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THE EFFECT OF SEISMIC LOADING SIMULATION ON THE RESPONSE VARIABILITY OF STRUCTURES

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Abstract. *This paper investigates the variability of the seismic response of structures subjected to various ensembles of ground motions in the framework of nonlinear dynamic analysis. In this kind of analysis, seismic loading, introduced in the form of excitation accelerograms, constitutes the most important source of uncertainty, as it depends on a number of random factors (e.g. distance from the fault source, magnitude, local soil conditions). In order to quantify the effect of seismic loading simulation on the response variability of structures, nonlinear dynamic analysis is carried out on a steel frame, subjected successively to three ground motion ensembles. The ensembles comprise, respectively, real ground motions and artificial accelerograms generated by two different algorithms, one in accordance with the Code provisions and one targeted to a more realistic simulation of the seismic event. Comparison of the three approaches is made by monitoring the statistical properties of the maximum displacement and interstorey drift induced by the assumed ensembles.*

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1 INTRODUCTION

Time-history analysis is increasingly becoming the preferred method in examining the seismic performance of structures as it provides with a more accurate description of the response of a structure subjected to specific realizations of the seismic event, especially in the case nonlinear behavior is observed. However, subjecting a structure to a single recorded motion is not sufficient for the evaluation of its seismic performance, since the seismic process is strongly stochastic. To account for this stochasticity, modern Seismic Codes require that an ensemble of accelerograms following specific standards be used for the analyses. In particular, Eurocode 8 [1] dictates the use of seven or more accelerograms if the demand is to be calculated by averaging the response values, and in any case at least three. As the selection of an adequate number of real ground motions to satisfy the seismological and geological conditions, as well as the Codes' provisions is not always possible, the description of the seismic motion may be made by using artificial and recorded or simulated accelerograms. Additionally, a second requirement must be satisfied regarding the compatibility of the ground motions to the Codes' smoothed elastic response spectra. This condition is expressed in EC8 [1] as follows:

- 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 in question.
- in the range of periods between $0.2T_1$ and $2T_1$, where T_1 is the fundamental period of the structure in the direction where the accelerogram 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.

With regard to recorded motions, spectral compatibility is achieved either by scaling in the time domain or by filtering in the frequency domain. It should be noted that in the first case only the amplitude of the motion is modified, whereas in the latter the frequency content of the motion is distorted.

Artificial accelerograms meeting the above requirements can be produced by algorithms created ad hoc. One such algorithm is utilized by SIMQKE [2], a software that will be employed in this work.

The use of accelerograms compatible to the smoothed spectrum of the Seismic Codes may be tempting, as it offers a common framework for the comparison of time history analyses to static analyses. However, the practice has been criticized over conservative, unrealistic estimations, as well as loss of the inherent response variability under recorded motions [3]. The main issue with spectrum compatible accelerograms lies with the nature of the Codes' smoothed response spectrum. This spectrum is determined using a Probabilistic Seismic Hazard Analysis and thus represents the cumulative contribution of risk from the seismic sources in a region for a given risk level. Therefore, it is characterized by high energy content over the whole range of structural periods. Naturally, an accelerogram possessing such properties will exhibit large spectral acceleration values. In addition, the lack of variance that is observed is also attributed to the dependence of the ground motions on a single power spectrum, derived from the Codes' response spectrum.

The purpose of this work is to investigate the influence of the method used for the simulation of seismic loading on the response variability of a structure. In this context, two methodologies for the generation of artificial seismic motions which follow different approaches will be presented and compared. Nonlinear dynamic analysis will be carried out on a case study (steel frame) using the SAP2000 v18.1.1 [4] software, in order to identify possible differences between the responses. The model will be subjected successively to seismic motions, categorized into three distinct groups, comprising natural recordings and artificial accelerograms derived from the SIMQKE software [2] and the algorithm of Sabetta and Pugliese [5]. Useful observations on the responses will be made through statistical processing of the results.

2 ALGORITHMS FOR THE GENERATION OF ARTIFICIAL SEISMIC MOTIONS

The SIMQKE software [2] generates ground motions, over which the user has full control in terms of morphological and spectral characteristics. The production of the motions is based upon the relationship between the power spectral density function and the response spectrum. By taking as input the latter, the software calculates the respective power spectrum $G(\omega)$, and from that it produces accelerograms using the spectral representation method [6], as follows:

$$X(t) = \sum_{n=1}^{N} A_n sin(\omega_n t + \phi_n) \tag{1}$$

where

$$A_n = 2\sqrt{G(\omega_n)\Delta\omega} \tag{2}$$

As $G(\omega)$ is time-invariant, the non-stationarity of the seismic motion is incorporated by multiplying the output accelerogram, $X(t)$, with a transient envelope intensity function, $I(t)$.

$$a(t) = I(t)\sum_{n=1}^{N} A_n sin(\omega_n t + \phi_n) \tag{3}$$

This approach leads to accelerograms whose spectra fit the Code's smoothed spectrum, but do not contain the features of natural records. Besides, it should be noted that the evolutionary power spectrum $G(\omega, t)$ of natural ground motions is often non-separable.

