Earthquakes and Structures, *Vol. 17, No. 3* (*2019*) *271-282*
DOI: https://doi.org/10.12989/eas.2019.17.3.271

Efficient damage assessment for selected earthquake records based on spectral matching

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(*Received April 8, 2019, Revised June 15, 2019, Accepted June 19, 2019*)

Abstract. Knowing the response of buildings to earthquakes is very important in order to ensure that a structure is able to withstand a given level of ground shaking. Thus, nonlinear dynamic earthquake engineering analyses are unavoidable and are preferable procedure in the seismic assessment of buildings. In order to estimate seismic performance on the basis of the hazard at the site where the structure is located, the selection of appropriate seismic input is known to be a critical step while performing this kind of analysis. In this paper, seismic analysis is performed for a four-story reinforced concrete ISPRA frame structure which is designed according to Eurocode 8 (EC8). A total of 90 different earthquake scenarios were selected, 30 for each of three target spectrums, EC8 spectrum, Uniform Hazard Spectrum (UHS), and Conditional Mean Spectrum (CMS). The aim of this analysis was to evaluate the average maximum Inter-story Drift Ratio (IDR) for each target spectrum. Time history analysis for every earthquake record was obtained and, as a result, IDR as the main measure of damage were presented in order to compare with defined performance levels of reinforced concrete bare frames.

Keywords: seismic damage assessment; target spectrum; earthquake records

1. Introduction

In order to ensure that a given structure is able to withstand a given level of ground shaking and maintain a desired level of performance, earthquake engineering analyses are unavoidable to obtain response of buildings to earthquakes. The most accurate method to evaluate behavior of structures during an earthquake is nonlinear dynamic analysis due to the complex, three dimensional, nonlinear, dynamic problems (Nooraie and Behnamfar 2012). It is also the most common and preferable procedure in the seismic damage assessment of buildings (Araújo *et al.* 2016). The main reasons of complex seismic damage assessment are vagueness and uncertainties related to the ground motion and structural modeling parameters of the available data such as location, size and resulting shaking intensity of future earthquakes (Deb and Kumar 2004).

In order to estimate seismic performance on the basis of the hazard at the site where the structure is located, the selection of appropriate seismic input is known as a critical step while performing nonlinear dynamic analysis (Behnamfar and Velni 2019). The current best practice in record selection is reviewed for the case of probabilistic seismic risk analysis (Cornell 1968, Bommer and Acevedo 2004, Field 2005, Baker 2008) and for code-based design (Iervolino and Manfredi 2008). Selection of recorded ground motions can be based on 1) geophysical parameters Magnitude (M) and Distance (R); 2) ground motion intensity measures; 3) spectral matching.

Earthquake magnitude (M) is one of the most important parameters used for evaluating nonlinear dynamic analysis. Due to the fact that earthquake magnitude can exert an influence on various response quantities and since released energy has a direct relation with shaking range resulting from earthquake center, it is considered to be an effective factor on selecting ground motion of the regions where there is no seismic data. Source to site distance (R) is the other basic parameter that plays a role while selecting ground motion by magnitude. That is because distance from a source of releasing energy leads to transform ground motion (Nooraie and Behnamfar 2012). Intensity measures, however, quantify the effect of ground motion records on structure. Today the most common measure of seismic intensity is peak ground acceleration, PGA and the first mode period spectral acceleration (Sa), where PGA has natural connection with inertial forces, and for specific types of structures (very stiff structures) maximum dynamic force, which appears in the structure, is directly relative to PGA (Pejovic and Jankovic 2015, 2017).

Spectral matching is the most commonly proposed earthquake record selection method by seismic codes and,

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ISSN: 2092-7614 (Print), 2092-7622 (Online)

as such, can be utilized in the framework of both force-based and performance based design (Katsanos *et al*. 2010). In the past 15 years, the use of spectrum matched records has become increasingly widespread for the estimation of nonlinear structural response. Spectrum matched records are artificially generated time histories of ground motion acceleration, or other relevant parameter, whose response spectral shapes are matched to a predetermined target spectrum and used as input to dynamic analysis (Carballo and Cornell 2000). While selecting ground motion records, the main aim is to select representative records of the ground motion at the site of interest and at the consistent source-to site distance. Also, there will generally be a requirement to ensure that the records of ground motion conform to some specified level of agreement with the ordinates of the design response spectrum, depending on whether records are selected by performing searches in terms of response spectral ordinates or in terms of seismological and geophysical parameters (Bommer and Acevedo, 2004). If structural response is to be estimated by selecting ground motions to match a target response spectrum, typical response spectrum associated with the specified large amplitude *Sa* value at a single period must be found (Whittaker *et al*. 2011).

Current codes such as Eurocode 8 - Part 1 (2011), the American Standard ASCE41-13 (2014), the New Zealand Standard NZS 1170.5:2004 (2004) implicitly recognize the variability of the seismic response of buildings introduced by input ground-motions by setting a minimum number of records. Number of records that need to be selected and scaled, vary from code to code, and with those records the aim is to provide realistic estimates of mean seismic demands.

In part 1 of Eurocode 8 (2011) the following criteria are established for the selection and scaling of ground motion records in the context of demand-based assessments of buildings: 1) the mean of the zero period spectral response acceleration values calculated from the individual time histories should not be smaller than the value of $a_g S$ for the site under study, a_g being the design ground acceleration on rock and S the soil parameter; and 2) within the range of period of $0.2\,T_1$ and $2.0\,T_1$, where T_1 is the fundamental period of the structure in the direction where the record will be applied, no value of the mean 5% damping elastic spectrum, calculated from all time histories, should be less than 90% of the corresponding value of the 5% damping elastic response spectrum (Araújo *et al*. 2016), (Bommer and Acevedo, 2004). In EC8 after the elastic response spectrum, seismic input for time-history analysis is defined. Two spectral shapes are defined, Type 1 and Type 2, where the latter applies if the earthquake contributing most to the seismic hazard has surface waves magnitude not greater than 5.5. On the contrary, the former type should be used.

