Scaling and Selection of Ground Motion Records for Nonlinear Response History Analysis of Structures

B.Ö. Ay & S. Akkar
Earthquake Engineering Research Center, Department of Civil Engineering, Middle East Technical University, Ankara, Turkey

SUMMARY:
Accurate estimation of structural response for a given target hazard level requires a suitable set of ground-motion records that represents a pre-defined scenario event. With the aim of assessing the seismic performance of structural systems, this study presents a ground motion selection and scaling methodology. Given a relatively larger ground-motion database, the method uses the differences between individual records and corresponding estimations of a ground-motion prediction equation in order to determine the optimum subset of recordings. This way, the procedure provides a group of modified records whose median elastic spectral ordinate matches with the target intensity level without excessively manipulating the inherent aleatory variability in the selected recordings. Considering the sensitivity of overall structural response to the nonlinear behavior, the procedure approximately estimates the median and dispersion of inelastic structural response. The case study suggests that, the methodology can be a useful tool for building performance assessment studies.

Keywords: ground-motion record selection and scaling, structural performance assessment

1. INTRODUCTION

Seismic response evaluation of structures for a given target scenario and corresponding intensity level can be accomplished by using either nonlinear static procedures or response history analyses. Among these approaches, nonlinear response history analysis is more desirable than before due to increased availability of ground-motion records and computer capabilities. Besides, using such detailed analyses are sometimes indispensable in structural engineering applications for evaluating the demand of ground motion excitations on critical structures. However, nonlinear response history analysis of structures involves still some issues related with assembling a suitable set of accelerograms that can properly represent the desired seismic level. The selected accelerograms should also result in unbiased structural response.

This paper presents a ground motion selection and scaling methodology in order to assess the structural response accurately without manipulating the inherent aleatory variability in the selected recordings excessively.

2. GROUND-MOTION RECORD SELECTION AND SCALING METHODOLOGY

Selection and amplitude scaling of real records to be used in response history analysis is desired and a practical way of assembling an input dataset because various concerns raised for synthetic accelerograms or frequency modification of strong-motion data do not come into the picture. Hence, this study focuses on the selection and amplitude scaling of real accelerograms in order to estimate the structural response accurately for a given target hazard level.

The methodology investigated in this paper constrains the scaling to the differences between each individual record and corresponding estimation from a ground-motion prediction equation (GMPE) model. This way, the method emphasizes the significance of preserving the basic seismological features of the records after being scaled. The final selection of the recording set is accomplished by estimating the standard deviation of all combinations resulting from a relatively larger candidate accelerogram dataset. The details of the selection and scaling procedure are described below.

2.1. Candidate Ground-motion Record Dataset and Final Selection of Accelerograms

The identification of accelerograms to be used in the evaluation of structures is a critical issue as it directly influences both the median estimation and the dispersion about this median. The literature for selecting a pre-defined number of recordings within a ground-motion dataset is developing consistently due to the recent intention of engineering practice.

Despite the rapid increase of available accelerograms and relative ease of obtaining these provided by different online sources (e.g., http://kyh.deprem.gov.tr/ftpe.htm), the current resolution of the strong-motion databanks usually do not provide sufficient number of recordings that fully comply with the geophysical and strong-motion parameter constraints of scenario earthquake. This is particularly valid for large magnitude events for which the data distribution is sparse and relatively non-uniform. Consequently common approach in earthquake engineering community is to obtain a candidate accelerogram bin suitable with the scenario earthquake described by main geophysical parameters. Then, final selection and scaling of recordings are accomplished in order to have a ground-motion set that fully match with the target intensity level of the scenario earthquake.

Among geophysical parameters, earthquake magnitude is a primary search parameter for ground-motion records within databases. There is a general consensus among most of the record selection strategies as well as building codes (e.g., ATC-58 (ATC, 2009)) to select the recordings from a magnitude interval representative of scenario event. For instance, previous studies imply that the selection of records should be based on narrow magnitude intervals with an average value equal to the scenario event. This can be accomplished by selecting records with magnitudes closely matching with the target value. This criterion is quantified as 0.25 units and 0.20 units at either side of the target magnitude by Stewart et al. (2001) and Bommer and Acevedo (2004), respectively. Besides, source-to-site distance is also used as a constraint in database quarries although it is widely accepted that, this parameter is less significant with respect to magnitude. Finally, the influence of style-of-faulting and soil type in response history analyses of structures may also be taken into consideration in record selection strategies. Recent studies promote the consideration of spectral shape instead of these geophysical parameters while assembling candidate recording dataset. However, main geophysical parameters are still primary query parameters in earthquake engineering community while selecting the candidate accelerograms. Consequently, this study assembles the candidate strong-motion dataset by considering the main geophysical parameters: candidate ground-motion records are selected from ±0.25 units of magnitude and ±25 km distance range by considering the magnitude and distance values imposed by the target scenario.

