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Moment resisting steel frames under repeated earthquakes

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Abstract. In this study, a systematic investigation is carried out on the seismic behaviour of plane moment resisting steel frames (MRF) to repeated strong ground motions. Such a sequence of earthquakes results in a significant damage accumulation in a structure because any rehabilitation action between any two successive seismic motions cannot be practically materialised due to lack of time. In this work, thirty-six MRF which have been designed for seismic and vertical loads according to European codes are first subjected to five real seismic sequences which are recorded at the same station, in the same direction and in a short period of time, up to three days. Furthermore, the examined frames are also subjected to sixty artificial seismic sequences. This investigation shows that the sequences of ground motions have a significant effect on the response and, hence, on the design of MRF. Additionally, it is concluded that ductility demands, behaviour factor and seismic damage of the repeated ground motions can be satisfactorily estimated using appropriate combinations of the corresponding demands of single ground motions.

Keywords: moment resisting steel frames; inelastic seismic analysis; repeated earthquakes; damage accumulation

1. Introduction

Modern seismic codes are based on the isolated and rare 'design earthquake' and ignore the effects of the repeated earthquake phenomena. However, a sequence of earthquakes results in a significant damage accumulation in a structure, because any rehabilitation action between two successive seismic motions cannot be practically materialized due to lack of time. Recently, Hatzigeorgiou and Beskos (2009) and Hatzigeorgiou (2010a,b,c) examined the influence of multiple earthquakes on the response of numerous single-degree-of-freedom (SDOF) systems and found that seismic sequences lead to increased displacement demands in comparison with the 'design earthquake'. With respect to multi-degree-of-freedom (MDOF) systems under seismic sequences, only few research works can be mentioned. The first one is the work of Fragiacomo *et al*. (2004) dealing with two low rise steel frames (three and five-storey high) under four different seismic sequences characterized by the repetition of one, two and three ground motions. However, according to Ruiz-

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Garcia and Negrete-Manriquez (2011), the repetition of the same record seems to be inappropriate for the realistic prediction of structural behaviour. Recently, Hatzigeorgiou and Liolios (2010) examined eight reinforced concrete planar frames under numerous real and artificial sequential ground motions. Thus, the need for the study of the inelastic seismic response of low-, medium- and high-rise steel framed structures to sequential ground motions is apparent.

This paper presents an extensive parametric study on the inelastic response of steel planar moment resisting frames (MRF) which are subjected to sequential strong ground motions. Thirty-six planar steel (MRF) designed for seismic and vertical loads according to European codes EC3 (1993) and EC8 (2005) by Karavasilis *et al*. (2007) are subjected to two families of seismic sequences. More specifically, the first one consists of five as-recorded seismic sequences which have been recorded at the same station, in the same direction and in a short period of time, up to three days, while the second one comprises sixty artificial seismic sequences. The time-history responses of these steel frames are evaluated by means of the structural analysis software RUAUMOKO (Carr 2008).

Comprehensive analysis of the created response databank is employed in order to derive important and useful results. More specifically, this study focuses on the results which are related to the following critical parameters: local or global structural damage, maximum displacements, interstorey drift ratios, development of plastic hinges and response using the incremental dynamic analysis (IDA) method (Vamvatsikos and Cornell 2002). Additionally, the accumulation of permanent displacements due to multiple earthquakes is also examined. Finally, simple and effective empirical expressions are developed to estimate ductility demand, behaviour factor and seismic damage of the sequential ground motions from the counterparts of single ground motions.

2. Description and modelling of structures

Thirty-six planar steel framed structures representing low-, medium- and high-rise MRF are considered in this study. These frames are regular and orthogonal with storey heights and bay widths equal to 3 m and 5 m, respectively. Furthermore, they have the following characteristics: number of stories n_s with values 3, 6, 9, 12, 15 and 20 and number of bays n_b with values 3 and 6.

Gravity load on the beams is assumed to be equal to 27.5 KN/m (dead and live loads of floors). The yield stress of the material was set equal to 235 MPa. The frames have been designed in accordance with the provisions of structural Eurocodes EC3 (1993) and EC8 (2005). The expected design ground motion was defined by the acceleration response spectrum of EC8 (2005) with a peak ground acceleration (PGA) equal to 0.35 g and a soil class B. Data of the frames, including values for n_s, n_b, beam and column sections and fundamental natural period of vibration, are presented in Table 1 taken from Karavasilis *et al*. (2007) and reproduced here for reasons of completeness. In that table, expressions of the form, e.g. 260 – 360 (1-4) + 240 – 330 (5-6) mean that the first four (1-4) stories have columns with HEB260 sections and beams with IPE360 sections, whereas the next two (5-6) higher stories have columns with HEB240 sections and beams with IPE330 sections (Androic 2000).

The relation between the fundamental period and the number of storeys of the structures appears in Fig. 1, where it is evident that the examined MRF database covers a wide range of periods.

An inelastic structural multi-degree of freedom (MDOF) system with linear viscous damping is used to model any of the frames considered. The dynamic equilibrium equation of such a system is given in the form (Chopra 2006)

