Analysis of code-based ground-motion selection procedures in terms of inelastic interstorey drift demands

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LISBOA 2012

SUMMARY:
All the most important seismic codes, allowing to adopt nonlinear dynamic analyses, prescribe the use of suites of accelerograms representative of the seismicity at the considered site and whose average pseudo-acceleration response spectrum is compatible with a given UHS in an appropriate period range. This calculation of the structural response may lead to wrong estimates when dealing with non-linear systems.
In this paper we focused on the definition of a procedure for estimating the reference mean structural response for non-linear structures. We first defined attenuation relationships for the inelastic demand on various SDOF and MDOF structures and then performed PSHA using the obtained models. We obtained the interstorey drift levels with a 475 year return period: these were used as reference response and compared to the estimates of the average response obtained by using spectrum-compatible suites of accelerograms with the same return period, selected according to different criteria.

Keywords: Accelerograms, Nonlinear dynamic analysis, Probabilistic seismic hazard analysis.

1. INTRODUCTION

Among the currently employed methods for the analysis and the design of structures potentially subjected to seismic actions, nonlinear dynamic analysis is the most accurate in describing the structural behaviour. Nonlinear time-history analyses allow to predict the response of every element of the structure, studying how they interact during the formation and propagation of damage. In this framework, the structural response for a given earthquake scenario is estimated by loading the structure with acceleration time-histories that are compatible with the scenario in question. So far, however, there are many open issues on selection procedures to obtain such sets of accelerograms.

Numerous approaches have been proposed for selecting recorded accelerograms in order to obtain robust estimates of the structural response. They can be divided in two main categories, depending on the target of the analysis to be performed (Cornell 2005; Baker and Cornell 2006; Hancock 2006; Watson-Lamprey and Abrahamson 2006; Bradley 2010; Iervolino, Galasso et al. 2010; Katsanos, Sextos et al. 2010; Baker, Lin et al. 2011; Buratti, Stafford et al. 2011): *i*) an analysis aimed at evaluating a central estimate, such as the mean or median, of the structural response (that may then be used for design purposes); ii) an analysis aimed at estimating the full distribution of the structural response. The latter type of analysis is required in earthquake loss assessment procedures in which one must not only consider the potential damage associated with the expected response, but also the damage due to the full range of possible responses that may be experienced under a particular scenario. On the other hand, the first type of analysis is widely used by design codes. Seismic codes prescribe the use of suites of ground motions that are representative of the seismicity at the site under consideration and whose average pseudo-acceleration response spectrum is compatible with a given Uniform Hazard response Spectrum (UHS) in an appropriate range of periods. The so obtained suites of ground motions are used to estimate the structural response, normally the interstorey drift, which is calculated considering the average of the results of the analyses performed using each ground motion.

While this calculation of the response is correct for linear structures, it may lead to wrong estimates when dealing with non-linear systems. Furthermore in this latter case the results may become sensitive to some selection parameters like magnitude, source-to-site distance, epsilon, scaling, etc. Following this approach, many studies have been made investigating the influence of different selection criteria on the structural response (e.g. (Haselton 2009)) but they are often limited in terms of number of structures considered and in terms of ground-motions used.

In this study, we tested different selection procedures on various SDOF and MDOF structures with different nonlinear behaviours. In the first stage of the study we defined a reference ground-motion data-set that we used to derive ground-motion prediction equations for spectral accelerations and PGA. These attenuation relationships were then adopted to derive UHS, through PHSA, for some case study sites. The so obtained UHS were employed to define a set of case study SDOF and MODF nonlinear systems that were characterized by different periods and behaviour factors. Attenuation relationships were then derived for the interstorey drift of these systems and used to perform PSHA. This latter analysis allowed to define the interstorey drift values corresponding to different return periods. They were then used as the reference response for assessing different spectrum-based ground-motion selection procedures.

The developed procedure allowed to investigate in a consistent and comprehensive way many issues related to code-based ground-motion selection procedures such as, for example, the effect of scaling the time histories, the influence of the range of magnitude and distance considered in the selection and the width of the interval of periods for which the compatibility is required.

2. DEFINITION OF REFERENCE UHS

In the first phase of the present work reference UHS in terms of PSA were defined by performing PSHA for a set of case study sites. In order to maintain consistency, the same ground-motion dataset was used in all the different stages of the study.

