procedure of the seven earthquake records was carried out according to IS 2800-14.

Subsequently, each of the motion records was scaled to its maximum PGA magnitude, to produce a record with the exact same frequency content. Afterwards, the acceleration response spectra of the newly scaled motion records with respect to 5% damping were modified to match the standard design spectra, between the period range of 0.2 T and 1.5 T, where T is the fundamental period of the structure.

5.2. Global Performance

Nonlinear dynamic analysis was carried out, using seven selected accelerogram records. Table (2) provides the maximum inter-story drift and base-shear ratio for each of the adjusted earthquake records. The comparison of dynamic and push-over analysis results, illustrated in Figure (9) shows an overall consistency between the two methods, up to 0.5% building drift. This indicates that since the structural behaviour depends mostly on the effect of the first mode in 0-0.5% building drift range, push-over and dynamic curves share a similar trend. While beyond 0.5% building drift, the dynamic results are underestimated with the push-over ones, because the effect of higher modes are not considered in the push-over analysis.

Table (2) gives the summary of maximum inter-story drift, building drift and base-shear ratio. The inter-story drift for the seven scaled earthquakes have been provided in Figure (10). As detailed in

Table 1. Selected earthquake records [20].

Earthquake	Station	EQ ID	Date	Magnitude	PGA(g)	Duration(s)
Irpinia	Brienza	A-BRZ000	23-11-1980	6.90	0.22	34.98
Northridge	Fletcher	FLE 234	17-1-1994	6.69	0.21	29.98
Manjil	Abbar	Abbar	21-06-1990	7.37	0.49	45.98
Tabas	Dayhook	Tabas	16-09-1978	7.35	0.41	20.98
Chi-Chi	TCU045	TCU045	20-09-1999	7.62	0.47	89.995
Chi-Chi	TCU047	TCU047	20-09-1999	7.62	0.36	89.995
Northridge	Univ. Hospital	UNI005	17-01-1994	6.69	0.35	39.99

Table 2. Summary of maximum drift and base-shear ratio.

Ground Motion	Max. Building Drift (%)	Max. Base-Shear Ratio (%)	Max. Inter-Story Drift (%)
A-BRZ000	0.96	14	1.28
FLE 234	1.56	16	2.75
Abbar	1.67	12.32	2.22
TCU045	1.56	15	2.28
TCU047	2.11	16	2.89
Tabas	1.84	13.07	2.79
UNI005	0.96	17	2.27
Average	1.52	14.77	2.35

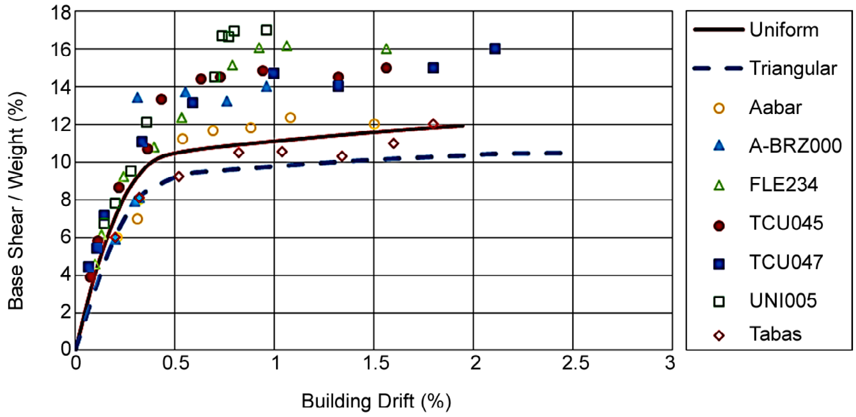

Figure 9. Comparison of push-over analysis and maximum response from dynamic analysis.