The algorithm of Sabetta and Pugliese [5], on the other hand, incorporates the dependence of the motion on the magnitude, the distance from the source and the geological conditions, in an attempt to generate more realistic accelerograms. Based on these parameters, the evolutionary power spectrum or physical spectrum of the motion, $PS(\omega, t)$, is calculated. Again, accelerograms are generated by spectral representation, with the substantial difference that the transient nature of the seismic process is incorporated in the coefficients of the series, $C_n(t)$, which are determined by the physical spectrum:

$$\alpha(t) = 2\sum_{n=1}^{N} C_n(t)\cos(\omega_n t + \phi_n) \tag{4}$$

where

$$C_n(t) = \sqrt{PS(\omega_n, t)\Delta\omega} \tag{5}$$

Obviously, the resulting motions do not match the Code's spectrum, but resemble the natural records in a time-history level and in the frequency domain, alike.

3 MODEL FORMULATION

The model to be tested, as shown in figure 1, is a three-storey steel moment-resisting frame, simulated using frame finite elements. Frame sections are shown in table 1. The joints were simulated using separate, rigid finite elements. Shear deformation at the joints was incorporated by adding panel zones. The inelastic behavior of the structure was simulated by addition of appropriate energy dissipation zones, which are governed by nonlinear material law with kinematic hardening. Geometric non-linearity was not considered necessary to include, as its impact will be negligible, due to the height of the frame. Rayleigh damping was used to obtain a damping ratio of 2% for the first and the third mode. Point masses of $180tn$ were assumed in the middle of each floor and $80.26kN/m$ gravity loading was applied to the beams.

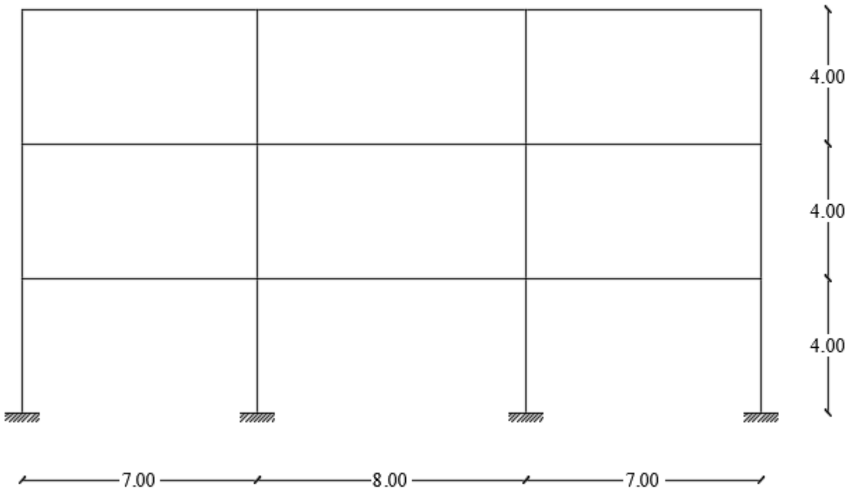

Figure 1: Three-storey steel moment-resisting frame

Table 1: Frame sections

Storey	Beams	Columns	
		Outer	Inner
1	IPE550	HEB360	HEB500
2	IPE550	HEB360	HEB500
3	IPE550	HEB360	HEB500

4 SEISMIC LOADING SIMULATION

4.1 Ensemble EU: recorded ground motions

The recorded motions for this work were extracted from the European Strong-Motion Database (ESD) [7]. The seismic component chosen from each event was the one exhibiting the maximum peak ground acceleration. Search parameters (presented below) were arbitrarily chosen, so that a direct comparison between real and artificial ground motions would be possible. Mean peak ground acceleration of around $0.25g$ was pursued for the ensembles as a common intensity measure. The accelerograms were used unscaled, as compatibility to the smoothed spectrum was not desired for this ensemble.