Table 1 Steel moment resisting frames considered in parametric studies

Frame	n_s	n_b	Sections (Columns HEB – Beams IPE)
1	3	3	240 – 330 (1-3)
2	3	3	260 – 330 (1-3)
3	3	3	280 – 330 (1-3)
4	3	6	240 – 330 (1-3)
5	3	6	260 – 330 (1-3)
6	3	6	280 – 330 (1-3)
7	6	3	280 – 360 (1-4) + 260 – 330 (5-6)
8	6	3	300 – 360 (1-4) + 280 – 330 (5-6)
9	6	3	320 – 360 (1-4) + 300 – 330 (5-6)
10	6	6	280 – 360 (1-4) + 260 – 330 (5-6)
11	6	6	300 – 360 (1-4) + 280 – 330 (5-6)
12	6	6	320 – 360 (1-4) + 300 – 330 (5-6)
13	9	3	340 – 360 (1) + 340 – 400 (2-5) + 320 – 360 (6-7) + 300 – 330 (8-9)
14	9	3	360 – 360 (1) + 360 – 400 (2-5) + 340 – 360 (6-7) + 320 – 330 (8-9)
15	9	3	400 – 360 (1) + 400 – 400 (2-5) + 360 – 360 (6-7) + 340 – 330 (8-9)
16	9	6	340 – 360 (1) + 340 – 400 (2-5) + 320 – 360 (6-7) + 300 – 330 (8-9)
17	9	6	360 – 360 (1) + 360 – 400 (2-5) + 340 – 360 (6-7) + 320 – 330 (8-9)
18	9	6	400 – 360 (1) + 400 – 400 (2-5) + 360 – 360 (6-7) + 340 – 330 (8-9)
19	12	3	400 – 360 (1) + 400 – 400 (2-3) + 400 – 450 (4-5) + 360 – 400 (6-7) + 340 – 400 (8-9) + 340 – 360 (10) + 340 – 330 (11-12)
20	12	3	450 – 360 (1) + 450 – 400 (2-3) + 450 – 450 (4-5) + 400 – 450 (6-7) + 360 – 400(8-9) + 360 – 360 (10) + 360 – 330 (11-12)
21	12	3	500 – 360 (1) + 500 – 400 (2-3) + 500 – 450 (4-5) + 450 – 450 (6-7) + 400 – 400 (8-9) + 400 – 360 (10-11) + 400 – 330 (12)
22	12	6	400 – 360 (1) + 400 – 400 (2-3) + 400 – 450 (4-5) + 360–400 (6-7) + 340 – 400 (8-9) + 340–360 (10) + 340 – 330 (11-12)
23	12	6	450 – 360 (1) + 450 – 400 (2-3) + 450 – 450 (4-5) + 400 – 450 (6-7) + 360 – 400 (8-9) + 360 – 360 (10) + 360 – 330 (11-12)
24	12	6	500 – 360 (1) + 500 – 400 (2-3) + 500 – 450 (4-5) + 450 – 450 (6-7) + 400 – 400 (8-9) + 400 – 360 (10-11) + 400 – 330 (12)
25	15	3	500 – 300 (1) + 500 – 400 (2-3) + 500 – 450 (4-5) + 450 – 400 (6-7) + 400 – 400 (8-12) + 400 – 360 (13-14) + 400 – 330 (15)
26	15	3	550–300 (1) + 550 – 400 (2-3) + 550 – 450 (4-5) + 500 – 400 (6-7) + 450 – 400 (8-12) + 450 – 360 (13-14) + 450 – 330 (15)
27	15	3	600 – 300 (1) + 600 – 400 (2-3) + 600 – 450 (4-5) + 550 – 450 (6-7) + 500 – 450 (8-9) + 500 – 400 (10-12) + 500 – 360 (13-14) + 500 – 330 (15)
28	15	6	500 – 300 (1) + 500 – 400 (2-3) + 500 – 450 (4-5) + 450 – 400 (6-7) +400 – 400 (8-12) + 400 – 360 (13-14) + 400 – 330 (15)
29	15	6	550 – 300 (1) + 550 – 400 (2-3) + 550 – 450 (4-5) + 500 – 400 (6-7) + 450 – 400 (8-12) + 450 – 360 (13-14) + 450 – 330 (15)
30	15	6	600 – 300 (1) + 600 – 400 (2-3) + 600 – 450 (4-5) +550 – 450 (6-7) + 500 – 450 (8-9) +500 – 400 (10-12) + 500 – 360 (13-14) + 500 – 330 (15)

Table 1 Continued

Frame	n_s	n_b	Sections (Columns HEB – Beams IPE)
31	20	3	600 – 300 (1) + 600 – 400 (2-3) + 600 – 450 (4-5) + 550 – 450 (6-10) + 500 – 450 (11-13) + 500 – 400 (14-16) + 450 – 400 (17) + 450 – 360 (18-19) + 450 – 330 (20)
32	20	3	650 – 300 (1) + 650 – 400 (2-3) + 650 – 450 (4-5) + 600 – 450 (6-10) + 550 – 450 (11-13) + 550 – 400 (14-16) + 500 – 400 (17) + 500 – 360 (18-19) + 500 – 330 (20)
33	20	3	700 – 300 (1) + 700 – 360 (2) + 700 – 400 (3) + 700 – 450 (4-5) + 650 – 450 (6-10) + 600 – 450 (11-13) + 600 – 400 (14-16) + 550 – 400 (17) + 550 – 360 (18-19) + 550 – 330 (20)
34	20	6	600 – 300 (1) + 600 – 400 (2-3) + 600 – 450 (4-5) + 550 – 450 (6-10) + 500 – 450 (11-13) + 500 – 400 (14-16) + 450 – 400 (17) + 450 – 360 (18-19) + 450 – 330 (20)
35	20	6	650 – 300 (1) + 650 – 400 (2-3) + 650 – 450 (4-5) + 600 – 450 (6-10) + 550 – 450 (11-13) + 550 – 400 (14-16) + 500 – 400 (17) + 500 – 360 (18-19) + 500 – 330 (20)
36	20	6	700 – 300 (1) + 700 – 360 (2) + 700 – 400 (3) + 700 – 450 (4-5) + 650 – 450 (6-10) + 600 – 450 (11-13) + 600 – 400 (14-16) + 550 – 400 (17) + 550 – 360 (18-19) + 550 – 330 (20)

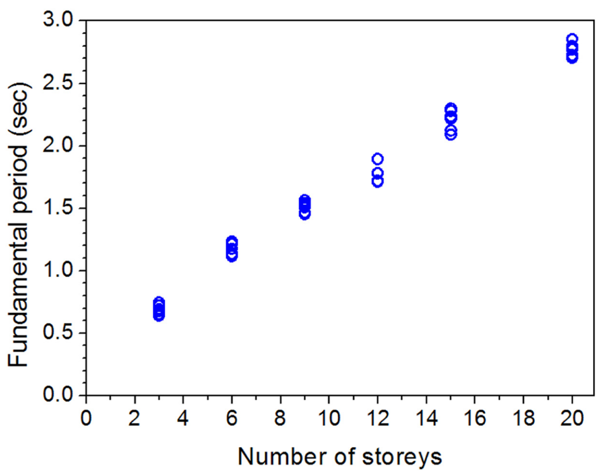

Fig. 1 Relation between numbers of storeys and fundamental period

$$M\ddot{u} + C\dot{u} + K^T u = -Ma_g \tag{1}$$

where M is the mass matrix, u the relative displacement vector, C the viscous damping matrix, K^T the tangent (inelastic) stiffness matrix, a_g the acceleration vector of the ground motion and the upper dots stand for time derivatives. The solution of the above equation of motion can be obtained by a stepwise time integration with iterations at every time step with the aid of the RUAUMOKO program (Carr 2008), which is an advanced finite element program for seismic analysis of framed structures. A brief description of the modelling details is provided in the following. Thus, in this work, a two-dimensional model of each structure is created in RUAUMOKO (Carr 2008) to carry out nonlinear dynamic analysis. Each beam-column finite element has two nodes and three degrees of freedom at each node. The finite element formulation is based on the displacement method of structural analysis. Beam and column elements are modelled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of beams and columns and assuming elastic-plastic kinematic linear hardening model ($H = 3\%$) material behaviour, as shown in Fig. 2.