2.1. Ground–motion data-set

We used a subset of the time–histories in the NGA-database (Power, Chiou et al. 2006). Not all the accelerograms reported were used: in fact the ones with no information about the moment magnitude, the source-site distance, the shear wave velocity in the upper 30 m and the rupture mechanism, were rejected. According to these criteria the accelerograms used in the analyses were 5523.

2.2. Attenuation models for PSA and PGA

Attenuation models were developed for PGA and PSA at 75 different periods spanning from 0.05 s to 5 s using the dataset defined in Section 2.1. The number of accelerograms used to evaluate PSA at different periods was not constant because recordings with too short Lowest Usable Periods were not considered. Ground–motion prediction equations were then developed considering moment magnitude, M_w, Joyner–Boore distance, R_{JB}, and shear wave velocity in the upper 30 m, $V_{S,30}$, as independent variables. Inter–event, intra–event and inter–component error terms were considered in the non–linear regression model.

The functional form adopted was:

$$\log_{10}\mathrm{psa}(T) = c_1 + c_2 \cdot M_w + c_3 \cdot (M_w - 6)^2 + (c_4 + c_5 \cdot M_w) \cdot \log_{10}\left(\sqrt{{R_{JB}}^2 + {c_6}^2}\right) + \\ + c_7 \cdot \log_{10}(V_{S,30}) \tag{2.1}$$

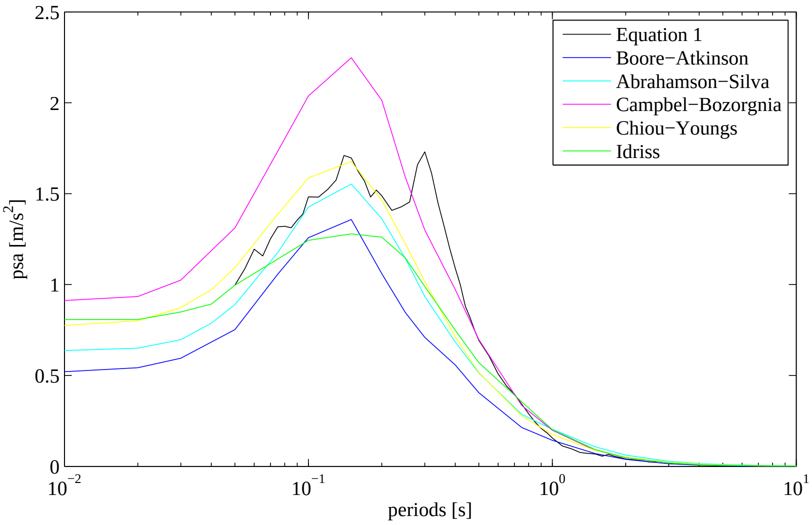

Figure 2.1. Comparison between different attenuation models (M_w = 5, R_{JB} = 10 km, $V_{S,30}$ = 1000 m/s^2)

The style of faulting was not included, as the regression analyses did not lead to statistically significant coefficients. A similar regression model was used by Buratti, Stafford et al. (2011).
The total standard deviation is obtained by combining the standard deviations of the error terms defined above as:

$$\sigma_T = \sqrt{\sigma_E^2 + \sigma_A^2 + \sigma_C^2} \tag{2.2}$$

where σ_E^2 is the variance of the inter–event term, σ_A^2 the variance of the intra–event term, and σ_C^2 the variance of the inter–component term. the PSA and PGA values are assumed to be lognormally distributed. This assumption has already been used by many researchers (Bazzurro, Cornell et al. 1998; Shome, Cornell et al. 1998; Cornell, Jalayer et al. 2002; Baker and Cornell 2006; Stoica, Medina et al. 2007) and is well supported by the distributions of residuals obtained with the regression analyses.

Although many authors have already proposed attenuation models for the ground–motion parameters considered, in the present work we have independently derived ground–motion prediction equations in order to achieve the highest possible consistency with the results that will be discussed in the following Sections. In fact, the accelerograms that will be used for deriving attenuation relationships for interstorey drifts (see Section 3.2) and that will be used to analyse UHS–based accelerogram selection criteria are the same used in this Section. Fig. 2.1 shows a comparison among the attenuation model derived in the present study and the models derived by Boore-Atkinson(2007), Abrahamson-Silva (2008), Campbell-Bozorgnia (2007), Chiou-Youngs (2008) and Idriss (2008) NGA model. The curves in Figure 2.1 correspond to the following scenario: M_w = 5.0, R_{JB} = 10.0 km, and $V_{S,30}$ = 1000 m/s^2.