- Magnitude: $M_w = 5.5 - 6.9$
- Local soil conditions: *stiff soil*, corresponding to ground type B in EC8.
- Epicentral Distance: $> 10km$, such that near-fault motions will be excluded
- Peak horizontal ground acceleration: $PGA = 2.1 - 2.9m/s^2$

Table 2: Description of the ground motions of the ensemble EU

S/N	Event	Station	Date	PGA [g]
EU01	Friuli (aftershock)	Forgaria-Cornio	11/09/1976	0.232
EU02	Friuli (aftershock)	Forgaria-Cornio	15/09/1976	0.264
EU03	Friuli (aftershock)	San Rocco	15/09/1976	0.236
EU04	Montenegro (aftershock)	Bar-Skupstina Opstine	24/05/1979	0.270
EU05	Campano Lucano	Brienza	23/11/1980	0.227
EU06	Kalamata	Kalamata-OTE Building	13/09/1986	0.272
EU07	Kefallinia island	Argostoli-OTE Building	23/01/1992	0.227
EU08	Kozani (aftershock)	Karpero-Town Hall	19/05/1995	0.265
EU09	Montenegro	Ulcinj-Hotel Olimpic	15/04/1979	0.294
EU10	South Iceland	Selsund	17/06/2000	0.278

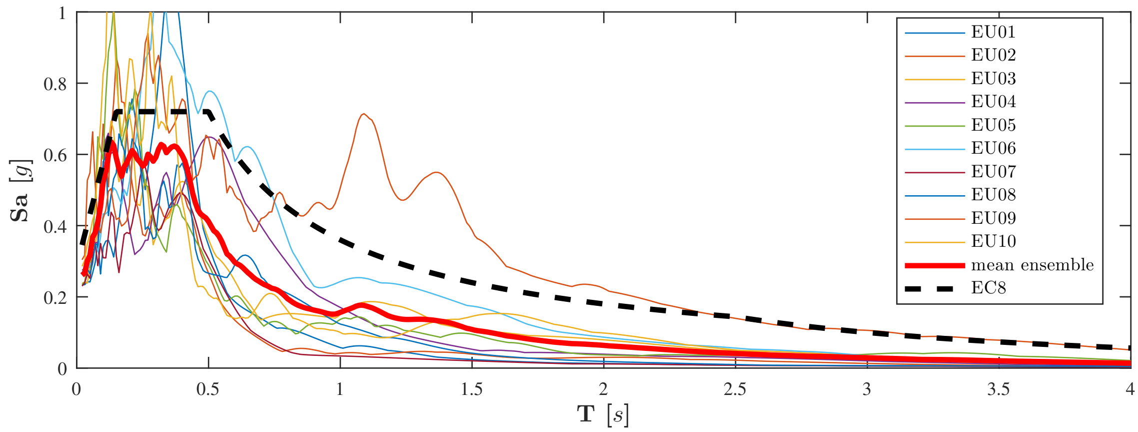

Figure 2: Comparative presentation of ensemble EU and EC8 5% damping elastic response spectra

4.2 Ensemble SIMQKE: artificial accelerograms compatible to smoothed elastic response spectrum

Artificial motions produced by the SIMQKE software compose the second ensemble. For the generation of the accelerograms the EC8 response spectrum corresponding to the studied structure was calculated. Spectral matching was pursued at an arbitrarily large number of periods (1235), while five optimization cycles were performed. Parameters for each accelerogram were input such that its graphical shape would match that of the respective motion from ensemble EU. More specifically, exact matching was achieved with regard to the durations and the peak ground accelerations. Compound envelope intensity functions were used to incorporate the non-stationarity of the seismic motions. To illustrate the aforementioned process, accelerograms #02 of both the EU and SIMQKE ensembles, along with relevant graphical information, are presented in figure 3. Response spectra for the latter ensemble are shown in figure 4.