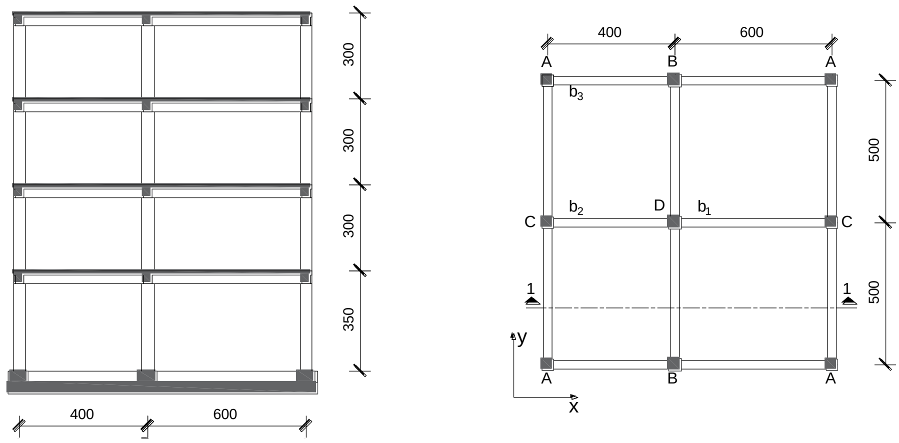
For the past two decades, the Uniform Hazard Spectrum (UHS) has been used as the target spectrum in design practice. It is computed by Probabilistic Seismic Hazard Analysis (PSHA) and constructed by selecting a frequency of exceedance for the hazard, developing the spectral acceleration at each period and plotting spectral acceleration versus period (Whittaker *et al*. 2011, Bulajić *et al*. 2012). It is suggested to select seven records which are compatible with the dominant earthquake scenario at the site of interest and that scenario is represented with magnitude (*M*) and the distance (*R*) which are key parameters obtained by disaggregation analysis (Bazzurro and Cornell 1999). In order to match the design level of the UHS, the selected records need to be scaled when it is necessary. Baker and Cornell (2016) concluded that the UHS represents nearly impossible earthquake scenario because the rate of observing a high positive ε (measure of how ground motion deviates from the expected mean) at all periods is much lower than the rate of observing a high ε at any single period. Although UHS is frequent target spectrum in structural dynamic analysis, it does not represent a spectrum caused by a single earthquake at a given site and thus leads to a conservative spectrum in higher hazard levels (Mousavi *et al*. 2012).

Thus Baker (2011) presented an alternative, Conditional Mean Spectrum (CMS) that provides the expected (mean) response spectrum, conditioned on occurrence of a target spectral acceleration value at the period of interest. Procedure for computing CMS involves:

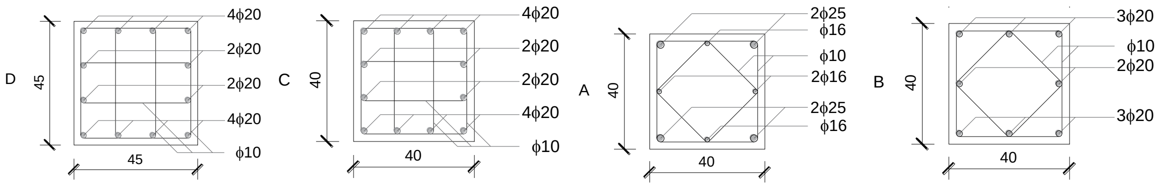
1. Determination of the target *Sa* at a given period T and the associated geophysical parameters *M*, *R* and ε;
2. Computation of the mean and standard deviation of the response spectrum, given *M* and *R*;
3. Computation of ε at other periods, given $\varepsilon\,(T^*)$;
4. Computation of CMS.

After it is computed, CMS can be used for selection of ground motion records. With CMS spectral shape associated with the target spectral shape $Sa(T^*)$ is obtained. Thus ground motions that match $Sa(T^*)$ can be treated as representative of ground motions that naturally have the target $Sa(T^*)$ value. The period ranges over which the CMS should be matched must be identified to find ground motions matching a target CMS, and this period range may include the periods of higher modes of vibration as well as longer periods that are seen to affect a nonlinear structure whose first-mode period has effectively lengthened (Baker 2011).

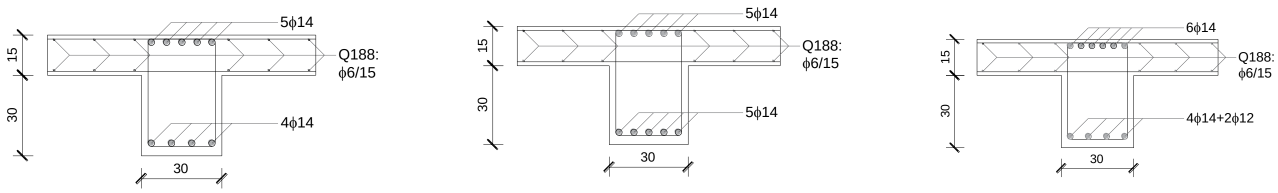
Selection of recorded ground motions are in most cases based on 1) geophysical parameters magnitude (*M*), source-to-site distance (*R*), and epsilon (ε); 2) ground motion intensity measures; and 3) spectral matching which will be the main focus in this paper. A brief review of previous investigation regarding selection of recorded ground accelerations based on spectral matching for different target spectrum EC8 spectrum, Uniform Hazard Spectrum (UHS) and Conditional Mean Spectrum (CMS) will be presented. Furthermore, seismic analysis will be performed for four-story reinforced concrete frame structure designed according to EC8 - ISPRA frame by selecting 30 different earthquake scenarios according to spectral matching in order to evaluate average maximum inter-story drift ratio. The aim is to compare obtained results from seismic analysis for each earthquake scenario in order to observe the difference of performance levels achieved by different target spectrums.



(a) Cross section of ISPRA frame (b) Floor plan of ISPRA frame



(c) Typical reinforcement in beams in first storey



(d) Location b_1 (e) Location b_2 (f) Location b_3

Fig. 1 Construction details of experimental model ISPRA frame (Dolšek 2008)

2. Case study

2.1 Experimental model description

The experimental full scale model on which seismic analysis is performed is presented in Fig. 1. It is a four-storey frame structure designed according to the previous version of Eurocode 2 (2013), and Eurocode 8 (2011). The structure has a total height of 12.5 m, with each story having a height of 3 m, except for the bottom story which has a height of 3.5 m. The structure has three frames and thus two bays in both *X* and *Y* direction. Raster of bays in *X* direction are 4 and 6 m, and raster of both bays in *Y* direction is 5 m in the direction in which the load was applied. Both columns and beams have a rectangular cross section, whereas columns' dimensions are 40/40 cm, except for column D which is 45/45 cm, and beams' dimensions are 30 cm width and 45 cm height. Thickness of the slab is 15 cm. Materials used in construction were concrete C25/30 and the B500 Tempcore reinforcing steel with characteristic yield strength of 500 MPa.

2.2 Numerical model description

Nonlinear analysis of the ISPRA numerical model in Fig. 2 was conducted by SeismoStruct (2016). The behavior of concrete elements was simulated with force-based plastic hinge (FBPH) elements with plastic hinges at the ends of the elements using Mander's model of confined concrete proposed by Mander *et al*. (1988) and the Menegotto-Pinto model (Menegotto and Pinto 1973) for reinforcing steel.