The importance of style-of-faulting as well as the site conditions on ground-motion amplitudes are also emphasized while selecting the candidate accelerograms. However, since the number of accelerograms that comply with all of these criteria is limited, the style-of-faulting and soil class criteria are partially relaxed. Since this study focuses on the site classes that are frequently encountered in the urban settlements, we constrained the choice of candidate ground motions from the two neighboring site classes; stiff and soft soil categories whose V_{S30} values range between 180 m/s < $V_{S30} \leq$ 360 m/s (NEHRP D) and 360 m/s < $V_{S30} \leq$ 760 m/s (NEHRP C). In any case, the scaling methodology still preserves the pertinent style-of-faulting and site class features of each candidate accelerogram inherently through the use of ground-motion prediction equations as discussed in the next section.

There are various studies in literature that proposed methods in order to select required number the accelerograms, n, within a candidate ground-motion bin that contains “k” accelerograms (Baker and

Cornell 2006; Luco and Bazzurro 2007; Baker, 2011). In essence, the required number of records, n, may depend on the codes, analysis methods, the objective of the response history analysis, etc. For instance, several codes as well as the Turkish earthquake code (TEC, 2007), require selecting 7 records (n=7) for median prediction of structural response. Alternatively, ATC-58 (2009) force the analyst to choose at least 11 accelerograms (n=11) for response history analyses in order to determine the full probability distribution of response. As a compromise among the various studies in the literature, this study sets the number of accelerograms used in response history analyses to 10 (n=10).

Another important subject is the total number of recordings in candidate bin (k). As indicated before, there are some practical constraints on the number of candidate accelerograms because it may turn out to be a difficult task to identify sufficient number of candidate accelerograms for site specific hazard scenarios when the analyst is opted to find reliable records in terms of metadata parameters (i.e., magnitude, source-to-site distance, fault mechanism, site class, etc.). Even if the analyst can afford to find large number of candidate records that comply with the constraints imposed by the scenario earthquake, decision on the optimum number is challenging because a small increase in "k" may result in a significant increase in the computation time (because the number of combinations increases) without gaining too much on the overall success of the selection procedure. This subject will be examined further below.

Selection of n records among k candidate accelerograms that are determined according to a scenario event corresponds to a combination problem. The number of possible combinations of records can be calculated by Eqn. 2.1. Hence, the number of possible combinations assembled for response history analyses is equal to C(k,n). So, the analyst should describe the selection criteria in order to define the optimum set among these possible combinations. The optimum set can be defined as the group that best matches the target intensity level with zero variance or the group that matches the target response spectrum mean and variance. The former criterion may indicate that the ground-motion set is useful if the estimation of median structural response is of concern. The latter approach is suitable for the probabilistic methods that assess the engineering demand. The presented method aims to reduce the dispersion about the median structural response as well as the accurate estimation of this median response (Ay and Akkar, 2012). Consequently, the method specifies the optimum recording ensemble as the one yielding the least standard deviation about the target.

$$C(k,n) = \binom{k}{n} = \frac{k!}{n! \cdot (k-n)!} \tag{2.1}$$

To examine the number of recordings in candidate set, this study carried out some basic statistical analyses. The relationship between the number of candidate recordings and the computational time for selecting and scaling the optimum recording set for elastic and inelastic structural analysis is investigated. For the presented methodology, the computational time to select and scale n optimum recordings from k candidate accelerograms depends on the number of possible combinations, C(k,n). Fig. 2.1 shows the total elapsed time to select and scale the optimum recordings as a function of C(k,n). In this example, n is kept as 10 but k is varied between 14 and 24. When k = 14, (C(k,n)=1001), the required time for selecting and scaling the optimum dataset is less than 1 second. On the other hand, if k = 22, which is a little bit over k = 20 (used in this study), the elapsed time reaches to 820 seconds. We note that when k = 20, the computational time is approximately 90 seconds on an ordinary PC.