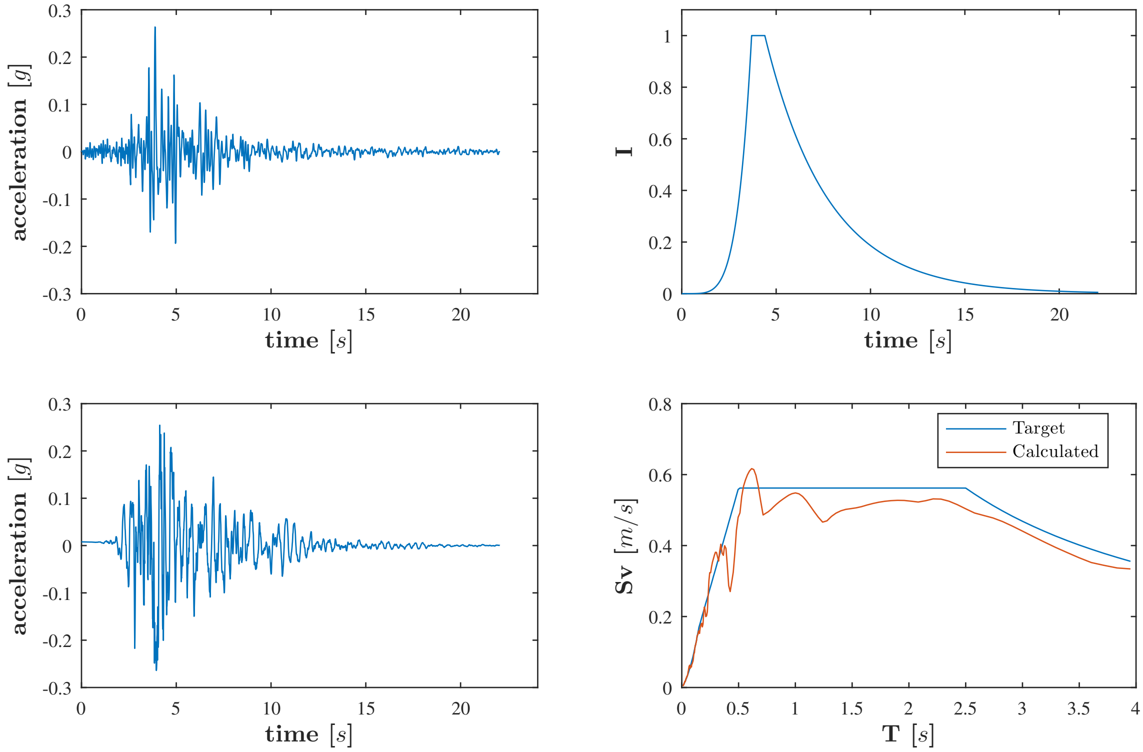

Figure 3: Left: comparative presentation of accelorgrams EU02 (top) and SIMQKE02 (bottom) Right: envelope intensity function (top), target and calculated response spectrum (bottom)

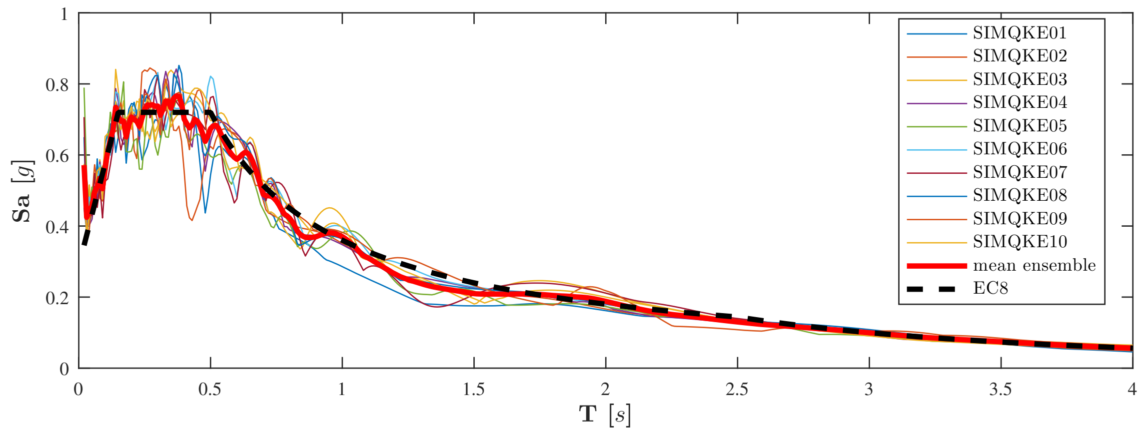

Figure 4: Comparative presentation of ensemble SIMQKE and EC8 5% damping elastic response spectra

4.3 Ensemble SABETTA: realistic artificial accelerograms

The third ensemble contains accelerograms generated by the algorithm created by Sabetta and Pugliese [5]. These motions are, by definition, considerably different from the first two sets, both in shape and in properties. The algorithm accepts as inputs parameters related to

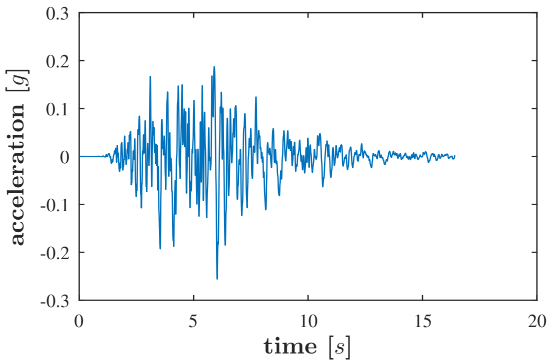

Figure 5: Accelerogram SABETTA02

seismological and geological conditions and therefore the user has little control over the resulting motion. The following values were input for this study targeting the aforementioned mean $PGA \simeq 0.25g$:

- Magnitude: $M_w = 6.0 - 6.8$
- Epicentral distance: $13 - 23km$
- Local soil conditions: *shallow alluvium*, characterized by mean shear-wave velocity, $V_{s,30} = 350 - 750m/s$ which approximately corresponds to ground type B in EC8.