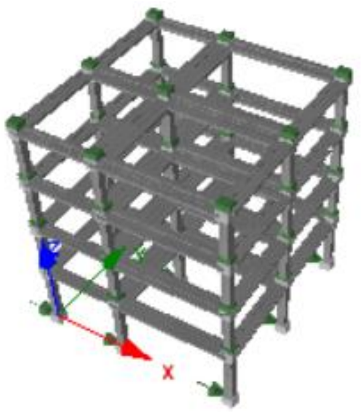
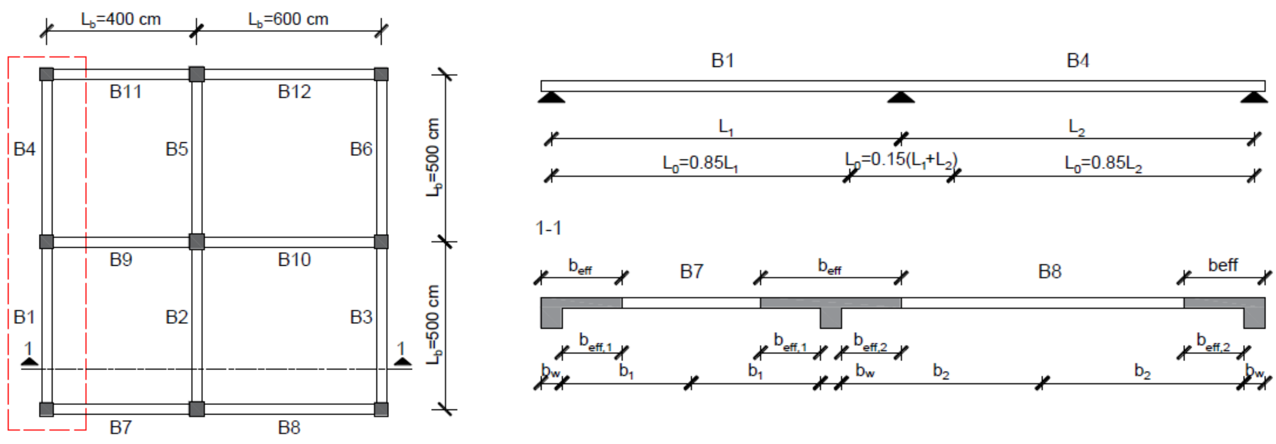


Fig. 2 Numerical model in SeismoStruct 2016 (SeismoStruct 2016)

Since reinforced concrete beams and the slab were constructed at the same time, they act as a monolithic section. Therefore, in the seismic assessment a contribution of slab to the stiffness and strength of beam was considered. In the Eurocode 8 (2011) is prescribed that slab reinforcement parallel to the beam and within the effective



(a) Layout of the beams in floor plan (b) Zero moment points and beam's effective width labels

Fig. 3 Definition of the beam's effective width

Table 1 The effective width of beams considered in analysis

Beam	Shape of beam	b_w $[cm]$	$b_{eff,1}$ $[cm]$	$b_{eff,2}$ $[cm]$	b_{eff} $[cm]$
B1, B3, B4, B6	L	30	50	-	80
B2, B5	T	30	50	50	130
B7, B11	L	30	40	-	70
B8, B12	L	30	60	-	90
B9	T	30	40	40	110
B10	T	30	60	60	150

Table 2 Values of concentrated load masses

Column	Storey 1	Storey 2	Storey 3	Storey 4
	m [t]	m [t]	m [t]	m [t]
1, 7	7.61	7.51	7.51	6.90
2, 8	11.23	11.13	11.13	10.52
3, 9	5.80	5.70	5.70	5.09
4	11.40	11.30	11.30	10.69
5	19.77	19.64	19.64	18.87
6	9.59	9.49	9.49	8.88
∑	90.07	89.12	89.12	83.46

flange width should be assumed in the design process to contribute to the beam flexural capacities, if it is anchored beyond the beam section at the face of the joint. In the Eurocode 2 (2013) are suggested values of effective width of beams for all limit states and are based on the distance l_0 between points of zero moments. It should be calculated according to next equations

$$b_{eff} = \sum b_{eff,i} + b_w \leq b \tag{1}$$

$$b_{eff,i} = min\begin{cases} 0.2 \cdot b_i + 0.1 \cdot l_0 \\ 0.2 \cdot l_0 \end{cases} \tag{2}$$

$$b_{eff,i} \leq b_i \tag{3}$$

where b_{eff} is the beam's effective width, b_w is the width of the beam, b_i is the one half of the distance between the beams and l_0 is the distance between the zero moment points. In the seismic analysis the distance b_i should be taken as $l_b/2$. In Fig. 3 is shown example for calculating effective widths in section 1-1. To calculate the, transversal beam spans should be taken as l_b (marked with red line in Fig. 3(a)), and in Table 1. are given calculated effective widths of beams considered in analysis.

Except effective width of beams, critical region length, or plastic hinges length were determined according to Eurocode 8 (2011). As structure was designed for ductility class high (DCH) and the behavior factor q=5, critical region length for beams were determined according to equation Eq. (4), and for columns according to Eq. (5)

$$L_p = 1.5\ h_w \tag{4}$$

Table 3 Comparison of natural periods for experimental and numerical model

Mode	Experiment	Numerical model	Error (%)
1	0.560	0.592	5.7
2	0.195	0.213	8.5
3	0.115	0.123	6.9

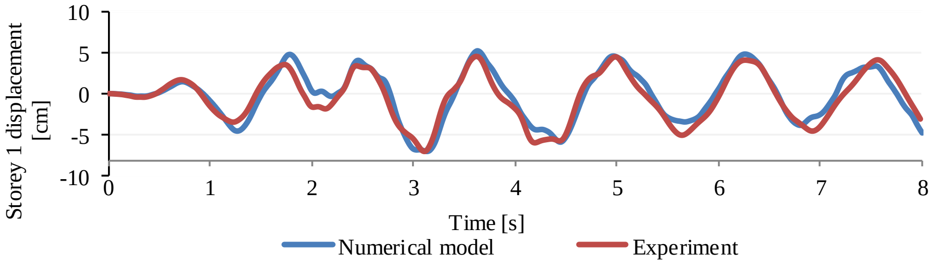
$$L_p = max\begin{cases} 1.5\ h_c \\ 1.5\ b_c \\ 0.6 \\ L_c/6 \end{cases} \tag{5}$$

Values of concentrated masses were obtained from the self-weight of the structure, permanent load of 2 kN/m^2, which presented floor finishing and partitions, and live load also of 2 kN/m^2. Concentrated masses as point loads on each column and for every storey are presented in Table 2.