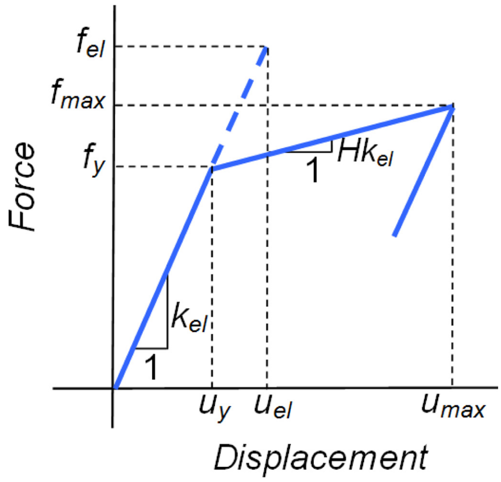

Fig. 2 Bilinear elastoplastic hysteretic model

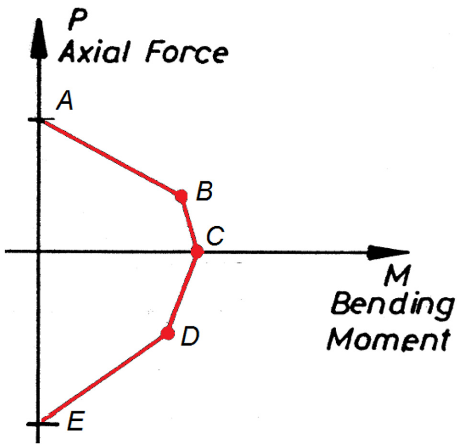

Fig. 3 *P-M* interaction diagram (Carr 2008)

Beam axial forces are assumed to be zero since all floors are considered to be rigid in plan to account for the diaphragm action of floor slabs. Characteristic input data for strength that are required by RUAUMOKO (Carr 2008) are the bending moment-axial force interaction diagrams for columns (see Fig. 3) and bending strength values for beams. Beams and columns are connected together by rigid joints without taking into account panel zone effects. The soil-structure interaction phenomenon is not examined, assuming fixed base conditions. Second-order effects (p-Δ effects) are taken into account.

3. Seismic input

The examined MRF are subjected to two sets of seismic sequences. The first strong ground motion set that has been used here consists of five real seismic sequences, which have been recorded during a short period of time (up to three days), at the same station, in the same direction and almost at the same fault distance. These seismic sequences are the following: Mammoth Lakes (May 1980-5 events), Chalfant Valley (July 1986-2 events), Coalinga (July 1983-2 events), Imperial Valley (October 1979-2 events) and Whittier Narrows (October 1987-2 events) earthquakes. The complete list of these earthquakes, which were downloaded from the strong motion database of the Pacific Earthquake Engineering Research (PEER) Center (PEER 2011) appears in Table 2.

These records are compatible with the soil class B assumed for the seismic design process, as mentioned in the previous section. Every sequential ground motion record from the PEER database becomes a single ground motion record (serial array) by applying a time gap equal to 100 sec between two consecutive seismic events. This gap has zero acceleration ordinates and is completely adequate to cease the motion of any structure due to damping before the action of the next event. Thus, Fig. 4 shows the time histories of the examined seismic sequences.

The elastic spectra for this database and for a viscous damping ratio $\xi = 2\%$, an appropriate value for steel structures, are presented in Fig. 5. For compatibility reasons with the design process, the seismic sequences are normalized to have maximum PGA equal to 0.35 g (Table 2). Thus, the

Table 2 Seismic input data - Real seismic sequences

No.	Seismic sequence	Station	Comp.	Date (Time)	Magnitude (M_L)	Recorded PGA(g)	Normalized PGA(g)
1	Mammoth Lakes	54099 Convict Creek	N-S	1980/05/25 (16:34)	6.1	0.442	0.350
				1980/05/25 (16:49)	6.0	0.178	0.142
				1980/05/25 (19:44)	6.1	0.208	0.165
				1980/05/25 (20:35)	5.7	0.432	0.341
				1980/05/27 (14:51)	6.2	0.316	0.250
2	Chalfant Valley	54428 Zack Brothers Ranch	E-W	1986/07/20 (14:29)	5.9	0.285	0.224
				1986/07/21 (14:42)	6.3	0.447	0.350
3	Coalinga	46T04 CHP	N-S	1983/07/22 (02:39)	6.0	0.605	0.289
				1983/07/25 (22:31)	5.3	0.733	0.350
4	Imperial Valley	5055 Holtville P.O.	HPV315	1979/10/15 (23:16)	6.6	0.221	0.350
				1979/10/15 (23:19)	5.2	0.211	0.334
5	Whittier Narrows	24401 San Marino	N-S	1987/10/01 (14:42)	5.9	0.204	0.336
				1987/10/04 (10:59)	5.3	0.212	0.350