Again, motion #02 of figure 5 is arbitrarily chosen as a representative sample of ensemble SABETTA, while figure 6 shows the ensemble's response spectra.

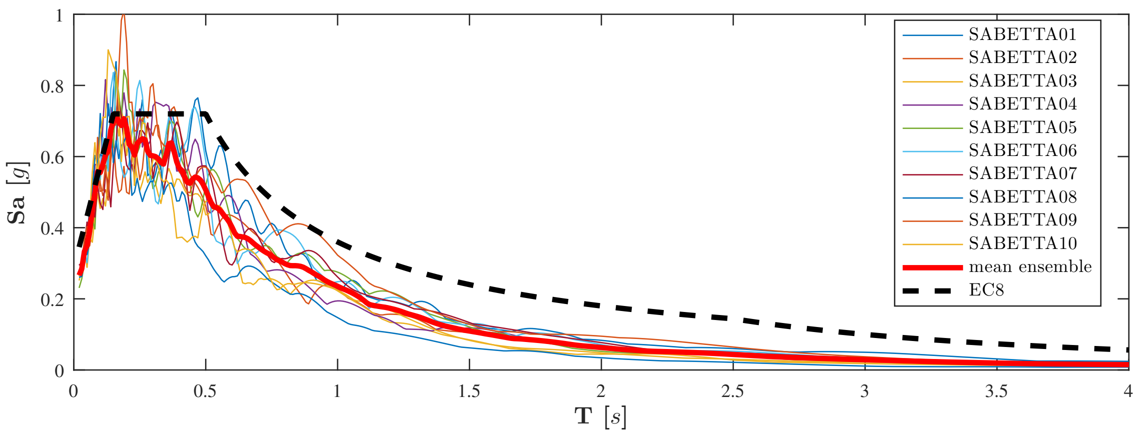

Figure 6: Comparative presentation of ensemble SABETTA and EC8 5% damping elastic response spectra

5 SEISMIC RESPONSE OF THE STRUCTURE

Information concerning the response of the structure can be extracted from the qualitative aspects of the ground motions even before the analyses are carried out. In this case, the stark differences among the assumed ensembles are indicative of the expected responses. At first, an observation can be made regarding the shape of the accelerograms. Those that stem from recorded motions exhibit strong variability as regards the total duration, the duration of the strong motion (approximately stationary part) and the relative position of the peak ground acceleration. Similar characteristics can be found in the motions composing ensemble SIMQKE, as they were intentionally produced to resemble the natural accelerograms. Nevertheless, the latter tend to overestimate the duration of the strong part and have a more gradual attenuation. This can be attributed in part to imperfections in the intensity envelope functions and to the optimization cycles that were executed for the better matching to the smoothed elastic response spectrum, during which the frequency content of the motion is altered. Lastly, the accelerograms belonging to ensemble SABETTA drastically differ from those of the above two ensembles. In particular, all of the motions loosely resemble an attenuated harmonic oscillation, while their duration does not exceed twenty seconds.

Regarding the mean elastic spectra, ensemble EU is situated below the smoothed Code's spectrum in the whole range of periods, ensemble SIMQKE adequately fits the Code's spectrum, and ensemble SABETTA matches the smoothed spectrum only in the ascending part, after which it follows a descending trend, similar to ensemble EU. Of particular interest are the shape and variance of the response spectra. In terms of shape, the natural records are very dissimilar from one another. Differences can be located in the values of the maximum spectral acceleration, which in some cases far exceeds the Code's spectrum's plateau, its position - record EU09 stands out, exhibiting peak value at a period higher than $T = 1s$ - and the period range in which remarkable energy content is encompassed - again, motion EU09 is distinctive as it shows high acceleration values in a large range of periods. The spectra of ensemble SIMQKE closely follow the smoothed elastic spectrum, with few deviations focused in the area around $T = 0.5s$. Finally, even though the spectra of ensemble SABETTA exhibit some dispersion around the mean, they do not deviate from it significantly, as far as the general course is concerned.