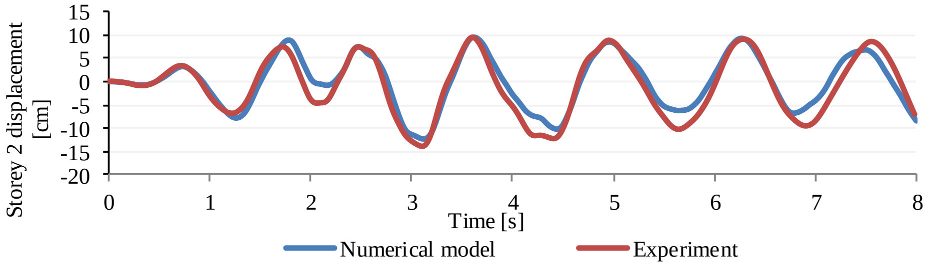
The accuracy of numerical model compared to the experimental one is firstly obtained by the comparison of natural periods for experimental and numerical model (Table 3).

The other check of compatibility of numerical to experimental model are resulted relative errors and correlation of values obtained from numerical modelling and experimental test for displacement, and base shear. In Fig. 4 and Fig. 5 are shown comparisons of displacements and base shear for numerical and nonlinear models.

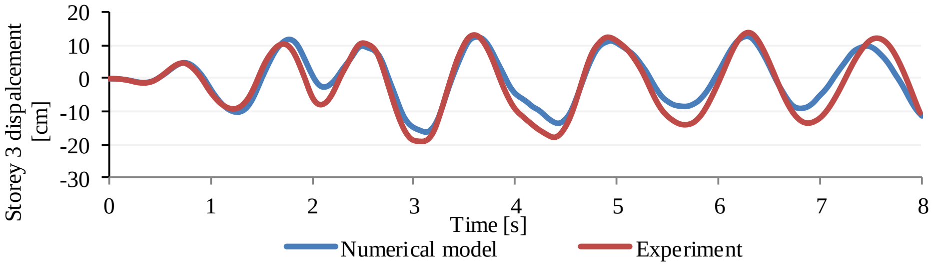
From the Table 4 it can be seen that the nonlinear numerical model also gave excellent results in terms of compatibility with hysteretic curves presented in Fig. 6. The



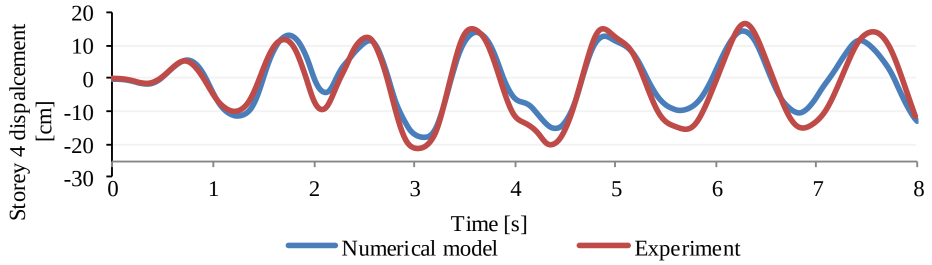
(a) 1st storey



(b) 2nd storey



(c) 3rd storey



(d) 4th storey

Fig. 4 Comparison of displacement results obtained from numerical and experimental model for every storey

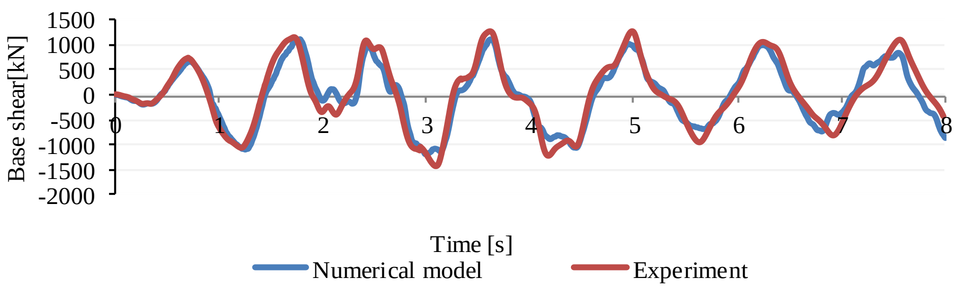


Fig. 5 Comparison of base shear results obtained from numerical and experimental model

mean relative error of the observed variables (displacement, base shear) was 15.92%, while the correlation R, given with Eq. (6)

$$R = \frac{n\sum xy - (\sum x)\cdot(\sum y)}{\sqrt{n(\sum x^2) - (\sum x^2)}\cdot\sqrt{n(\sum y^2) - (\sum y^2)}} \quad (6)$$

where x is experimental obtained values, y numerically obtained values and n is the number of pairs of data, was an excellent 0.95. That proves the compliance and applicability of the calibrated numerical nonlinear model for the parametric analysis of a model building with infilled frames according to expected behaviour.

Table 4 Mean relative error [%] and correlation R

Four-storey building ISPRA		Mean relative error [%]/correlation R
Displacement	1st floor	14.94/0.94
	2nd floor	11.11/0.95
	3rd floor	15.21/0.95
	4th floor	16.11/0.95
Base shear		17.24/0.95
Mean values		14.92/0.95

In order to make hysteretic curve clearer, in Table 5 is presented comparison of base shear and energy absorption capacity in 3rd and 5th cycle of hysteretic curve presented in Fig. 6 (b)-(c). In brackets are deviation values of numerical model results in regards to experiment results.

3. Selection of different earthquake scenario

Spectral matching is the most commonly proposed earthquake record selection method by seismic codes and, as such, can be utilized in the framework of both force-based and performance based design (Katsanos *et al*. 2010). In the past 15 years, the use of spectrum matched records has become increasingly widespread for the estimation of nonlinear structural response. Spectrum matched records are artificially generated time histories of ground motion acceleration, or other relevant parameter, whose response spectral shapes are matched to a predetermined target spectrum and used as input to dynamic analysis (Carballo

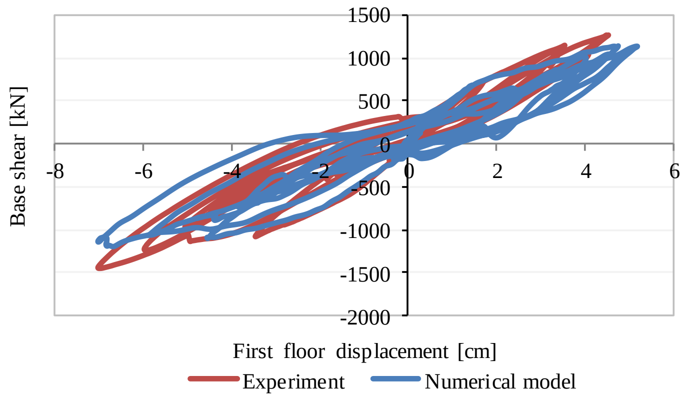
Table 5 Comparison of base shear capacity and energy absorption in 3rd and 5th cycle of hysteretic curve

Response cycle	3rd			5th		
	Experiment	Numerical model	+/-	Experiment	Numerical model	+/-
Base shear capacity [kN]	1266	1123	(-11,3%)	1269	1143	(-9,9%)
Energy absorption [J]	43900	50700	(+15,48%)	21600	24500	(+13,4%)

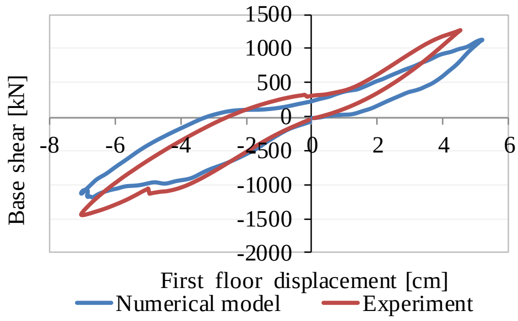
and Cornell 2000).