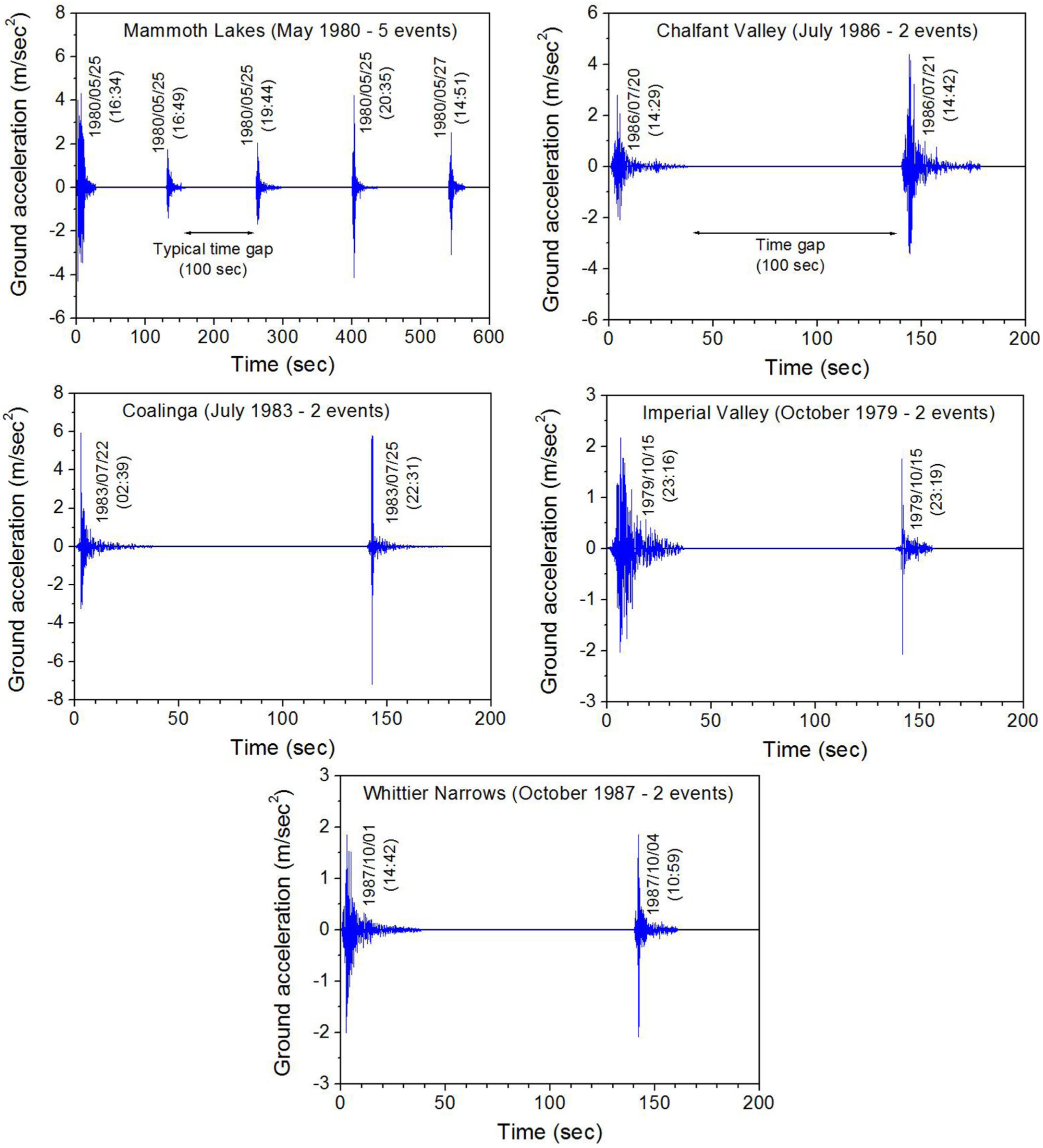

Fig. 4 Ground acceleration records of the examined seismic sequences

aforementioned sequential ground motions are multiplied by: 0.7919 (Mammoth Lakes), 0.7830 (Chalfant Valley), 0.4775 (Coalinga), 1.5837 (Imperial Valley) and 1.6509 (Whittier Narrows).

The second strong ground motion set that has been used here consists of sixty (60) artificial seismic sequences. Firstly, 10 single artificial records (from now on referred to as $R01{\sim}R10$) are generated by using the specialized software SRP (Karabalis *et al.* 1992) to make 10 real seismic records compatible with respect to the characteristics of the design process i.e., Type 1 spectrum of EC8 (2005), Soil B conditions and for PGA = 0.35 g, as described in Hatzigeorgiou and Beskos (2009). Based on the principles of engineering seismology (Gutenberg and Richter 1954, Joyner and Boore 1982), Hatzigeorgiou and Beskos (2009) showed that for every seismic event with PGA