Of course, the decision on the number of candidate recordings, k, cannot be given by the mere consideration of the computational time. The reduction in dispersion about the median target hazard level for the "n" selected and scaled recordings is the other factor that should be accounted for during this decision process. The left panel in Fig. 2.2 shows the variation of dispersion about the median elastic target hazard as a function of k when n = 10. The plot demonstrates that the reduction in dispersion flattens when k is 20 and the increase in k after this level does not introduce any significant improvement in the reduction of dispersion. Considering the observations on the computational time, choosing k as 20 seems to be the most favorable decision for the efficiency and accuracy of the presented procedure. This decision is re-assessed for nonlinear systems. The right panel of Fig. 2.2 presents the variation of standard deviation as a function of k for inelastic spectral ordinates for R = 4 (strength reduction factor, R, is the ratio of elastic to yield strength). The reduction in the standard deviation becomes stable after k = 16, which advocates that the choice of k = 20 would generally warrant the most efficient results for our procedure.

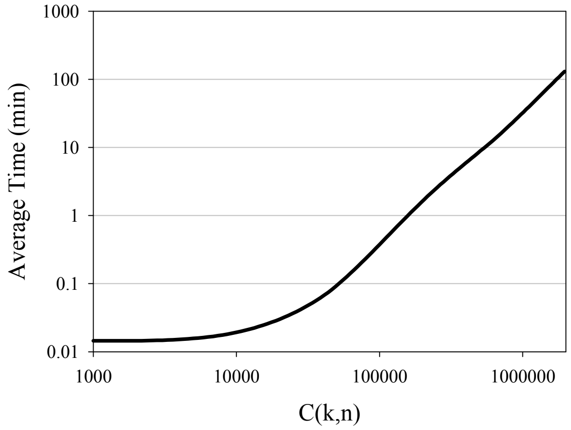

Figure 2.1. The increase in the computational time of the proposed procedure as a function of possible number of combinations

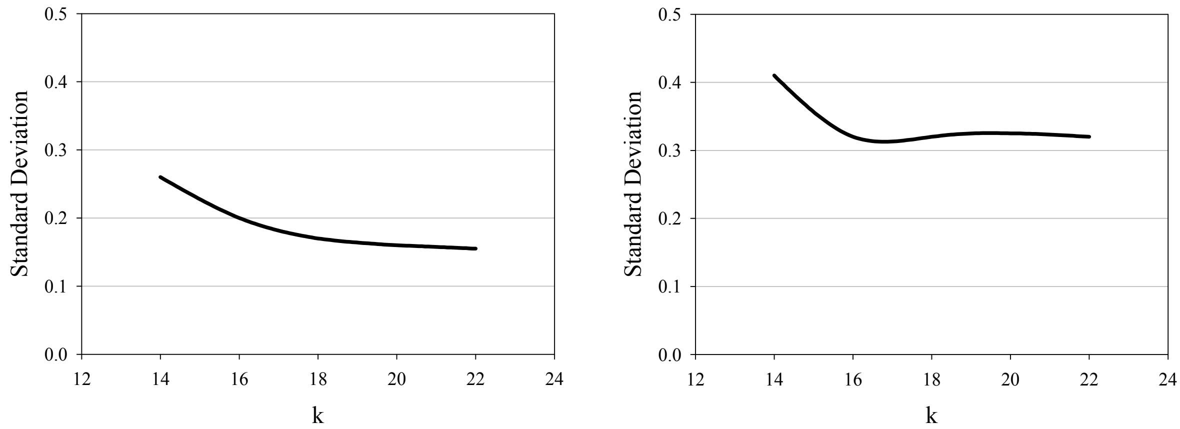

Figure 2.2. Variation of dispersion as a function of k for elastic systems (left panel) and inelastic systems (right panel)

2.2. Scaling Methodology

The scaling methodology linearly scales the records in acceleration domain in order to assure a target spectral level while it principally aims to preserve the inherent uncertainty in the selected recordings. To achieve this goal, the procedure constrains the modification of the records to the difference between the actual ground motion and corresponding GMPE estimation.

In order to reflect the aleatory variability inherent in the nature of earthquakes, the procedure constrains the scaling to a parameter ($\varepsilon\sigma_{Sdi}$) defined as the logarithmic difference between the spectral displacement of the record, $S_{d,i}$ (T), and the corresponding median ground motion estimation obtained from the selected GMPE ($\bar{S}_{d,i}$ (T)). (Although the entire procedure will be described over spectral displacement, any other spectral value such as PS_a or peak ground motion value; PGA, PGV, etc. can be used in this scaling procedure. This study uses spectral displacement as it is a common ground-motion intensity measure in engineering applications). Eqn. 2.2 describes the calculation of $\varepsilon\sigma_{Sdi}$.

$$\varepsilon\sigma_{Sdi} = \ln(S_{d,i}) - \ln(\bar{S}_{d,i}) \tag{2.2}$$

In previous section, the importance of style-of-faulting as well as the site conditions on ground-motion amplitudes is discussed. The value of $\varepsilon\sigma_{Sdi}$ implicitly considers these parameters through the median estimation of used ground-motion prediction model. Consequently, the scaling procedure preserves the pertinent soil class of each candidate recording as well as the rupture mechanism while computing $\varepsilon\sigma_{Sdi}$.