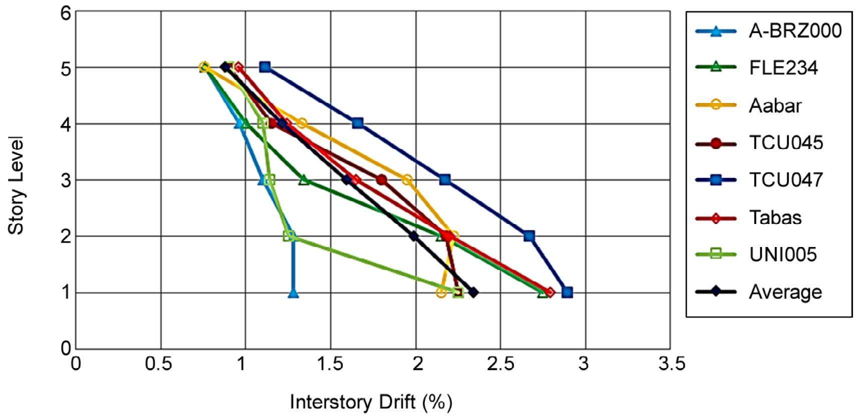

Figure 10. Inter-story drift ratios from dynamic analysis using ground motion records.

Table (2) and Figure (10), the average inter-story drift ratios satisfy the IS 2800-14 provision for not exceeding the limit of 2.5%.

5.3. Member-Level Response

Guide 360 provides the plastic rotation limits, based on the ratio of reinforcement, confinement, and the ratio of shear demand to strength, for beams and columns that have flexural dominant behaviour. As shown in table 3, the average plastic rotation of the first story columns has exceeded the corresponding LS and CP limit values and is in accordance with the push-over results.

6. Conclusions

In this paper, the seismic behaviour of a five-story reinforced concrete building, designed based on Iranian codes, was assessed. In order to evaluate the

Table 3. Member-level evaluation.

Story Level	Beam Rotation (rad)			Column Rotation (rad)		
	Guide 360 Limits		Average Max. Plastic Rotation	Guide 360 Limits		Average Max. Plastic Rotation
	LS	CP		LS	CP	
1	0.01	0.02	0.021	0.012	0.016	0.0165
2	0.01	0.02	0.019	0.012	0.016	0.0043
3	0.01	0.02	0.016	0.012	0.016	0.0037
4	0.01	0.02	0.021	0.012	0.016	0.0023
5	0.01	0.02	0.018	0.012	0.016	0.00235

structural performance, nonlinear static (push-over) and dynamic analyses were conducted. Accordingly, seven adjusted earthquake records were employed. Both global and local level performances were observed. The numerical results provided the following major points:

- Push-over analysis can provide reliable data on the elastic as well as the inelastic behaviour of the frame structures. If adequately modelled, nonlinear static analysis provides insight into the locations which are prone to experience large seismic deformations. This, of course, calls for certain degree of attention in lateral load pattern selection, as well as the F-D relationship definition.
- The push-over analysis results show that there is no failure mechanism tendency in the building stories. The well-known soft-first-story mechanism, has been avoided thanks to the IS 2800-14 provisions.
- The results obtained for the frames confirm that a push-over analysis provides reliable estimates about seismic behaviour of low-rise buildings [18], although there is no absolute consistency between push-over and dynamic results.
- As the push-over curves have underestimated the seismic capacity of the structure above the building drift of 0.5%, it can be concluded that the displacement response estimation based on the push-over curves and design demand diagram are not realistic. This is due to the fact that, the fixed load distributions do not have the capability to capture higher mode effects in the inelastic range. Hence, the importance of selecting the suitable lateral load patterns is reflected [21].
- By comparing the results of the two lateral load pattern, it can be concluded that the uniform one produces a higher initial stiffness and base-shear. That is, for the same base-shear force, the uniform load pattern displays a lower roof displacement demand.
- The nonlinear dynamic results indicate that the inter-story drift requirements of Iranian codes, Guide 360 and IS 2800-14, for CP level of performance, have been met.
- Although in the selection of an appropriate set of excitation records, based on IS 2800-14 provisions, a proper care has been taken, each record exhibits different set of results in terms of seismic capacity. This variety is the result of different frequency content, duration and amplitude. Future works can be aimed at performing nonlinear time history analysis using different ground motions to provide less divergent data.

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