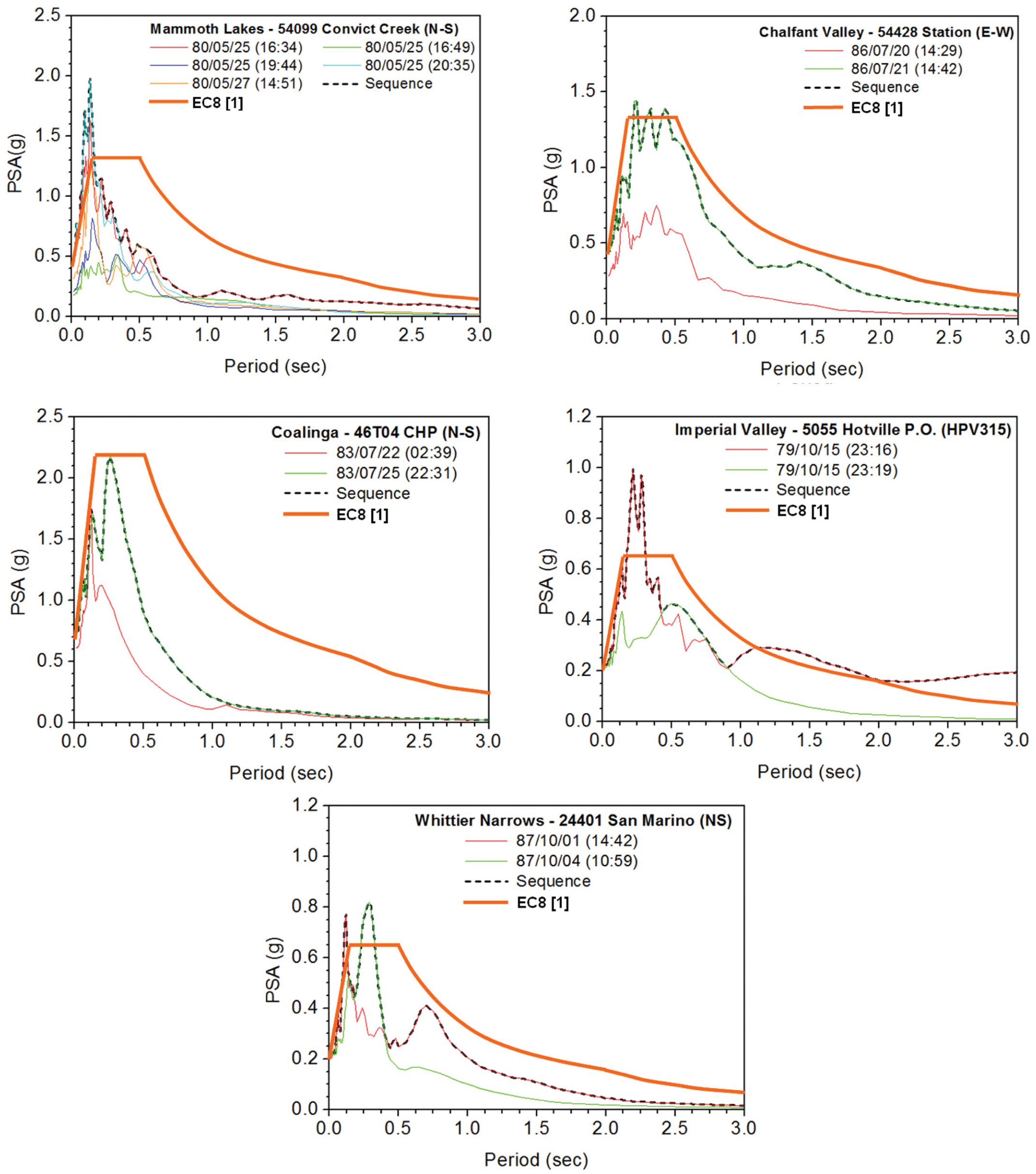

Fig. 5 Response spectra of the examined real seismic sequences

equal to $A_{g,max}$, there will be 2 earthquakes with PGA equal to $0.8526{\cdot}A_{g,max}$ and 3 earthquakes with PGA equal to $0.7767{\cdot}A_{g,max}$. In reality, structures can be subjected to any possible combination of the above events. Thus, three different sets are considered here (Hatzigeorgiou and Beskos 2009):

(a) 20 synthetic sequences of two seismic events with identical PGA = 0.8526·0.35 g = 0.298 g,

(b) 20 synthetic sequences of three seismic events with identical PGA = 0.7767·0.35 g = 0.272 g and

(c) 20 synthetic sequences of three seismic events with dissimilar PGA's where, without loss of generality, the two smaller events (with PGA = 0.8526·0.35 g = 0.298 g) precede and are followed by the bigger one (with PGA = 0.35 g). Assuming that these events appear in a short period of time, they can be characterized as the foreshock, the mainshock and the aftershock.

Similarly to the real records, every artificial sequential ground motion becomes a single ground motion record (serial array) by applying a time gap equal to 100 sec between two consecutive seismic events.

4. Response of MRF under seismic sequences

This section presents selected results for the inelastic behaviour of the considered MRF under repeated earthquakes. Each of these frames is firstly analyzed for the vertical loads. Then, with the deformed shape taken as the initial displaced shape, nonlinear time history analysis is carried out for the whole gamut of the aforementioned seismic sequences by direct time-integration using Newmark's average acceleration technique (Chopra 2006) available in RUAUMOKO (Carr 2008).

This study focuses on the following basic design parameters: local or global damage index according to Park and Ang (1985) and Krawinkler and Zohrei (1983) models, maximum horizontal floor displacements and interstorey drift ratios. Furthermore, the incremental dynamic analysis (IDA) technique is also applied to examine the performance of MRF under multiple earthquakes. Finally, the development of permanent displacements is also investigated.

4.1 Determination of cumulative damage

The Park and Ang (1985) damage index (DI), the best known and most widely used one, is defined for an element as a combination of maximum deformation and hysteretic energy as

$$\mathrm{DI} = \frac{\delta_m}{\delta_u} + \frac{\beta}{\delta_u P_y}\int dE_h \tag{2}$$

where δ_m is the maximum deformation of the element, δ_u is the ultimate deformation, β is a model constant parameter to control strength deterioration ($\beta = 0.025$ for well-designed steel structures), $\int dE_h$ is the hysteretic energy absorbed by the element during the earthquake and P_y is the yield strength of the element. Furthermore, the Krawinkler and Zohrei (1983) damage index, which is based on the concept of fatigue of steel structures, is also considered. These two local damage models can also be extended to the storey or the whole structure (global damage index), by appropriate summation of local damage indices. Thus, a global damage index, DI_G, which is a structure damage index related to the global seismic behaviour of the structure comprising m-members, can be determined as

$$\mathrm{DI}_G = \sum_i^m \mathrm{DI}_i^2 / \sum_i^m \mathrm{DI}_i \tag{3}$$

where DI_i is the local (member) damage index of the i-member (Vasilopoulos and Beskos 2006).

Fig. 6 shows the local damage according to Park and Ang (1985) and Krawinkler and Zohrei

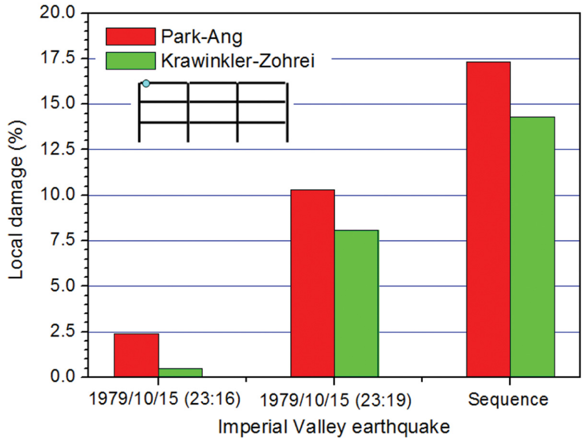

Fig. 6 Local damage (upper-left beam) for a 3-storey MRF under the Imperial Valley earthquakes