Based on the above observations, it would be reasonable to assume that the response values of the structure to the second ensemble will be higher than the other two, on average. Furthermore, the variability of the spectra of ensemble EU as well as the lack of it in the case of the other two ensembles are expected to lead respectively to responses with high variance on the one hand and uniformity and similar peak acceleration values on the other.

The influence of the dispersion of the spectra is better illustrated in figure 7, where the response spectra of the three ensembles are presented, normalized with regard to the spectral acceleration at the fundamental period of the structure, $S_a(T = 1.32s)$.

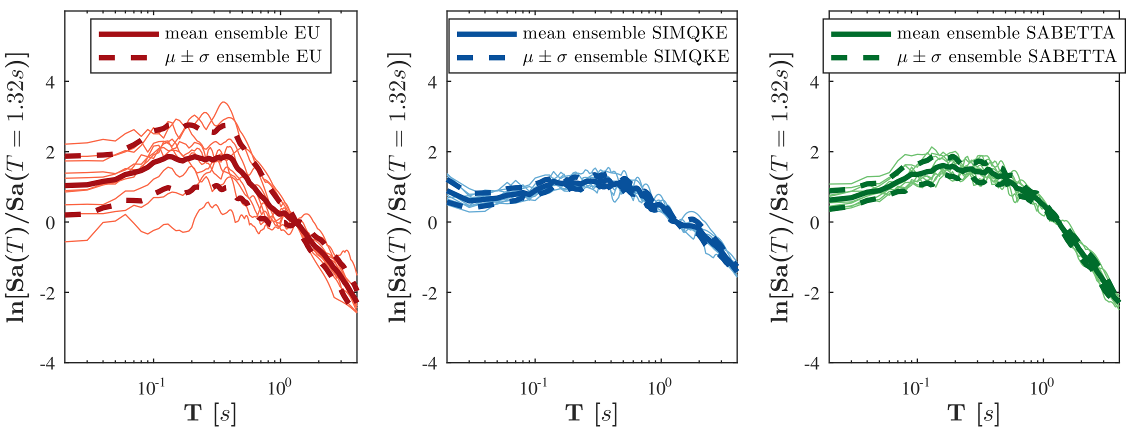

Figure 7: Comparative presentation of the three ensembles' response spectra, normalized with regard to the spectral acceleration at the fundamental period of the structure, $S_a(T = 1.32s)$

The response values are presented in terms of absolute floor displacements and interstorey drift, measured at the middle of each storey, where the center of mass lies. Maximum displacements of each floor induced by the three excitation ensembles, along with their respective mean values are presented in figure 8. It is observed that ensemble EU leads to responses with particularly large dispersion. Conversely, responses to the ensembles comprising artificial accelerograms are mostly concentrated around the mean, in total agreement with what had been predicted.

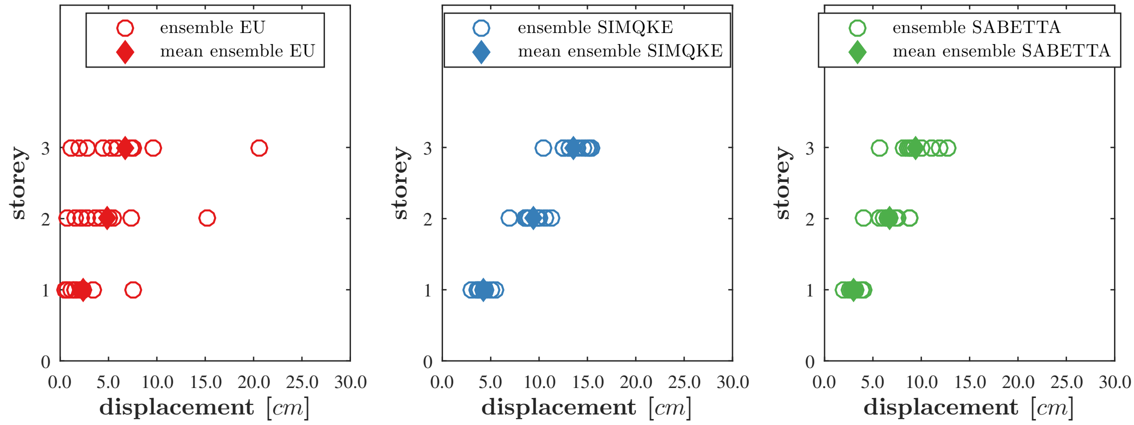

Figure 8: Maximum displacement values of the frame induced by the seismic loading

The above results are also presented in the form of a bar diagram in figure 9, in which the response to each excitation can be located. It is noted that ground motion EU09 subjects the structure to disproportionately large displacements in comparison to both the rest of the ground motions of ensemble EU and those of the other two ensembles. Looking back at figure 2 it is found that the ninth accelerogram is the one that exhibited especially high spectral acceleration values in the area around the fundamental period of the structure. This relationship between the maximum seismic response of the structure and the spectral acceleration at its fundamental period is to a large extent observed for all three ensembles.