The scaling procedure linearly modifies each accelerogram to its individual target level instead of scaling all records to a common spectral value. To define individual target levels, of which mean value exactly matches with the target spectral displacement value ($\bar{S}_{dtarget}(T)$), the procedure makes use of θ, a parameter called as scaling origin. Eqn. 2.3 shows the calculation of θ.

$$\theta = \ln(\bar{S}_{dtarget}) - \ln\left(\frac{\sum_{i=1}^{n} \exp(\varepsilon\sigma_{Sdi})}{n}\right) \tag{2.3}$$

Finally, using the parameter θ for a set of n recordings, the procedure modifies each individual recording with the scaling factor, γ_i, that is given in Eqn. 2.4.

$$\gamma_i = \frac{S_{dtarget,i}(T)}{S_{d,i}(T)} = \frac{\exp(\theta + \varepsilon\sigma_{Sdi})}{S_{d,i}(T)} \tag{2.4}$$

As shown in Eqn. 2.4, $S_{dtarget,i}$ is a linear function of $\varepsilon\sigma_{Sdi}$ for elastic systems. In other words, the scaling procedure described above let the analyst to define the distribution of scaled ground motions about $\bar{S}_{dtarget}(T)$. Assuming log-normal distribution for spectral response, one can calculate the expected value ($\lambda_{Sdtarget}$) and standard deviation ($\zeta_{Sdtarget}$) of the scaled ground motions using Eqn. 2.5 and Eqn. 2.6, respectively. In these equations, $\mu_{\varepsilon\sigma}$ is the average of $\varepsilon\sigma_{Sdi}$ values of n recordings.

$$\lambda_{S_{dtarget}} = \theta + \frac{\sum_{i=1}^{n} \varepsilon\sigma_{Sdi}}{n} = \theta + \mu_{\varepsilon\sigma} \tag{2.5}$$

$$\zeta_{S_{dtarget}} = \sqrt{\frac{1}{n-1} \cdot \sum_{i=1}^{n} (\varepsilon\sigma_{Sdi} - \mu_{\varepsilon\sigma})^2} \tag{2.6}$$

The linear relationship established between individual target level and $\varepsilon\sigma_{Sdi}$ is strictly valid for systems behaving in elastic range and fails for inelastic levels. Since the correlation between inelastic response and $\varepsilon\sigma_{Sdi}$ decreases with increasing level of inelasticity, Ay and Akkar (2012) concluded that the optimum recording set that results in a reduced dispersion for a linear structural system is not necessarily the best choice when the system behaves in inelastic range. Thus, their study proposed a new parameter, $\varepsilon\sigma_{ISdi}$ that correlates better with inelastic response. A linear combination of PGV with a spectral ordinate is proposed as the estimators of $\varepsilon\sigma_{ISdi}$ through the functional forms presented in Eqn. 2.7. In Eqn. 2.7, the inelasticity level of a single degree of freedom (SDOF) system having a vibration period of T is represented with strength reduction factor, R.

$$\varepsilon\sigma_{ISdi} = c_1(R,T) \cdot \varepsilon\sigma_{Sdi} + c_2(R,T) \cdot \varepsilon\sigma_{PGV} + c_3(R,T) \tag{2.7 a}$$

$$c_1(R,T) = 1 - 0.72\ln(R) + 0.7T\ln(R) - 0.21T^2\ln(R) \tag{2.7 b}$$

$$c_2(R,T) = 0.81\ln(R) - 0.78T\ln(R) + 0.23T^2\ln(R) \tag{2.7 c}$$

$$c_3(R,T) = 0.22\ln(R) - 0.4T\ln(R) + 0.15T^2\ln(R) \tag{2.7 d}$$

Detailed information on the proposed scaling and selection procedure can be obtained in Ay and Akkar (2012).

3. CASE STUDY

The selection and scaling procedure presented here is verified by using a first-mode dominant multi degree of freedom (MDOF) system and the results are compared with the selection and scaling methodology that is based on Conditional Mean Spectrum (CMS) presented by Baker (2011). This section shows the comparison of these selection and scaling methodologies by considering the estimation of structural response.