2.3. PSHA

Using the attenuation models derived in Section 2.2, we performed a Probabilistic Seismic Hazard Analysis in order to obtain the UHS associated to the return period commonly used for life safety limit states (475 years).

The PSHA was carried out using CRISIS2007, a software developed by the Universidad Nacional Autonóma, México. The site considered for the analysis corresponds to Bologna, Italy. The source models defined by the INGV to derive the latest Italian Hazard maps were adopted; therefore no linear or punctual sources were considered. The seismicity of each zone was characterized by the Gutenberg-

Richter recurrence relationships the parameters of which were calculated using the seismic catalogue CPTI04 and the completeness intervals CO-04.4 (Meletti and Montaldo 2007).

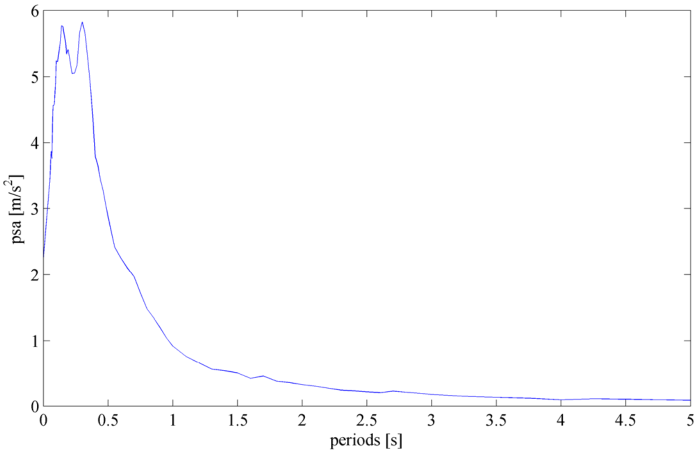

Figure 2.2. Uniform Hazard Spectrum (return period: 475 years)

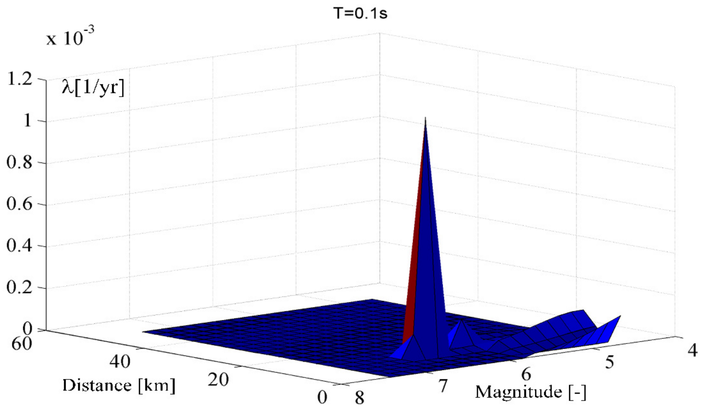

Figure 2.3. Disaggregation M_w-R_{JB} (fundamental period: 0.1s)

The PSHA gives, for each period, the level of the intensity measure considered (PSA in this case) associated to different mean annual frequencies of exceedance. Once the return period was fixed (e.g. 475 years) the UHS depicted in Fig. 2.2 was obtained by repeating the PSHA for every natural period considered. This spectrum will be used in the following as reference for ground motion selection procedures.

Another important result of the PSHA is the disaggregation. This latter allows to identify the seismic scenario (in terms of magnitude and distance) with the largest contribution to the hazard, in terms of one of the intensity measure considered, at the site under investigation. Figure 2.3 shows the disaggregation for the PSA at T = 0.1 s with a return period of 475 years. It can be pointed out that, if the fundamental period is smaller than 1 s, just one modal value could be identified, while for longer periods we record multimodal disaggregations. In Table 2.1. we list the couples of M and R_{JB} identified through the disaggregation for 5 of the 75 periods.