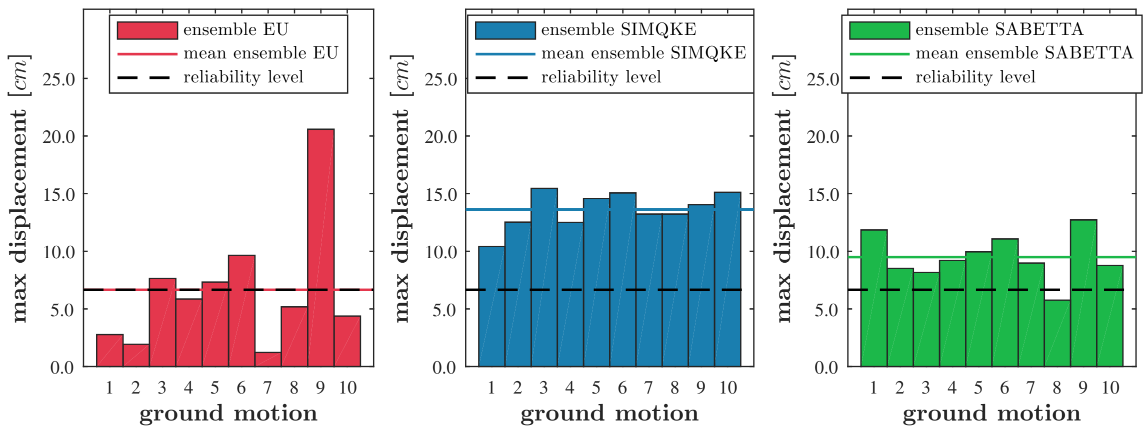

Figure 9: Maximum displacement values of the frame (3rd storey) induced by the seismic loading

Table 3 quantitatively presents the statistical properties of the responses. In addition to what has already been mentioned, and in conjunction with figure 9 the significance of the extreme response to excitation EU09 to the shaping of the mean of the first ensemble becomes apparent. By setting the minimum mean displacement value, that is of ensemble EU, as a common reliability level, $r = \mu_{EU}$, a rough estimate of the structural reliability of the frame can be made.

Table 3: Statistical properties of the maximum responses (3rd storey)

	$\boldsymbol{\mu}^{*}$[cm]	$\boldsymbol{\sigma}^{\dagger}$[cm]	CV‡	$\mathbf{R_{10}}(\boldsymbol{u} > \boldsymbol{\mu})^{\S}$	$\mathbf{R_{10}}(\boldsymbol{u} > \boldsymbol{r})^{\|}$
EU	6.55	5.57	0.84	0.40	0.40
SIMQKE	13.62	1.56	0.11	0.50	1.00
SABETTA	9.50	2.01	0.21	0.40	0.90

* mean value
† standard deviation
‡ coefficient of variation
§ relative frequency of exceedance of mean value
‖ relative frequency of exceedance of reliability level

At this point it should be recalled that Eurocode 8 [1] allows the calculation of the demand from the mean of the maximum responses if more than seven time-histories are used. By examining figure 10 and the frequency of exceedance of the reliability level (Table 3) it becomes clear that ensemble SIMQKE, comprising artificial accelerograms fitted to the Code's response spectrum, errs on the side of caution. Indeed, high correlation is observed between mean spectral acceleration at the position of the fundamental period of the structure and the mean of the maximum displacement values, for all three excitation ensembles. This suggests that the above acceleration, or at least the maximum spectral acceleration, would have been a more appropriate intensity measure for defining the ensembles.