3.1. Structural Model

Estimating earthquake induced hazard is a challenging subject that contains both structural and ground motion uncertainties. Besides the random nature of earthquake phenomena, variability in structural properties also plays an important role in performance assessment studies. In this section, a structural model that is developed by a statistical study investigating general characteristics of Turkish reinforced concrete (RC) building stock is used. Fig. 3.1 shows the three dimensional model and corresponding plan view of the RC moment resisting frame structure, a customized clone of a real building in Bakırköy, İstanbul. This building was designed by using Probina Orion (Prota, 2007) according to the codes and specifications used in Turkey for buildings located in Seismic Zone 1 that describes the most seismic prone in Turkey.

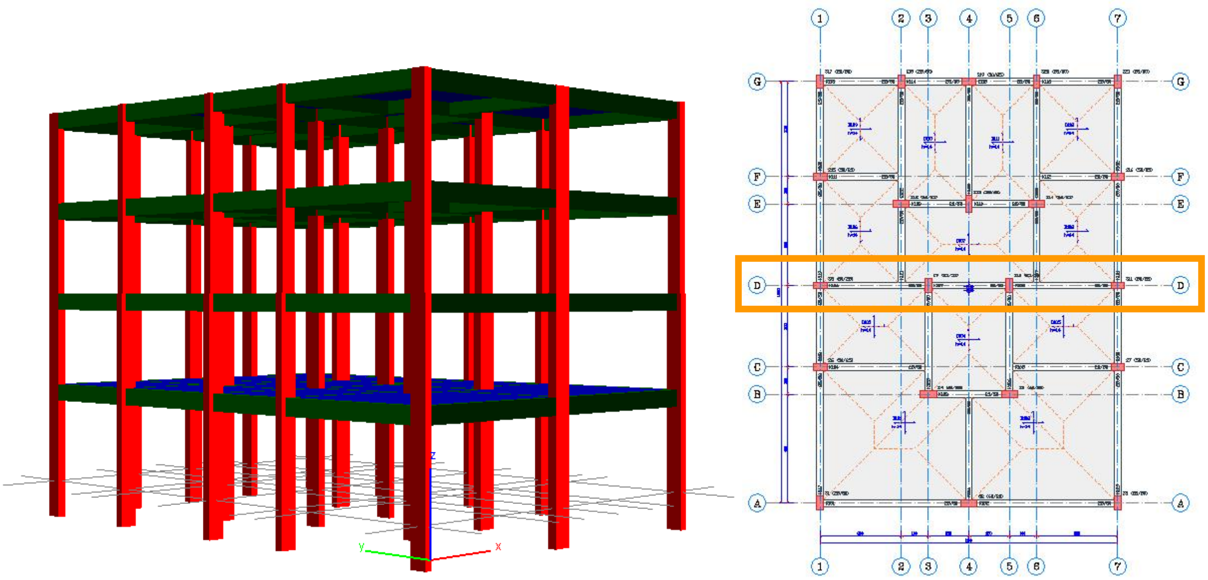

Figure 3.1. 3D model of 4 storey moment resisting frame structure and corresponding plan view

In order to estimate the seismic response of this structure, a representative two dimensional frame (The right panel of Fig. 3.1 shows the orientation of the selected frame as plan view) is employed. The rectangular column and beam dimensions of the four story, three bay moment resisting frame (4MRF-DD) are shown in Fig. 3.2. The structural analyses of this model are conducted by using SeismoStruct (Seismosoft, 2010) platform (version 5.2.2). Mander et al. (1998) nonlinear concrete model and bilinear steel model is used to represent nonlinear material behavior. The frame members are modeled as inelastic force-based fiber elements.

The eigenvalue analysis of 4MRF-DD frame yields the first and second mode period of the model as 0.61 seconds and 0.20 seconds, respectively. The corresponding modal mass percentages are obtained as 0.84 and 0.11. According to these results, the structure is assumed as first-mode dominant and pushover loading is applied accordingly. The left panel in Fig. 3.3 shows the pushover curve (base shear vs. roof displacement) of the 4MRF-DD frame and corresponding bilinear idealization using ATC-40 (ATC, 1996) procedure. The idealized pushover curve results in a postyield stiffness ratio equals to 3.03%. The right panel in Fig. 3.3 displays the conversion of idealized pushover curve to the spectral ordinates (ADRS) by using first-mode parameters of the system.

According to the ADRS conversion, the yield pseudo-spectral acceleration ($PS_{a,y}$) of the SDOF system is found as 369 cm/s^2, which corresponds to the spectral displacement ($S_{d,y}$) value of 3.53 cm.

