16th World Conference on Earthquake, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 1893
Registration Code: S-1461750543

A POSSIBLE REVISION OF THE CURRENT SEISMIC DESIGN PROCESS

M. Fasan(1), A. Magrin(2), C. Amadio(3), G.F. Panza(4) , F. Romanelli(5) , F. Vaccari(6) , S. Noè(7)

(1) *PhD Student, Department of Engineering and Architecture, University of Trieste, Italy, marco.fasan@phd.units.it*
(2) *olim Research Fellow, Department of Mathematics and Geosciences, University of Trieste, Italy, amagrin@units.it*
(3) *Professor, Department of Engineering and Architecture, University of Trieste, Italy, amadio@units.it*
(4) *olim Professor, Department of Mathematics and Geosciences, University of Trieste, Italy, giulianofpanza@fastwebnet.it*
China Earthquake Administration, Institute of Geophysics, Beijing, P.R. China
International Seismic Safety Organization (ISSO) - www.issoquake.org
(5) *Professor, Department of Mathematics and Geosciences, University of Trieste, Italy, romanel@units.it*
(6) *olim Research Fellow, Department of Mathematics and Geosciences, University of Trieste, Italy, fvaccari@me.com*
XeRiS - eXact lab Srl, Trieste, Italy, info@xeris.it
(7) *Professor, Department of Engineering and Architecture, University of Trieste, Italy, noe@units.it*

Abstract

We present a possible Performance Based Seismic Design (PBSD) process grounded in a scenario based definition of the seismic input. The proposed procedure aims to address the following considerations, arisen from the analysis of seismic phenomena: a) any structure at a given location, regardless of its importance, is subject to the same shaking as a result of a given earthquake, b) it is impossible to determine with precision when a future earthquake of a given intensity/magnitude will occur, c) insufficient data are available to develop reliable statistics for earthquakes. Following these considerations, the seismic input at a given site - determined on the basis of the seismic history and the seismogenic zones, nodes and faults - is defined using the Neo Deterministic Seismic Hazard Assessment. Two different analyses are carried out at different levels of detail. The first one (*Regional Scale Analysis*) provides the "*Maximum Deterministic Seismic Input*" as a response spectrum at the bedrock ($MDSI_{BD}$), similarly to what is proposed by the codes. The second one (*Site Specific Analysis*) takes the site effects into account, providing a site specific seismic input ($MDSI_{SS}$); This approach envelops uncertainties by means of a wide range of NDSHA simulations rather than quantifying them probabilistically and provides realistic site specific seismograms that could be used to run time history analysis even where no registrations are available. Reviewing the standard PBSD procedure, $MDSI_{SS}$ is always associated with the worst structural performance acceptable for a building (*Target Performance Level*). Thus the importance of the structure (risk category) is taken into account by changing the structural performance level to check, rather than to change the seismic input. The procedure is a generalized enhancement of what has already been applied for the seismic upgrading of several critical and essential buildings (e.g. schools) of the Trieste Province (Italy).

Keywords: performance based seismic design; seismic input; neo deterministic seismic hazard assessment; collapse prevention; MDSI;

1. Introduction

The "Performance-based seismic design" (PBSD) is the "coupling of expected performance level with expected levels of seismic ground motion" [1]. Structural performance levels are assessed in terms of damage occurring in a structure due to a given seismic input. These conventional performance levels are often defined as: Operational Limit (OL), Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) [2]. Usually, the earthquake associated to them is chosen as a function of a fixed probability of exceeding it (P_{EY}) in a range of time Y (reference average life), i.e. choosing a "Mean Return Period" P_R of the earthquake using the well-known relation P_R=-Y/ln(1-P_{EY}).