While selecting ground motion records, main aim is to select representative records of the ground motion at the site of interest and at the consistent source-to site distance. Also there will generally be a requirement to ensure that the records of ground motion conform to some specified level of agreement with the ordinates of the design response spectrum, whether records are selected by performing searches in terms of response spectral ordinates or in terms of seismological and geophysical parameters (Bommer and Acevedo 2004). If structural response is to be estimated by selecting ground motions to match a target response spectrum, typical response spectrum associated with the specified large amplitude Sa value at a single period must be found (Whittaker *et al*. 2011). For graphical explanation of earthquake scenario selection method, flow chart in Fig. 7 is presented.

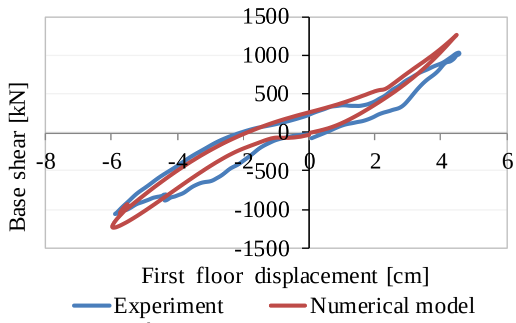
3.1 General data of used spectrums



(a) Comparison of experimentally and numerically obtained hysteretic curves for ISPRA model



(b) 3rd cycle of hysteretic curve



(c) 5th cycle of hysteretic curve

Fig. 6 Hysteretic curve

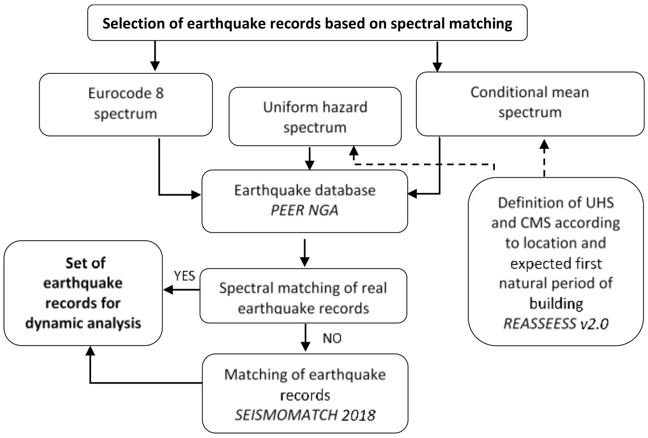
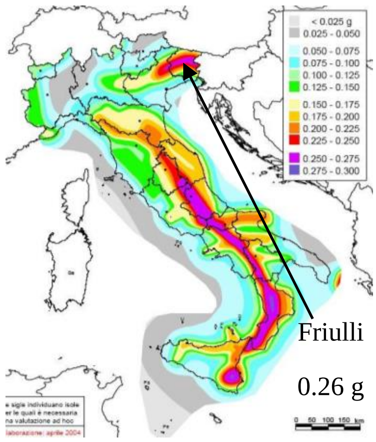
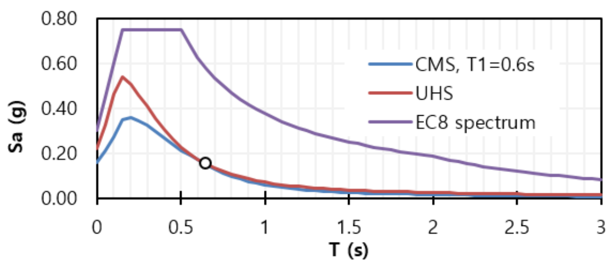


Fig. 7 Flow chart of earthquake selection method



(a) Seismic hazard map for Italy (Ghobarah 2004)



(b) Conditional Mean Spectrum (CMS), Uniform Hazard Spectrum (UHS) and EC8 spectrum selected as target spectrums

Fig. 8 Data of used spectrum

Based on experimental model, earthquake record from location Friulli with longitude: 13-103365 and latitude: 46.225918 shown in Fig. 8(a) was chosen for obtaining general data of used spectrums. To define Uniform Hazard Spectrum (UHS) and Conditional Mean Spectrum (CMS) according to location data and first natural period of used building, Reasseess V2.0 (Chioccarelli *et al*. 2018) was used. Thus, for CMS selected period is 0.6 s according to measured and numerically obtained first natural period of examined ISPRA building given in Table 3.

For definition of Eurocode 8 (2011) spectrum, elastic spectrum type 2 was used with expected PGA according to Hazard map for Italy shown in Fig. 8(a), while all three target spectrums are shown in Fig. 8(b).

Table 6 Evaluation of selection of records based on real and scaled records according to performance measures

Spectrum	Type of records	Mean absolute error (MAE)	Root mean squared error (RMSE)	Mean absolute percentage error (MAPE)
EC8	unscaled	0.24	0.49	55.31
	scaled	0.04	0.16	11.38
UHS	unscaled	0.09	0.27	44.67
	scaled	0.03	0.19	25.99
CMS	unscaled	0.03	0.15	70.84

Table 7 Evaluation of performance measures for matched spectrums in respect to target spectrums

Spectrum	Type of records	Mean absolute error (MAE)	Root mean squared error (RMSE)	Mean absolute percentage error (MAPE)
EC8		0.03	0.11	9.21
UHS	matched	0.03	0.18	21.26
CMS		0.02	0.12	15.11

3.2 Earthquake records set selection

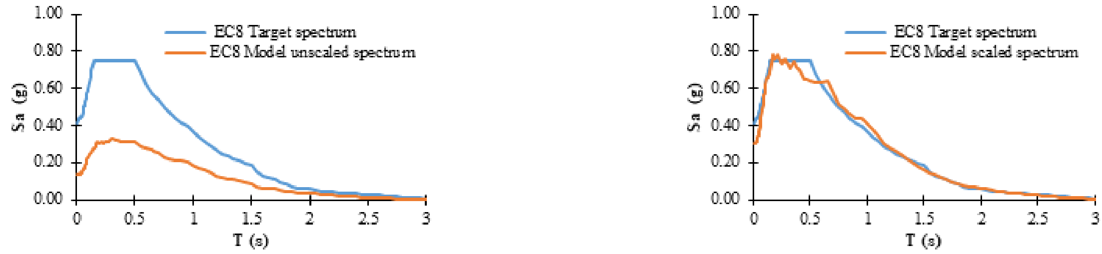
Set of ground motion records was selected from PEER Ground Motions Database (The new NGA West 2 database). In Fig. 9 comparison of EC8, UHS, and CMS target spectrum with model spectrum obtained from PEER database is presented. Model spectrum is the average spectrum among selected 30 earthquake spectrums.