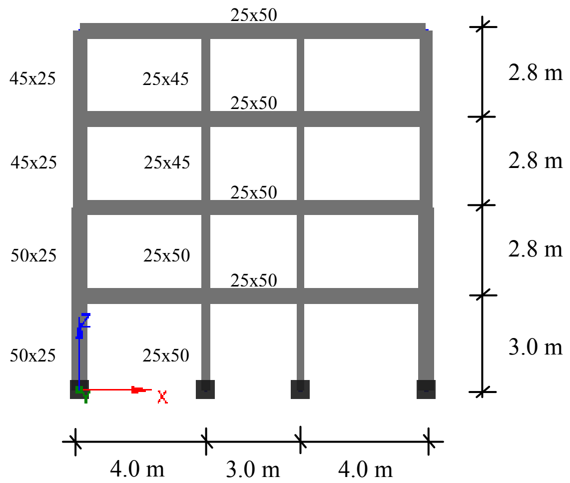

Figure 3.2. 2D model of 4 storey moment resisting frame (4MRF-DD)

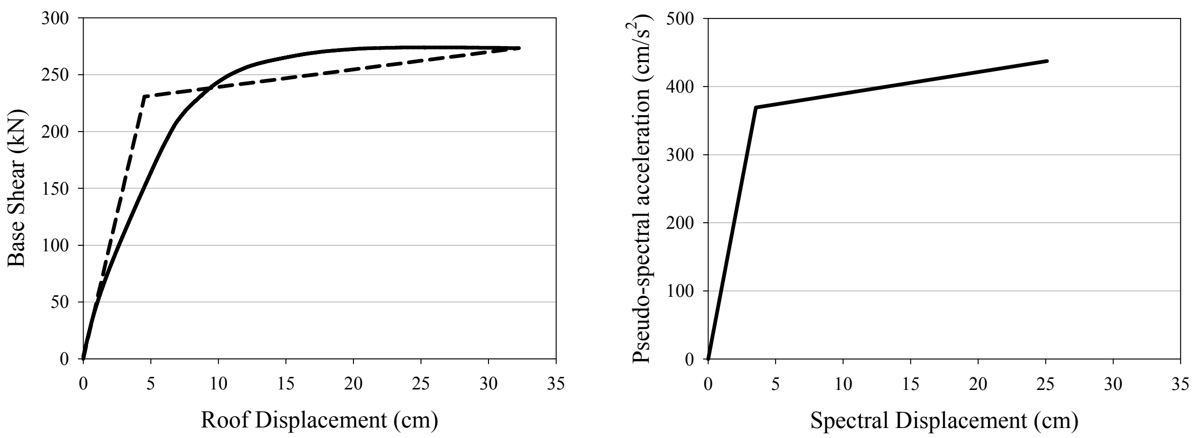

Figure 3.3. The pushover curve of the 4MRF-DD frame (left panel) and ADRS conversion (right panel)

3.2. Earthquake Scenario and Selection of Final Ground-Motion Ensemble

To examine the procedure, a target scenario determined from a probabilistic seismic hazard analysis that is conducted for the considered building (T_1=0.61 seconds) is used. The fault source that is likely to produce damaging future earthquakes in the area of interest is strike–slip. The considered building is located on a soft soil (NEHRP D). The deaggregation of the probabilistic seismic hazard analysis yielded $M_{w,target}$=6.25, $R_{JB,target}$ = 10.5 km and ε_{target} = 1.90. The deaggregation results correspond to an elastic target spectral displacement ordinate of 13.93 cm according to the Akkar and Bommer (2010) ground-motion prediction equation.

Based on the criteria discussed for selecting the candidate ground-motion data, 20 accelerograms are assembled from a magnitude range in the vicinity of target scenario magnitude (6.10 ≤ M_w ≤ 6.40). The distance range of selected candidate accelerograms is 0 km ≤ R_{JB} ≤ 25 km. The strength reduction factor, R, is estimated as 4 from the target spectral displacement value that is obtained from the deeaggregation of the hazard and ADRS curve. Among the recordings in candidate ground-motion dataset, a final ground-motion bin containing 10 accelerograms is selected. This is achieved by using the $\varepsilon\sigma_{ISdi}$ estimations through Eqn. 2.7 and corresponding standard deviation ($\zeta_{Sdtarget}$) estimations from all combinations of accelerogram subsets. The optimum recording set is determined as the one that yields the smallest estimated dispersion.

In order to compare the presented method, an alternative ground-motion set is assembled by using a procedure that is based on CMS. Conditional mean spectrum is a new spectral shape introduced by Baker (2011), for selecting and scaling the strong-motion recordings to estimate accurate median structural response. Table 3.1 shows the geophysical parameters of the records in candidate ground-motion set. The records that are selected by using the methodology investigated in this study and by CMS are also given in the last two columns of Table 3.1.