Table 2.1. M_w and R_{JB} associated to the modal values from the disaggregation (for T = 2 s we report the two most significative combinations of M_w/R_{JB})

		M_w [-]	R_{JB} [km]			M_w [-]	R_{JB} [km]
T = 0.1 s	modal value 1	6.393	10.101	T = 1 s	modal value 1	5.812	5.051
T = 0.3 s	modal value 1	6.393	10.101	T = 2 s	modal value 1	5.086	0
T = 0.5 s	modal value 1	5.812	7.576		modal value 2	5.812	5.51

3. DEFINITION OF REFERENCE NONLINEAR DISPLACEMENTS

3.1. Structures considered

Once the UHS in terms of PSA was calculated, the structures to be subjected to the nonlinear dynamic analyses were defined. Both Single Degree of Freedom (SDOF) and Multi Degree of Freedom (MDOF) elastic–plastic systems were considered. These structures were defined from a simulated design procedure starting from the UHS obtained in Section 2.3. In particular the yielding force of the SDOF systems were calculated using behaviour factors, q, spanning from 1 to 5 and considering the natural periods 0.1 s, 0.3 s, 0.5 s, 1.0 s, and 2.0 s. A 5% hardening ratio was considered. Three MODF systems with 2, 4, and 10 degrees of freedom were considered. Their mechanical properties were defined using the same behaviour–factor values adopted for the SDOF systems while the natural periods assumed were 0.3 s, 0.5 s and 2.0 s for the 2– and 4–degrees of freedom systems and 0.5 s, 1.0 s and 2.0 s for the 10–degrees of freedom system. Each structure was then analysed with the same subset of records from the Next Generation of Attenuation (NGA) database described in Section 2.1. Both interstorey and roof drifts were evaluated, as these parameters are the most widely used to characterize nonlinear structural response.

3.2. Attenuation models in terms of drift

We performed a second regressions analysis to evaluate a prediction model for the displacements. The same functional form used to define the spectral accelerations has been used also for the various measures of drift considered:

$$\log_{10} X_{max}(T) = c_1 + c_2 \cdot M_w + c_3 \cdot (M_w - 6)^2 + (c_4 + c_5 \cdot M_w) \cdot \log_{10}\left(\sqrt{{R_{JB}}^2 + {c_6}^2}\right) + \\ + c_7 \cdot \log_{10}(V_{S,30}) \qquad (3.1)$$

where X_{max} is the maximum value of interstorey or roof drift.

With reference to the SDOF structures, the empirical relationship obtained for q = 1 was compared with the attenuation model calculated for the linear elastic oscillators (Fig. 3.1). Since for a unitary behaviour factor the yielding strength of the elastoplastic systems coincides with the elastic force applied on the elastic ones, the expected excursions in the plastic range of the nonlinear structures are small.

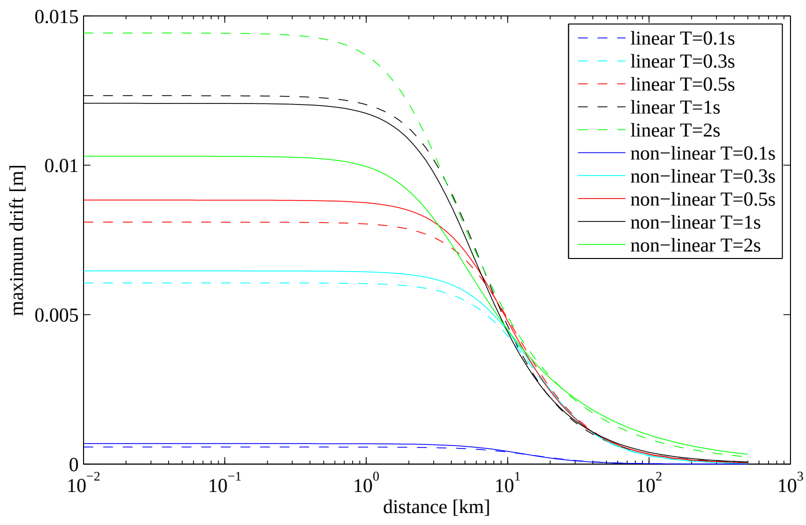

Figure 3.1. comparison between elastic and elastoplastic attenuation relationships in terms of maximum displacement (q=1)