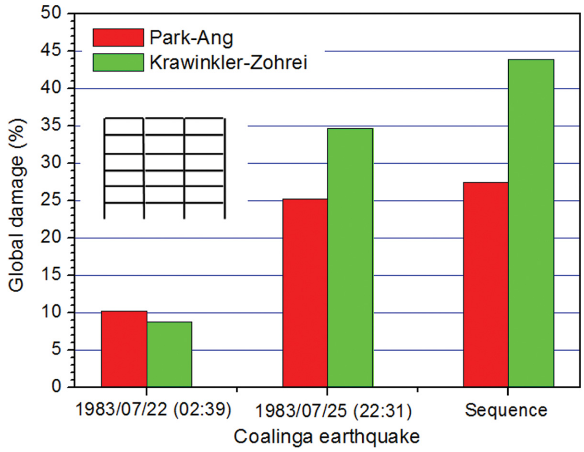

Fig. 7 Global damage for a 6-storey MRF under the Coalinga earthquakes (1983)

(1983) of the upper-left beam of a 3-storey/3-bay MRF under the Imperial Valley earthquakes (1979), examining both the single seismic events and the seismic sequence. Comparing the worst/ maximum damage value for the single seismic records with the case of seismic sequence, this structural parameter increases by 71% and 72%, according to Park and Ang (1985) and Krawinkler and Zohrei (1983) models, respectively.

Furthermore, Fig. 7 shows the global (total) damage in terms of Park and Ang (1985) and Krawinkler and Zohrei (1983) damage indices for a six-storey/three-bay MRF under the Coalinga earthquakes (1983), examining both the single seismic events and the seismic sequence. Comparing the worst/maximum damage value for the single seismic records with the case of seismic sequence, this structural parameter increases by 12% and 27%, according to Park and Ang (1985) and Krawinkler and Zohrei (1983) models, respectively.

Additionally, Fig. 8 depicts the global damage in terms of Park and Ang (1985) damage index for 3 MRF structures with 3, 6 and 9 storeys under the *R*01, *R*07 and *R*09 artificial single events and the *R*01-07-09 artificial seismic sequence.

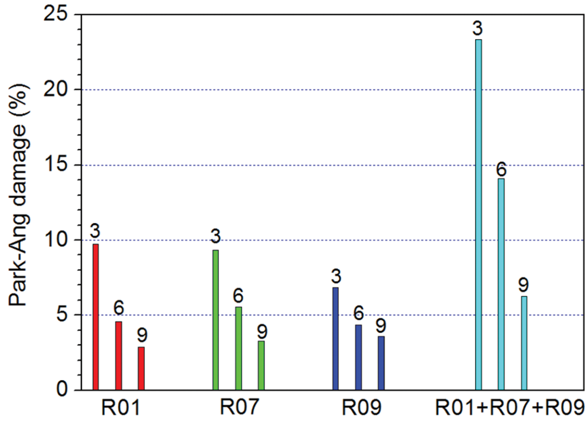

Fig. 8 Global damage for 3-, 6- and 9-storey MRF structures

From all the above results and additional ones not shown here due to lack of space, it is evident that seismic sequences lead to increased damage, both at local and global level, in comparison with the corresponding single seismic events. However, the majority of the existing investigations and all the seismic codes examine these parameters only for the 'idealized' case of single earthquakes.

4.2 Maximum seismic displacements

This section examines the maximum seismic displacements of the considered steel framed structures. Thus, the maximum horizontal displacement profiles of a three-storey/three-bay MRF under the Imperial Valley earthquakes (1979) are shown in Fig. 9, both for single and sequential ground motions.

Furthermore, Fig. 10 examines the maximum seismic displacements of a 20-storey MRF under an artificial seismic sequence (*R*01-04-09) in comparison with the corresponding responses under the single seismic records (*R*01, *R*04 and *R*09).

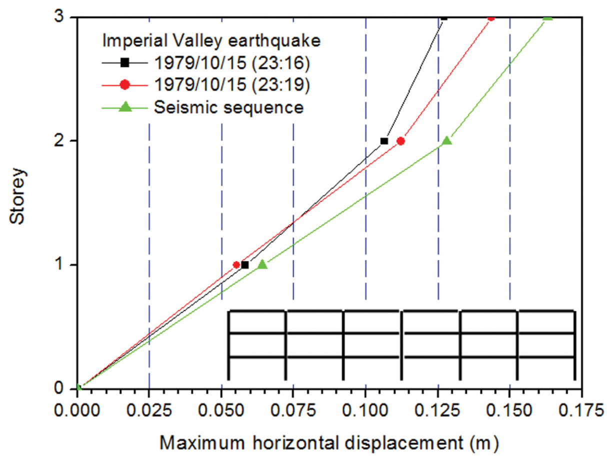

Fig. 9 Maximum displacements for a 3-storey MRF under the Imperial Valley earthquakes

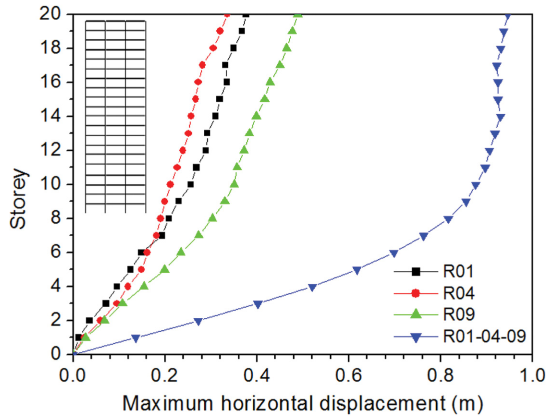

Fig. 10 Maximum displacements for a 20-storey MRF under an artificial seismic sequence

It is evident that, due to the multiplicity of earthquakes, increased displacement demands are required where, in this example, the maximum displacement for the case of seismic sequence appear to be increased by 100% in comparison with the worst/maximum displacement for any of the examined single records.

4.3 Interstorey drift ratio (IDR)

The interstorey drift ratio (IDR) is the maximum relative displacement between two stories normalized to the storey height. According to FEMA356 (2000), values of IDR larger than 5% may lead to collapse. However, this IDR limit value does not account for effects of cumulative damage due to repeated inelastic deformation (Ghobarah *et al*. 1999) and its use may not be always correct. Thus, Fig. 11 shows the IDR of a three-storey/six-bay MRF which is subjected to the triple seismic sequence of artificial records *R*07-09-06. It is evident that the seismic sequence leads to larger IDR's in comparison with the corresponding single events. Furthermore, the examined single earthquakes lead to a stable structure (IDR < 5%), while the sequential ground motion leads to collapse (IDR > 5%).