Unscaled model spectrums are shown on the left hand sides, while scaled model spectrums are presented on the right hand sides of Fig. 9 (a)-(c). Some of performance measures as Mean Absolute Error (MAE), Root Mean Squared Error (RMSE), and Mean Absolute Percentage Error (MAPE) are evaluated in Table 6**,** from which it can be seen that scaling model spectrums to target spectrums shows better results of performance measures.

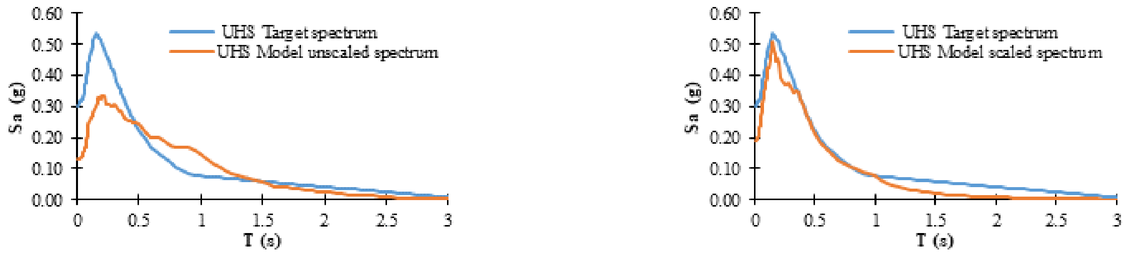
However, comparing original spectrum obtained from real accelerogram and target spectrum obtained from PEER NGA database as shown on subfigures on left hand side in Fig. 10, great differences were observed and spectrums based on real unscaled earthquake records could not be used. Thus, real unscaled individual accelerograms were matched to individual target spectrums in SeismoMatch 2016 to obtain mean target spectrum. Total of 90 earthquake records have been chosen (30 spectrums for 30 records for every target spectrum). Subfigures on Fig. 10 from the right hand side are presenting original and matched accelerogram.

Difference between the target and matched spectrum consequently have caused the change in values of performance measures Mean Absolute Error (MAE), Root Mean Squared Error (RMSE), and Mean Absolute Percentage Error (MAPE), and final shapes of evaluated spectrums Fig. 11. New, finally obtained values of performance measures are presented in Table 7.

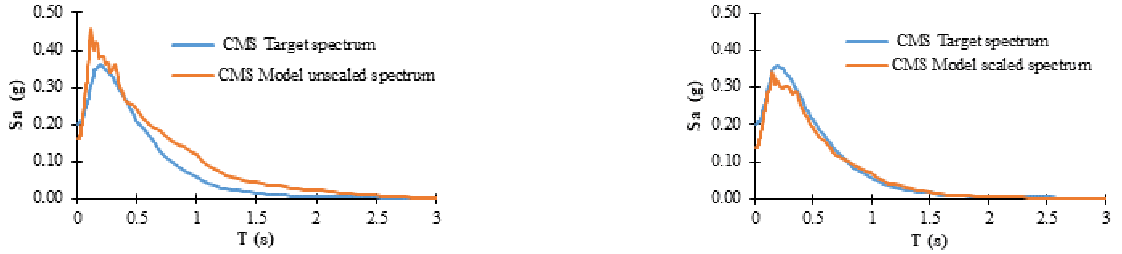
4. Results and discussion



(a) EC8 spectrum

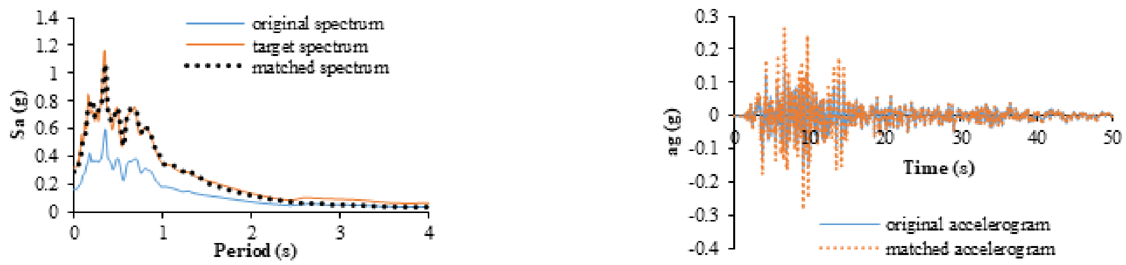


(b) UHS spectrum

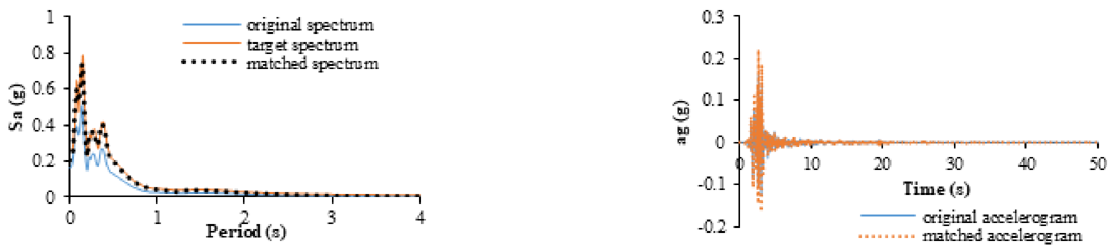


(c) CMS spectrum

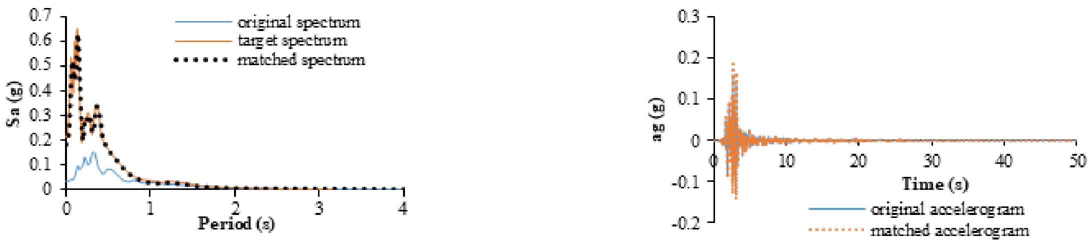
Fig. 9 Comparison between spectrum obtained by PEER NGA database (*PEER Ground Motion Database - PEER Center*) for real (unscaled) spectrum and scaled or matched