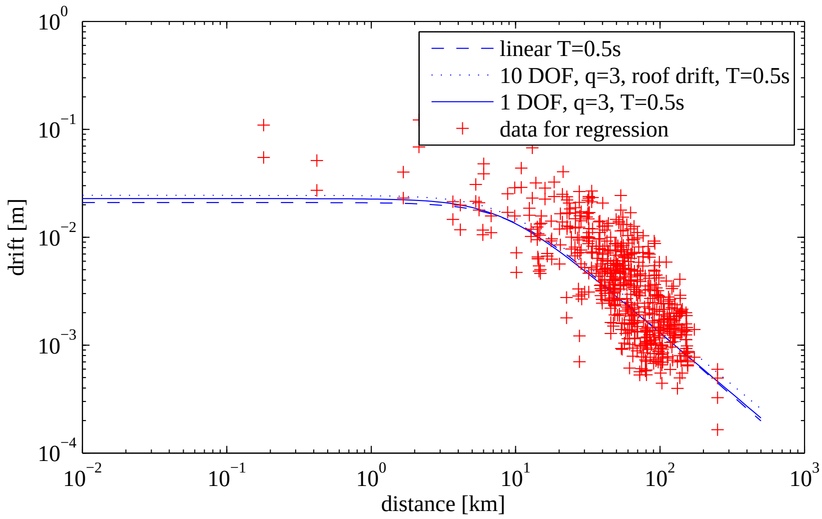

Figure 3.2. GMPE (M_w = 6.0, $V_{S,30}$ = 1000 m/s^2, T = 0.5 s) for the elastic case, the SDOF elastoplastic structure and the 10 degree of freedom system (q = 3) and the roof drift data used for the regression (M_w = 6.0 ± 0.5)

Figure 3.1 confirms that the elastic and elastoplastic attenuation relationships, both expressed in terms of maximum displacement, show a very similar trend with the exception of the oscillator characterized by a fundamental period of 2.0 s. It is believed that such a discrepancy between the displacements predicted by the two models is due to the fact that almost a half of the recordings applied induced non linear deformations on the elastoplastic structure and, therefore, a comparison between the behaviour of this oscillator and the elastic one is not significant.

It is worth noticing that the equal displacement rule was not valid in many cases, as already observed by other researchers (Bozorgnia, Hachem et al. 2010).

Figure 3.2 compares the Ground Motion Prediction Equations (for M_w = 6.0 and $V_{S,30}$ = 1000 m/s^2) obtained for the elastic and the SDOF elastoplastic structure characterized by a fundamental period of 0.5 s with the elastoplastic 10 degree of freedom system when the behaviour factor is equal to 3. In the latter case the structural response considered is the maximum roof drift. Figure 3.2 also shows, with red crosses, the data used for the regression (M_w = 6.0 ± 0.5). It should be noticed that before comparing the displacements (and the attenuation relations) associated to the MDOF oscillator to the two other types of displacements, the first ones had to be divided by the participation factor, which was, in this case, 1.4899.

As for the elastic case, some tests were made to verify if the functional form adopted was appropriate to represent the data. Particular attention was devoted to the quantiles of the residuals. It was observed that the distribution of logarithm of the standardized residuals of the displacements followed the Normal distribution in a closer way for longer vibration periods than for shorter ones. The behaviour factor seemed to have no particular influence on the normality of the residuals. Although the hypothesis of lognormal distribution could still be considered valid, an improvement in the regression analyses could be reached by replacing the logarithmic transformation with an exponential transformation.

3.3. PSHA in terms of drift

The crucial point in evaluating the performance of accelerogram selection procedures is the definition of a reference structural response: in the present work the effectiveness of the considered criteria was studied comparing the response estimated with sets of ground–motions selected according to different criteria (see Section 4) to the structural response levels with the return period related to the limit state considered, 475 years in this case (Bozorgnia, Hachem et al. 2010). These levels were defined by carrying out a second PSHA using the attenuation models described in Section 3.2. Through this