Table 3.1. Geophysical parameters of candidate accelerograms and final suits of recordings obtained from the presented method (Ay and Akkar, 2012) and CMS (Baker, 2011)

EQ NAME	M_W	R_{JB} (km)	SOIL TYPE	FAULT TYPE	Ay and Akkar (2012)	CMS (Baker, 2011)
ECDE7329	6.10	11.0	D	SS		X
PEER0033	6.19	16.0	C	SS		
PEER0095	6.24	3.5	D	SS	X	X
PEER0265	6.33	13.8	C	SS		X
PEER0266	6.33	18.5	D	SS		X
PEER0300	6.20	8.8	C	N		X
PEER0367	6.36	7.7	D	R	X	X
PEER0459	6.19	9.9	C	SS		
PEER0460	6.19	12.1	D	SS		
PEER0461	6.19	3.5	D	SS	X	X
PEER0548	6.19	21.6	D	SS		X
PEER0558	6.19	6.4	D	SS		X
PEER0718	6.22	17.6	D	SS	X	X
PEER1126	6.40	14.1	C	N		
PEER2628	6.20	0.0	C	R	X	
PEER3473	6.30	5.7	C	R	X	
PEER3475	6.30	0.0	C	R	X	
TGMB0001	6.10	6.4	D	N	X	
TGMB0120	6.40	0.0	D	N	X	
TGMB3183	6.30	2.2	C	SS	X	

3.2. Analyses and Results

Using the ground motion ensembles obtained from alternative selection and scaling methodologies, inelastic spectral analysis of the idealized SDOF system and nonlinear response history analysis of the considered MDOF system are conducted. Since the structure is assumed to have a first-mode dominant behavior, we used maximum roof drift as the global response parameter of the analyzed structure.

Considering the idealized pushover curve and corresponding ADRS parameters, the nonlinear response of the SDOF system is represented by a nondegrading bilinear hysteretic model with 3% postyield stiffness. The SDOF response values are assumed as lognormally distributed. The comparisons between alternative ground motion selection and scaling methodologies are based on the median and dispersion values. The comparative statistics are described for a constant strength value of 4 as determined in the previous section. The investigated procedure estimates the median and the standard deviation values as 13.7 cm and 2.58 cm, respectively. Meanwhile, SDOF analysis of accelerograms assembled by the investigated study yields a median value of 14.3 cm whereas the dispersion about this median is found as 1.91 cm. Strong-motion recordings selected and scaled by the CMS-based procedure result in 13.2 cm as the median value and 1.98 cm as the standard deviation value. For this specific case, the SDOF results advocate that the performance of the presented procedure is better with respect CMS-based procedure due to the smaller dispersion in nonlinear response. Besides, the methodology proposed by Ay and Akkar (2012) can provide preliminary information about the probability distribution of structural response through estimated median and dispersion statistics.

The selection and scaling strategies are also compared using nonlinear response history analyses of 4MRF-DD frame presented above. Using SeismoStruct, nonlinear dynamic analyses are performed by employing Hilber-Hughes-Taylor integration method (Hilber et al., 1977) and 4% Rayleigh damping for the first mode. The response statistics are assumed as lognormally distributed. Median and standard deviation of response are obtained in order to define the roof drift distribution. The median value of maximum roof drift is estimated as 17.5 cm by the presented procedure whereas the standard deviation about median is estimated as 3.32 cm. The accelerograms that are selected and scaled according to our procedure yield a median and standard deviation value of 16.4 cm and 1.82 cm, respectively. The CMS-based procedure results in a median value of 16.4 cm. The standard deviation about this median is 3.31 cm.

These results are summarized in Table 3.2 for both SDOF and MDOF analysis. As it can be inferred from Table 3.2, the estimated and observed values obtained by the investigated procedure are in fair agreement.

Table 3.2. Probability distribution parameters

	SDOF			**MDOF**		
	Ay and Akkar (2012) estimated	**Ay and Akkar (2012) observed**	**Baker (2011) observed**	**Ay and Akkar (2012) estimated**	**Ay and Akkar (2012) observed**	**Baker (2011) observed**
Median	13.7 cm	14.3 cm	13.2 cm	17.5 cm	16.4 cm	16.4 cm
Standard Deviation	2.58 cm	1.91 cm	1.98 cm	3.32 cm	1.82 cm	3.31 cm

According to the results given above, it is seen that the chosen hysteretic model fairly represents the post-elastic behavior of the investigated frame. Consequently, for a given hysteretic model, it is concluded that the investigated procedure proposed by Ay and Akkar (2012) approximately estimates the likely distribution of the structural response which conveys useful information for rapid performance assessment studies.