(a) EC8 spectrum

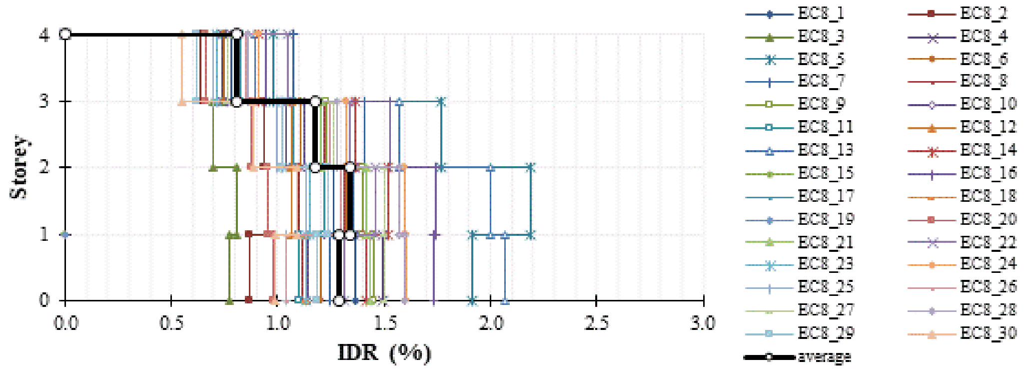


(b) UHS spectrum

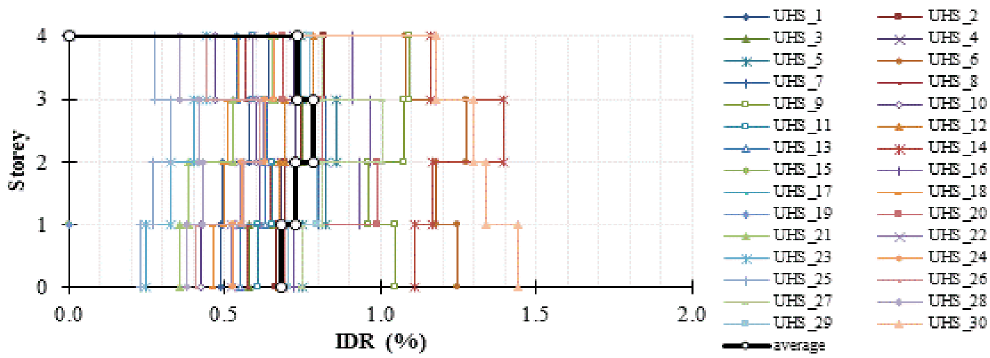


(c) CMS spectrum

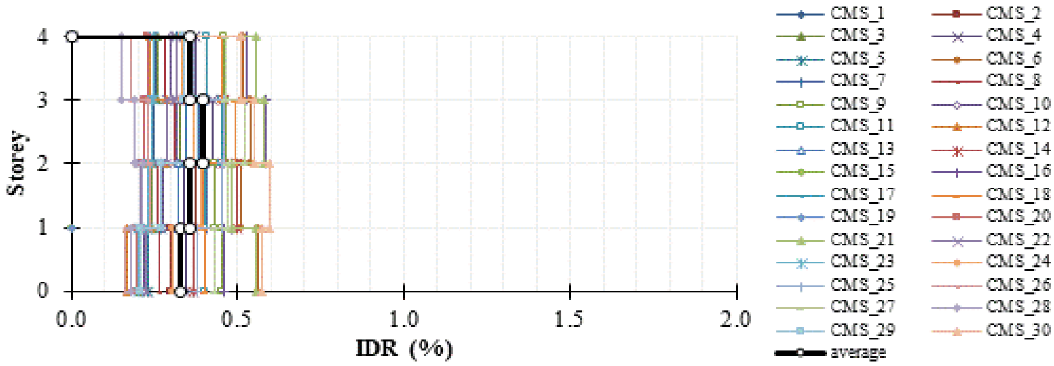
Fig. 10 Example of spectral matching of earthquake records with target spectrums



(a) EC8 target spectrum



(b) UHS target spectrum



(c) CMS target spectrum

Fig. 12 IDR profiles for every target spectrum

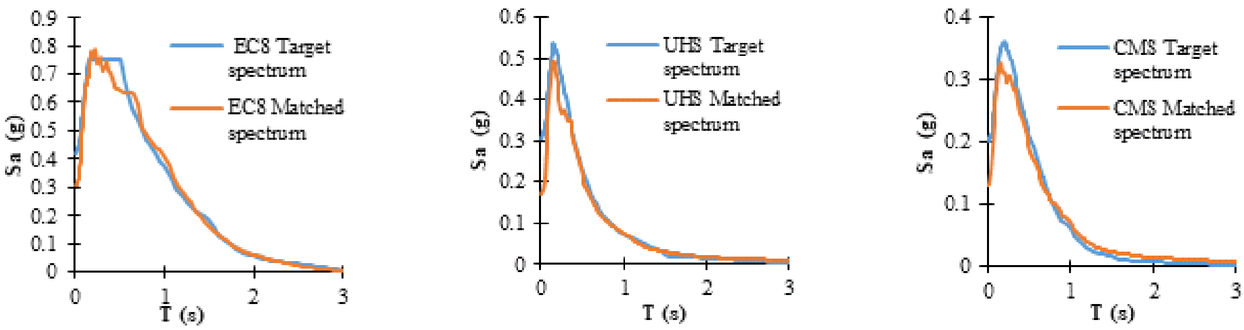
From seismic analysis are obtained results regarding Inter-storey Drift Ratio (IDR) of four-storey ISPRA building for all earthquake records and each target spectrum. These results are presented in Fig. 12 and it can be seen that for each target spectrum and belonging 30 matched earthquake records, average IDR is obtained.

Performance levels of structures under the earthquake loads describes the damage condition that must be satisfactory for a given building and a given ground motion. They are presenting the physical damage of buildings that may occur during the earthquake event. Damage for every structural performance level is defined by value of Inter-storey Drift Ratio (IDR) by Ghobarah (2004) and description of damage condition needed to understand the physical state of the building for the end user.

Possible performance levels are: slight damage= immediate occupancy; moderate damage=damage control; extensive damage=life safety; near collapse=collapse prevention according to (HAZUS-MH). In Table 8 is presented comparison of IDR according to structural performance levels and corresponding structure type for (Ghobarah 2004), (FEMA 356 2000), and (SEAOC 1995).