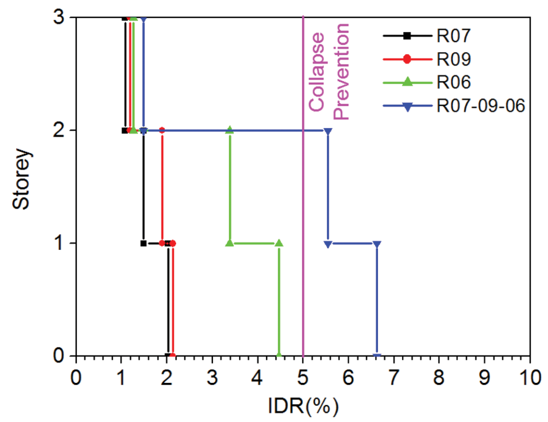

Fig. 11 IDR for a 3-storey MRF under single earthquakes and seismic sequence

4.4 Incremental dynamic analysis (IDA) technique for sequential ground motions

All the examined structures have been analyzed using the IDA technique. Fig. 12 shows selected results for a three bay/three storey MRF under the *R*01-07-04 artificial seismic sequence. Although the elastic response spectra of these seismic events and the corresponding seismic sequence are almost identical, the latter one leads to a noticeably different response and requires increased displacement demands in comparison with the separated ground motions. As it is expected, the increased displacement demands lead to higher values of drift and damage. Furthermore, Fig. 13 depicts the IDA curves versus the maximum IDR values for a twenty-storey frame under the Coalinga (1983) seismic sequence. This figure also shows (with dashed lines) the four seismic performance levels *SP*1 (negligible damage), *SP*2 (minor to moderate damage), *SP*3 (moderate to major damage) and *SP*4 (major damage) according to SEAOC Blue Book (1999) with the corresponding maximum IDR values for steel MRF being 0.5%, 1.5%, 2.5% and 3.8%, respectively. It is obvious that the seismic sequence for the same seismic intensity leads to higher values of IDR than those of the separated earthquakes, especially for the *SP*3 and *SP*4 levels.

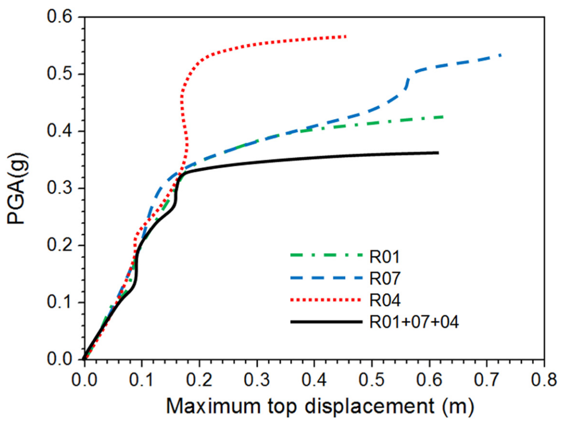

Fig. 12 IDA curves for a 3-storey MRF under single earthquakes and seismic sequence

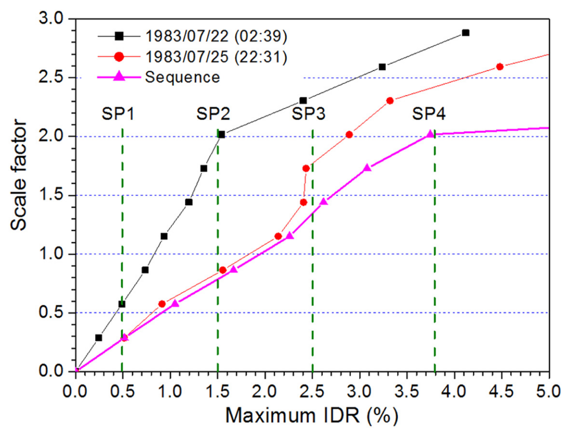

Fig. 13 IDA curves for a 20-storey MRF under Coalinga (1983) seismic sequence

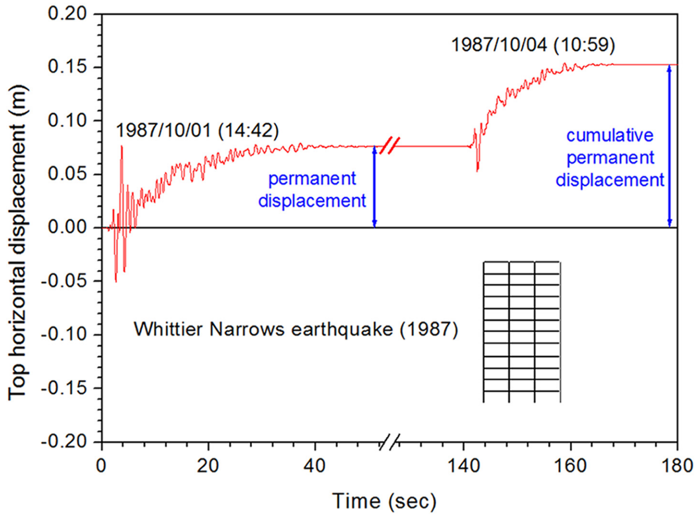

Fig. 14 Permanent displacements for a 12-storey MRF - Whittier Narrows earthquake (1987)

4.5 Development of permanent displacements for seismic sequences

In many cases, the satisfaction of the objectives at every seismic performance level requires an accurate knowledge of permanent structural displacements. This work shows that the multiplicity of earthquakes strongly influences the permanent displacements and therefore these events should be taken into account to achieve reliable estimation of this kind of displacements. Fig. 14 shows selected results for a three bay/twelve storey MRF under the Whittier Narrows (1987) seismic sequence. More specifically, this figure depicts the time history of horizontal displacement for the top of this MRF and clearly shows the accumulation of permanent displacement.