4. SUMMARY AND CONCLUSION

In this study, a new ground motion selection and scaling methodology proposed by Ay and Akkar (2012) is verified. The objective of the presented methodology is to assess the structural response accurately without excessively manipulating the inherent aleatory variability of the selected recordings. The limitations of this procedure are studied by comparing the results with CMS-based selection and scaling.

The presented procedure comprises of determination of candidate accelerograms, final selection of the ground-motion set and scaling of subject records to the target intensity level. In this study, the determination of candidate records is accomplished by using the geophysical parameters imposed by the scenario earthquake. The optimum number of recordings in candidate ground-motion dataset is also examined. As part of this paper, the final selection and scaling methodology is briefed as well.

A case study is provided to test the presented methodology in terms of structural response statistics. Records having reliable magnitude, site class, style-of-faulting and source-to-site distance information are used. Candidate accelerograms are selected from events with similar values of target magnitude and distance. Within these recordings, two ground-motion sets, one from the presented method and the other from CMS-based method, are assembled. Using these accelerograms, both SDOF and MDOF analyses of a RC structure, a redesign of a real building located in İstanbul, are conducted.

According to the results, the median roof drift values obtained from compared methodologies are similar. Besides, the comparisons advocate that the procedure proposed by Ay and Akkar (2012) would result in reduced dispersion for the examined structure. These statements hold for both SDOF and MDOF analyses. Moreover, fairly acceptable approximations to the observed distribution parameters are achieved by the estimations of Ay and Akkar (2012), which is promising for performance estimation of structural systems. However, further investigations are necessary for structural response parameters different than the maximum roof drift. Besides, buildings with significant higher mode effects should be examined carefully to verify the versatility of the presented methodology. Nevertheless, the case study results have shown that the investigated procedure is capable of providing a suitable ground-motion set for structural response assessment studies and it is capable of estimating the approximate distribution of structural response.

REFERENCES

Applied Technology Council, ATC-40. (1996). Seismic evaluation and retrofit of concrete buildings. Rep. No. ATC-40, Applied Technology Council, Redwood City, CA.

Applied Technology Council, ATC-58. (2009). Guidelines for seismic performance assessment of buildings. ATC-58 50% Draft, Redwood City, CA.

Ay, B.Ö. and S. Akkar. (2012). A Procedure on Ground Motion Selection and Scaling for Nonlinear Response of Simple Structural Systems, *Earthquake Engineering and Structural Dynamics*. DOI: 10.1002/eqe.1198.

Baker JW, Cornell CA. (2006). Spectral shape, epsilon and record selection. *Earthquake Engineering and Structural Dynamics*. **35:9**,1077–1095.

Baker JW. (2011). Conditional Mean Spectrum: Tool for ground-motion selection. *Journal of Structural Engineering*. **137:3**,322–331. DOI 10.1061/(ASCE)ST.1943-541X.0000215.

Bommer JJ, Acevedo AB. (2004). The use of real earthquake accelerograms as input to dynamic analysis. *Journal of Earthquake Engineering*. **8 : (Special Issue 1)**,43–91.

Hilber H.M., Hughes T.J.R., Taylor R.L. (1977). Improved numerical dissipation for time integration algorithms in structural dynamics. *Earthquake Engineering and Structural Dynamics*. **5: 3**, 283-292.

Luco N, Bazzurro P. (2007). Does amplitude scaling of ground motion records result in biased nonlinear structural drift responses? *Earthquake Engineering and Structural Dynamics*. **36:13**,1813–1835.

Mander, J.B., Priestley M.J.N., Park R. (1988) Theoretical stress-strain model for confined concrete. *Journal of Structural Engineering*. **114: 8**, 1804-1826.

Prota (2007). Probina Orion – An analysis and design platform for reinforced concrete structures.

Seismosoft (2010). SeismoStruct – A computer program for static and dynamic nonlinear analysis of framed structures. Available from URL: www.seismosoft.com

Stewart JP, Chiou SJ, Bray JD, Graves RW, Somerville PG, Abrahamson NA. (2001). Ground motion evaluation procedures for performance-based design. PEER Report 2001/09, Pacific Earthquake Engineering Research Center, University of California, Berkeley.

TEC. (2007) Specification for buildings to be built in seismic zones. TEC-07, Ministry of Public Works and Settlement, Government of Republic of Turkey, Ankara, Turkey.