Table 8 Comparison of IDR (%) according to structural performance levels and structure type

	Structural performance level	RC frames IDR (%) (Ghobarah 2004)	(FEMA 356 2000)	(SEAOC 1995)
	Slight damage	IDR<0.20	IDR<1.0	IDR<0.50
	Moderate damage	0.20<IDR <1.0	1.0<IDR <2.0	0.50<IDR <1.50
	Extensive damage	1.0<IDR <3.0	2.0<IDR <4.0	1.50< IDR <2.50
	Near collapse	IDR>3.0	IDR>4.0	IDR>2.50



(a) EC8 spectrum (b) UHS spectrum (c) CMS spectrum

Fig. 11 Example of spectral matching of earthquake records

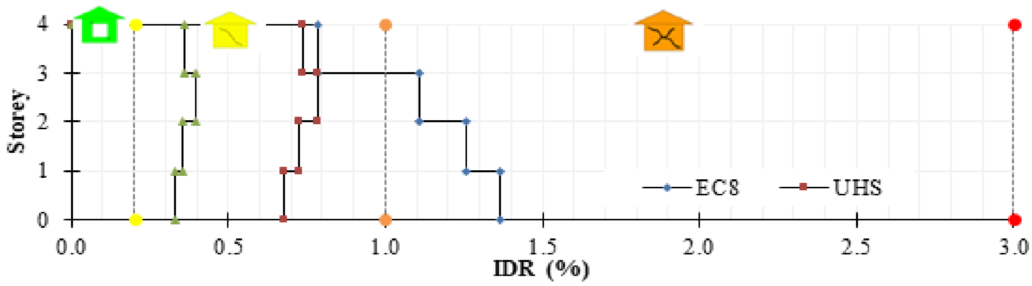


Fig. 13 Mean IDR profiles for three spectrum in accordance with performance levels

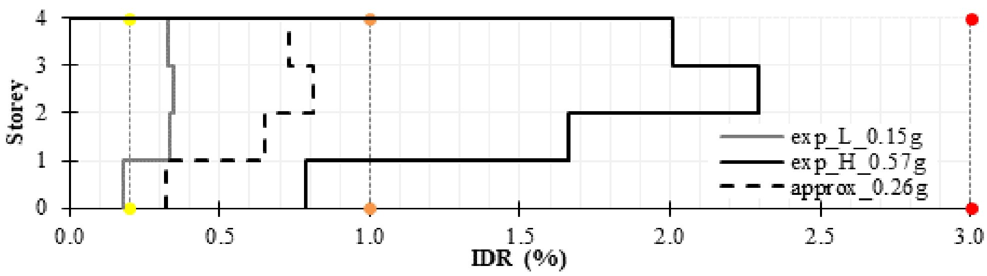


Fig. 14 Mean IDR profiles for experimental results of low and high test and approximation of possible damage for 0.26 g

Table 9 Performance objectives for buildings (SEAOC, 1995)

Earthquake design level (probability of exceedance)	Structural performance levels			
	Immediate occupancy	Damage control	Life safety	Collapse prevention
Frequent (50% PE in 30 years)				
Occasional (50% PE in 50 years)				
Rare (10% PE in 50 years)				
Very rare (2% PE in 50 years)				

Unacceptable performance	Basic objective	Essential service objective	Safety critical objective

Structural performance levels are also related to different earthquake return periods in terms of maximum IDR as shown in Table 9 according to Vision 2000 (SEAOC 1995).

However, in this paper obtained IDR are compared with performance levels by Ghobarah (2004). In order to make an evaluation of obtained results in regard to defined performance levels and possible damage, mean IDR profiles from Fig. 12 are summarized in Fig. 13 with damage level limits. According to the results, it can be seen that IDR obtained for UHS and CMS target spectrums are within the same damage limits.

Experimental results of low level (0.15 g) and high level (0.57 g) test conducted on ISPRA building (HAZUS-MH) considering IDR are shown in Fig. 14. In order to be able to compare results fro the experiment and the ones obtained from seimic analysis, IDR is obtained by interpolation for PGA of 0.26 g, which is the PGA of observed location Friulli. This IDR is further compared on Fig. 15 with the ones obtained earlier from seismic analysis for EC8, UHS and CMS target spectrums and it can be seen that the best agreement was with the UHS target spectrum.

4. Conclusions

The selection of appropriate seismic records is the one of the most critical steps while performing the nonlinear dynamic time history analysis in order to estimate seismic performance in the basis of the hazard at the site where the

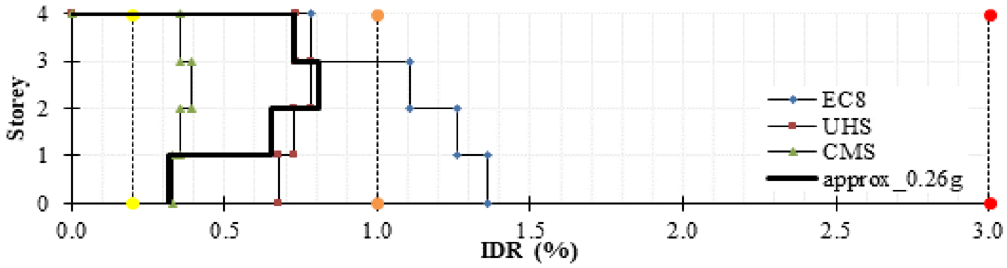


Fig. 15 Comparison between approximated results for 0.2 6 g and observed results from three spectrum

structure is located. There are different ways of selecting ground motions, however in this paper 90 ground motion records are selected based on spectral matching, 30 for each of three target spectrums, EC8 spectrum, Uniform Hazard Spectrum (UHS), and Conditional Mean Spectrum (CMS).

Seismic analysis was performed on numerical model of based on experimentally tested ISPRA building. Accuracy of numerical model is obtained and approved by performance measures and comparison of displacements, base shear and hysteretic curve. Time history analysis for 90 earthquake records were obtained. As a result, Inter-story Drift Ratios (IDR) as a main damage measure were presented in order to make comparison with defined performance levels of reinforced concrete bare frames. There damage levels are defined with four limiting values of IDR. All mean IDRs for every target spectrum were mutually compared.

It is observed that UHS and CMS target spectrum give IDRs within the same limits of performance levels, in this case moderate damage. However, according to EC8 this IDR is within the limits that describe extensive damage, that is a proof that code recommendation is over conservative.

Comparing these results with the ones from interpolation based on low and high test of ISPRA experimental model, it was concluded that IDR obtained from experimental model fits the best to the UHS target spectrum. Final conclusion is that EC8 spectrum is over conservative that can be good for design reserve but in that case it is not efficient. However, conditional mean spectrum is a new type of spectrum that should be evaluated with additional numerical analyses.

Funding

This paper was supported by Croatian Science Foundation under the project UIP-2017-05-7113 Development of Reinforced Concrete Elements and Systems with Waste Tire Powder—ReCoTiP.

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