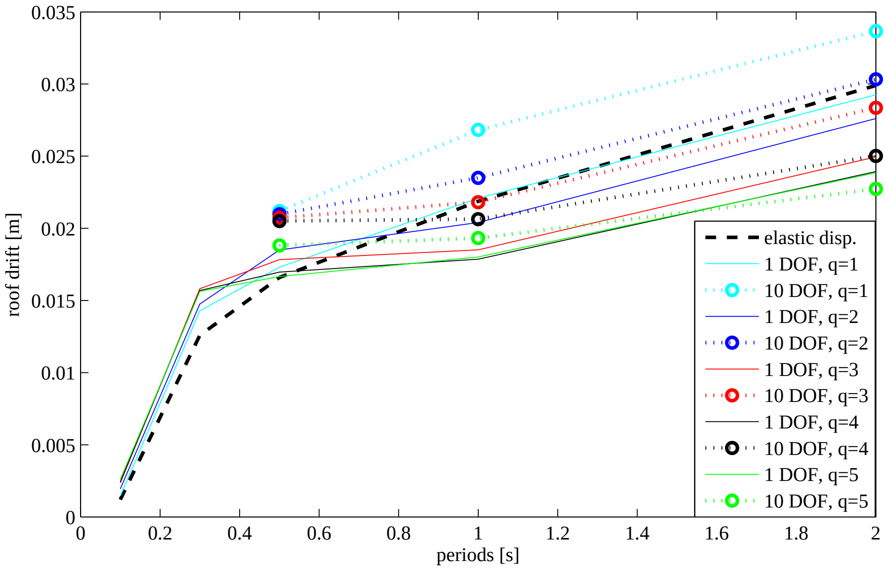

Figure 3.3. Uninform Hazard elastic displacement response Spectrum vs Inelastic Uniform Hazard displacements/drifts Spectrum for SDOF systems and 10–degree of freedom systems (q = 1,2,3,4,5)

process we obtained the maximum displacements (for the SDOF oscillators) and the interstorey and roof drifts (for the MDOF oscillators) with a 475 years return period. Figure 3.3 shows the uniform hazard elastic displacement response spectrum and the inelastic uniform hazard displacements/drifts for SDOF systems and 10–degree of freedom systems. The behaviour factor spans from 1.0 to 5.0.
As already observed in Section 3.2, the PSHA in terms of drift gives, for the longest fundamental period considered, a multimodal disaggregation.

4. ANALYSIS OF THE GROUND–MOTION SELECTION PROCEDURES

The aim of the present work was to assess the compatibility between the UHS and sets of accelerograms selected according to different criteria. The general idea was to make a preselection of the recordings contained in the database in order to obtain groups of time histories characterized by the same particular properties (e.g. the same interval of source-site distance). We wanted to evaluate how the application of each of these criteria to the data-set affected the composition of the spectrum compatible suites of ground motions. In particular, we wanted to check whether there were selection procedures allowing to identify the accelerograms that generated on the system a structural response comparable to the one expected.

The criteria taken under consideration were:

- maximum magnitude;
- maximum distance;
- preselection in terms of a combination of magnitude and source-site distance;
- preselection in terms of compatibility of the individual accelerograms;
- preselection in terms of width of the periods range for which the compatibility is required;
- preselection in terms of a combination of magnitude and source-site distance of scaled accelerograms.

4.1. Results

In this section we present an application of the method proposed on a 4-degree of freedom system. We considered 12 values of magnitude spanning from 5.8 to 7.41 and 10 values of distance from 5 to 105 km. These were the central values of the intervals used for the selection. For each combination of the aforementioned M_w and R_{JB} we chose the time histories characterized by a magnitude included in the interval M_w±0.2 and by a distance belonging to the range R_{JB}±20 km. Among the identified accelerograms, we chose those with an avarage pseudo-acceleration response spectrum compatible with the UHS in a range of periods including the fundamental one (in this case T = 0.5 s). The so obtained suites of ground motions were used to estimate the mean structural response, which was calculated by averaging the results of the analyses performed using each ground motion. In Figures 4.1 and 4.2 a comparison between this structural response an the reference displacement is shown for both unscaled and scaled ground motions (considering q = 1). The error between the two displacements was calculated for every combination of M_w and R_{JB} with the general expression:

$$\Delta_{ij} = \left| \frac{\delta_{ij} - \delta_{ij,ref}}{\delta_{ij,ref}} \right| \tag{4.1}$$

where δ_{ij} is the roof drift associated to the i-th period and the j-th behaviour factor. If the root-mean-square difference between the average spectrum and the UHS in the range of periods of interest was larger than 0.2 the suite of accelerograms were rejected: in this case we fixed Δ_{ij} = 1. Two restrictions on the scaling factor was also imposed: it had to be smaller than 5 and larger than 1/5.