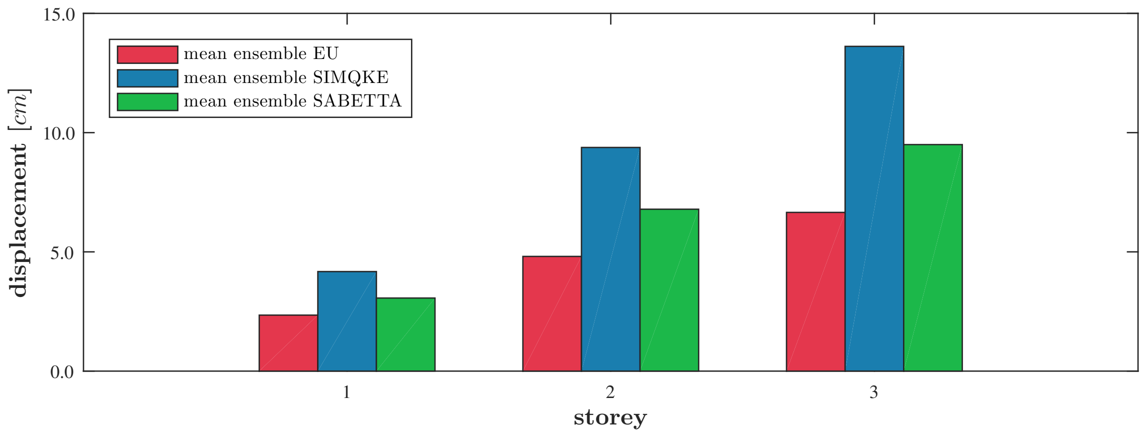

Figure 10: Mean of the maximum displacement values induced by the seismic loading

As a closing remark, it should be mentioned that the results of this study are fully in line with what was previously observed concerning the lack of dispersion of responses to artificial ground motions. Following this observation it should be emphasized that if maximum displacement values are used, natural recordings lead to a much more conservative calculation of the demand, as shown by the maximum drift in figure 11.

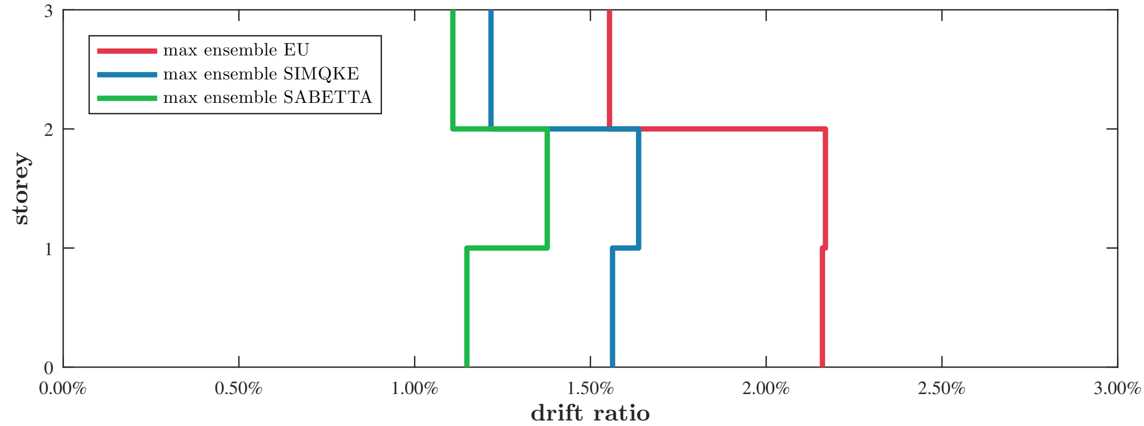

Figure 11: Maximum storey drift

6 CONCLUSIONS

This work has investigated and attempted to quantify the influence of the simulation methods used in generating seismic motions on structural response in the context of non-linear dynamic analysis. For this purpose, a steel moment-resisting frame was subjected successively to three ground motion ensembles. The use of recorded motions and artificial accelerograms produced by two algorithms which follow conspicuously different approaches, namely SIMQKE [2] and the algorithm of Sabetta and Pugliese [5] was decided.

The analyses revealed stark differences between the responses to natural recordings and artificial accelerograms. It is observed that responses to natural recordings show significant dispersion compared to the responses to the artificial ensembles. Furthermore, noticeable difference in the mean of the maximum responses and little difference in variance is found between the artificial ensembles. The strong uniformity observed in the responses to ensemble SIMQKE, which comprises accelerograms fitted to the Code's response spectrum, confirms earlier studies [3, 8]. Additionally, ensemble SIMQKE resulted in higher mean displacements of the control nodes. These observations are largely attributable to the correlation between the responses and the spectral acceleration of the excitation in the position of the fundamental period of the structure.

This work is in agreement with similar studies carried out in RC structures [8, 9] and contributes to the understanding of the seismic motion simulation and its impact on structural response variability. Further investigation is deemed necessary, using a greater number of accelerograms and more reliable intensity measures (e.g., maximum spectral acceleration) for the selection and/or generation of the ground motions [10].

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