5. Estimation of ductility demands, damage and behaviour factor for multiple earthquakes

Section 4 makes clear that multiple earthquakes require increased displacement and ductility demands in comparison with the corresponding single events. This section proposes a simple and effective empirical formula to determine the ductility demands for a seismic sequence from the counterparts of single earthquakes. The global displacement ductility factor, μ, can be defined in terms of the maximum displacement u_{max} at the top level of the examined buildings and the corresponding yield displacement u_y at first yielding, as (Chopra 2006)

$$\mu = \frac{u_{max}}{u_y} \tag{4}$$

In order to estimate the cumulative ductility for a sequence of strong ground motions, various empirical expressions can be developed. This work proposes the following simple and rational relation, which has been successfully applied for RC frames (Hatzigeorgiou and Liolios 2010)

$$\mu_{seq} = 1 + \left[\sum_{i=1}^{n} \langle \mu_i - 1 \rangle^p \right]^{1/p} \quad (5)$$

Eq. (5) indicates that the cumulative ductility, μ_{seq}, for a sequence of n-strong ground motions (seismic events) results from the corresponding ductility demands, μ_i, for each one of those events, where p is a parameter controlling the combination of single ductilities and $\langle\ \rangle$ symbolizes the Macauley brackets used here in order to eliminate the influence of weak ground motions, i.e., those for $\mu_i < 1$. For example, for a triple seismic sequence with $\mu_1 < 1.0$, $\mu_2 < 1.0$ and $\mu_3 > 1.0$, Eq. (5) provides the expected ductility demand as $\mu_{seq} = \mu_3$. Parameter p can be equal to 1.0, for a simple and direct summation of ductility demands, or 2.0, which corresponds to the square root of the sum of the squares (SRSS) combination rule, a well-known procedure in earthquake engineering to obtain seismic design response. In order to achieve the best fit for the parameter p, for the examined structures and their seismic inputs, this work uses the nonlinear solver of the MS-EXCEL program, which gives the optimum value of parameter $p = 1.32$. Therefore, Eq. (5) becomes

$$\mu_{seq} = 1 + \left[\sum_{i=1}^{n} \langle \mu_i - 1 \rangle^{1.32} \right]^{1/1.32} \quad (6)$$

It should be noted that the value $p = 1.32$ is quite similar to that obtained for reinforced concrete frames (Hatzigeorgiou and Liolios 2010), which was found to be equal to 1.3048. Fig. 15 compares the cumulative ductility demands of the 'exact' dynamic inelastic analyses with those of the proposed empirical Eq. (6). Eventhough there is a dispersal of results in Fig. 15, it is clear that the model results obtained from Eq. (6) are in good agreement with those obtained from the well-accepted dynamic inelastic analyses.

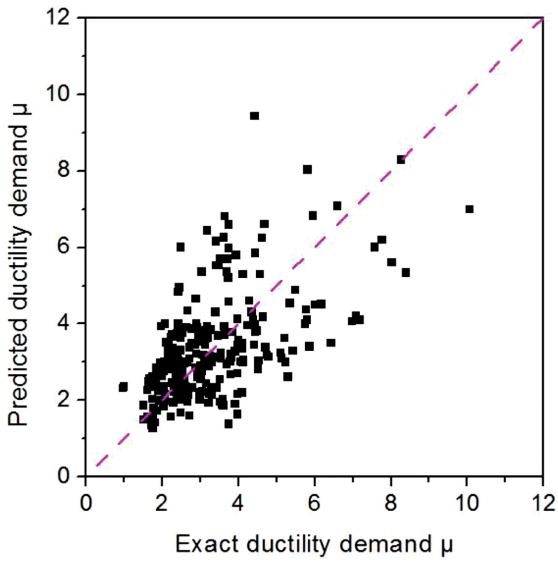

Fig. 15 Comparison of proposed model with 'exact' results

The obtained in this work response databank enables one to also establish an expression for the damage d_{sec} due to a sequence of n-earthquakes in terms of the corresponding damages d_i for the single earthquake components. Indeed a relation of the form

$$d_{seq} = \left[\sum_{i=1}^{n} d_i^r\right]^{1/r} \tag{7}$$

has been proposed and the appropriate value of the factor r for the Park and Ang (1985) damage model has been determined through nonlinear regression analysis to be $r = 1.40$. A comparison between the approximate damage predictions of Eq. (7) and the corresponding ones obtained by "exact" dynamic inelastic analyses reveals that their relation can be described by a figure very similar to Fig. 15.

In order to control the ductility demands μ (and therefore the structural damage) in the case of sequential ground motions, one can adopt an appropriate reduced behaviour factor, q_{red}. This reduction in the behaviour factor should be considered in earthquake-prone regions, where the reappearance of seismic events may have a high probability of occurrence. For a given sequential ground motion and target ductility demand μ, the reduced behaviour factor q_{red} should be obtained through an iterative and time-consuming process, which requires re-design of MRF structures in every step. This specific problem will be treated by the authors in a future work. As a practical alternative, the approximation

$$q_{red} \cong \left(\frac{\mu}{\mu_{seq}}\right) q_{des} \tag{8}$$

where q_{des} is the design code behaviour factor, can be satisfactorily used. The above approximation has been compared against the more accurate empirical expressions of Hatzigeorgiou (2010b,c) to estimate behaviour factors proposed for SDOF systems under multiple earthquakes and has been found close enough for practical purposes.

6. Conclusions

The inelastic behaviour of 36 planar steel MRF under repeated strong ground motions is examined in this paper. Five real and sixty artificial seismic sequences have been applied to study in detail these phenomena. According to this investigation, the following conclusions can be highlighted:

1. The seismic damage for multiple earthquakes is higher than that for single ground motions. Examining real seismic sequences, it is found that due to multiplicity of earthquakes, the local and global damage values can be increased by 72% and 27% or more, respectively.
2. Seismic sequences require increased displacement demands in comparison with single seismic events. Thus, it is found that due to multiplicity of earthquakes, the maximum top horizontal displacement can be increased by 100% or more. This feature is very important and should be taken into account for the seismic design of structures either by the conventional force-based or especially by the more recent displacement-based design method, which requires a high accuracy estimation of displacements. Furthermore, the displacement demands can be controlled using

appropriately reduced behaviour factors, taking into account the multiplicity effect of earthquakes.

3. The ductility demands of structures appear to be increased under sequential ground motions. A simple and effective empirical expression, which combines the ductility demands of single ground motions, is provided to estimate cumulative ductility demands due to sequential ground motions. Furthermore, simple and effective empirical expressions are also proposed for the estimation of damage and the behaviour factor under sequential ground motions.
4. The incremental dynamic analysis (IDA) technique under sequential ground motions has been investigated and has showed that seismic sequences lead to different responses than the corresponding ones for single seismic events.

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