The shaking parameter (usually the spectral acceleration) related to the mean return period P_R is calculated using the Probabilistic Seismic Hazard Analysis approach (PSHA) [3]. Although adopted by the main international seismic codes, this method has fallacies as pointed out by several seismologists, statisticians and professionals [e.g. 4-8]. To name a few:

- earthquakes are considered as if they are independent events;
- the number of registered strong earthquakes is too limited to constrain the probabilities of their occurrence;
- the probability of exceedance - a dimensionless quantity - is confused with the rate of exceedance - a frequency;
- validation of the results is not possible (it would take thousands of years to develop a reliable statistics);
- the choice of the reference average life and the probability of exceedance are rather arbitrary;
- the use of low probabilities of exceedance leads to unrealistic values of shaking as a result.

It has to be highlighted that the scientific community is divided on this issue and several papers have been written to support PSHA [e.g. 9, 10]. In this paper we tackle the problem of the Seismic Hazard Assessment for a Peformance-Based Seismic Design (PBSD) with an engineering perspective showing that PSHA is not suitable to check some performance levels.

Usually seismic codes change the reference average life Y as a function of the importance of the structure (risk category) that is related with the (hypothetical) consequences of its failure (the more dangerous the consequences, the longer the average life). The probability of exceedance P_{EY} is related with the structural performance level to check: the lesser the acceptable damage, the higher the probability of exceedance. Let us consider the following *exemplum fictum*: Using the Italian Building Code (NTC08) [11] a residential building should be designed to reach the Collapse Prevention Level for an earthquake with P_R=975 years (i.e. a response spectrum whose accelerations are supposed to have P_{EY}=5%/Y=50 years). On the other side, an Essential Building (e.g. a school) should be designed to reach this level when impacted by an earthquake with P_R=1462 years (i.e. P_{EY}=5%/Y=75 years). Focusing on the Collapse Prevention Level this means that if an earthquake consistent with a P_R=1462 years happens, the residential building designed in accordance with the Italian Building Code could collapse. If an earthquake with P_R=2000 years happens, even a school would fall down. This procedure is usually justified on the basis of an economic assessment of the cost to build structures in seismic areas. However, the standard breakdown of the overall cost of a building is: 8-18% for structural components, 48-62% for non-structural components and 20-44% for contents [12]. Then costs optimization using a probabilistic value of ground motions when evaluating the Collapse Prevention Level appears to be unreasonable, at least for three reasons: (a) the fallacy of the return period concept and (b) the benefits (reduction of costs) due to a PSHA decrease of ground motion involve a very small percentage of the overall cost (the structural components) (c) it does not take into account the post-earthquake recovery costs. When assessing the Collapse Prevention Level, the situation that could involve the loss of the structure is dealt with. Given the fact that an engineer cannot control the earthquakes phenomena (so far nobody can tell with precision when and where an earthquake will happen) but can govern the building performance through the designing procedure, the least we can do is to use an "upper-bound" ground motion to design buildings against the collapse. As a rule, an "upper-bound" ground motion should be used to assess every structural performance that involves the highest level of damage eligible for the building under design (e.g. Collapse Prevention Level for Ordinary Buildings or Immediate Occupancy for Hazardous Buildings).

Following PSHA, the probability of exceedance could be reduced but the use of low probabilities results in unreasonable values of shaking [13-15]. As a consequence, an approach different from PSHA is needed [6]. A possible solution is to adopt the Deterministic Seismic Hazard Assessment (DSHA) approach, which is usually a scenario based approach where the hazard is chosen as the maximum ground motion of a set of individual earthquakes (magnitude and distance) that could happen at a site. The reason for using deterministic spectral accelerations, as written in the NEHRP Recommended Seismic Provisions for New Buildings (FEMA P-750) [16], is that "*deterministic ground motions provide a reasonable and practical upper-bound to design ground motions*". Accordingly, in this work the acceleration response spectrum, called *Maximum Deterministic Seismic Input* (MDSI), at a given site is defined by means of the envelope obtained from a large number of realistic ground motions simulated using the Neo Deterministic Seismic Hazard Assessment (NDSHA); for a recent review of the procedure see Panza et al. [17].

It is worth noting that some seismic codes (e.g. ASCE 7 [18]) already use the 84th percentile spectral values determined with standard DSHA to cap PSHA in areas close to active faults. The reason is that "*the probabilistic analysis had flaws that cannot be corrected with our current state of knowledge*" [19]. So *de facto* buildings have been designed using deterministic values of ground motion in all the major seismic zones of the U.S. even if these values seem to be the result of a probabilistic analysis.