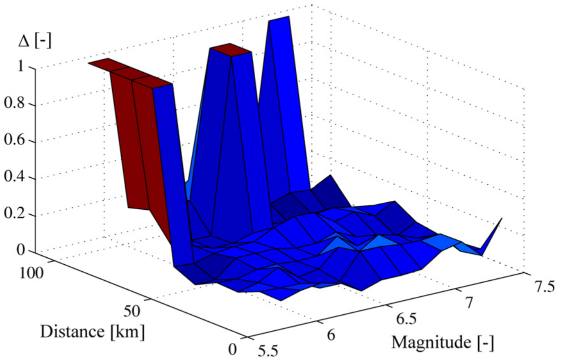

Figure 4.1. drift error when considering unscaled accelerograms (period range 0.48 - 0.55, q = 1)

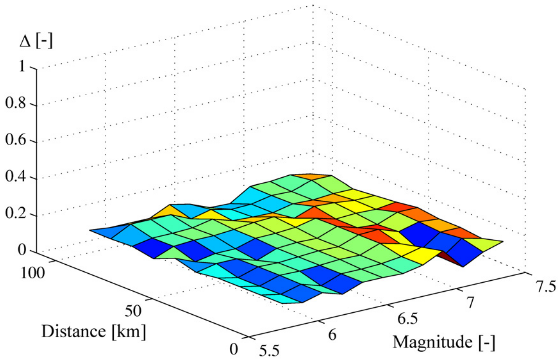

Figure 4.2. drift error when considering scaled accelerograms (period range 0.48 - 0.55, q = 1)

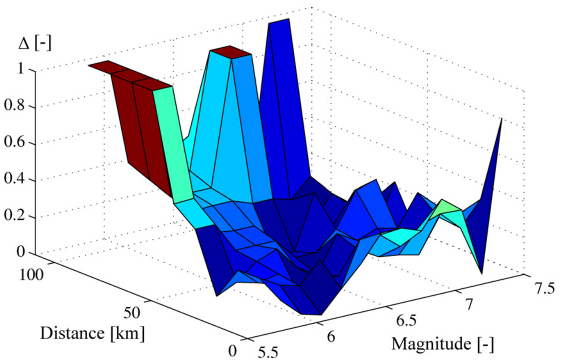

Figure 4.3. drift error when considering unscaled accelerograms (period range 0.48 - 0.55, q = 5)

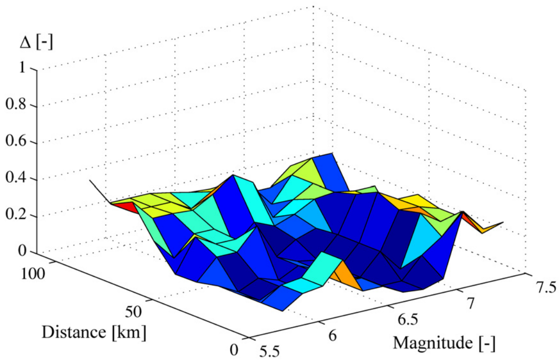

Figure 4.4. drift error when considering scaled accelerograms (period range 0.48 - 0.55, q = 5)

Figures 4.3 and 4.4 show the values of Δ when the behaviour factor considered was 5.

It can be noticed that with the introduction of the scaling procedure, at least one suite of accelerograms with a root-mean-square difference smaller than 0.2 can be found and, in general, that the error between the displacements is less influenced by the range of magnitude and distance used for the selection. Another observation that can be made is that an increase of the behaviour factor produces an increment of error. Nevertheless, it is still possible to identify an area of the surfaces, corresponding to ranges of distance and magnitude that include the values returned by the disaggregation, where Δ remains relatively small.

5. CONCLUSIONS

A reference structural response is needed for the study of the reliability of various selection criteria of the accelerograms used in nonlinear dynamic analyses. In the present paper a procedure to evaluate the reference displacement associated to a desired return period is presented.

The return period considered is 475 years and 75 periods and 5523 time histories from the NGA-Database have been analyzed. This data was necessary to calibrate the empirical model that predict the pseudo-acceleration once the magnitude, the source-site distance and the $V_{S,30}$ were known. A Uniform Hazard Spectrum associated to the chosen return period was identified using a Probabilistic Seismic Hazard Analysis. This UHS was then used to design SDOF and MDOF elastoplastic structures once the behaviour factor q was introduced. The next step was to define an attenuation model from the drifts induced on the oscillators by the time histories and to perform a second PSHA using this ground motion prediction equation. We were then able to identify the displacements expected with a fixed return period.

A comprehensive study of the reliability of different selection procedures is currently under development and will constitute the subject of future contributions.

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