2. Maximum Deterministic Seismic Input - MDSI

MDSI is calculated by means of a large number of NDSHA simulations. NDSHA avoids the use of Ground Motion Prediction Equations (GMPE). Instead, it is a physics-based approach, which supplies realistic seismograms of earthquakes that can occur at a given site [17, 20]. It relies on the available knowledge about the potential sources and on the properties of the media crossed by earthquake waves. MDSI can be defined both as a response spectrum or as a set of seismograms and it is independent from the arbitrary choice of the reference average life and the probability of exceedance. In the next sub-sections, its application to the Italian territory is briefly described. For further detail see Fasan et al. [21].

2.1 Regional Scale Analysis (RSA)

At regional scale, the seismic sources are discretized into cells spaced 0.2°x0.2° and distributed within the ZS9 seismogenic zones [22] and nodes [23-25]. The magnitude value associated with each source is set equal to the maximum among that of the seismogenic nodes, of the events reported in the catalogs of the Italian, Slovenian and Croatian earthquakes [26-28] and a minimum of 5. These sources are modelled as double-couple size-and-time scaled point sources (STSPS) [17, 21] with a focal mechanism consistent with the seismogenic zone. The properties of the inelastic media crossed by earthquake waves is set equal to those given by Brandmayr et al. [29]. To account for the spatial uncertainties of epicentres a smoothing procedure is applied according to Panza et al. [20]. The stochastic nature of the source rupture process is taken into account by means of the PULSYN algorithm [30]. Differently from Fasan et al. [21] the number of rupture models generated for each source has been increased from 100 to 300, a value that has been found more suitable to stabilize the solution at the percentiles of interest. Synthetic seismograms are then computed using the modal summation technique [31] in far field conditions and the matrix impedance method [32] in near field conditions. The calculation is done at each node of a 0.2°x0.2° grid over the Italian territory at a cutoff frequency of 10 Hz.

The resultant response spectrum has been found to be a meaningful parameter for structural design [16]. Therefore, the MDSI response spectrum is calculated as the resultant (here called "*Res*", i.e. Maximum Direction, see Fig. 1) rather than the maximum between the components of ground motion in the horizontal plane (here called "*Max_xy*"), which are dependent on the reference system (see Fig. 5a and Fig. 7).

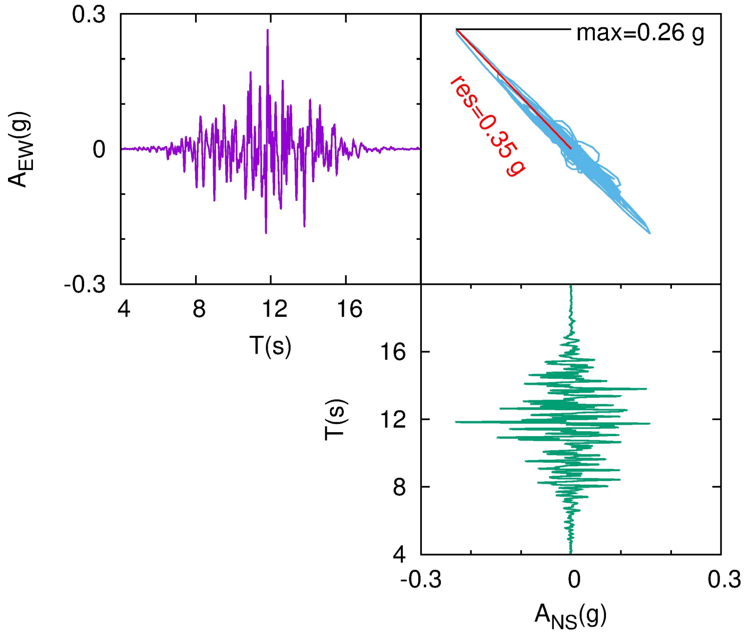


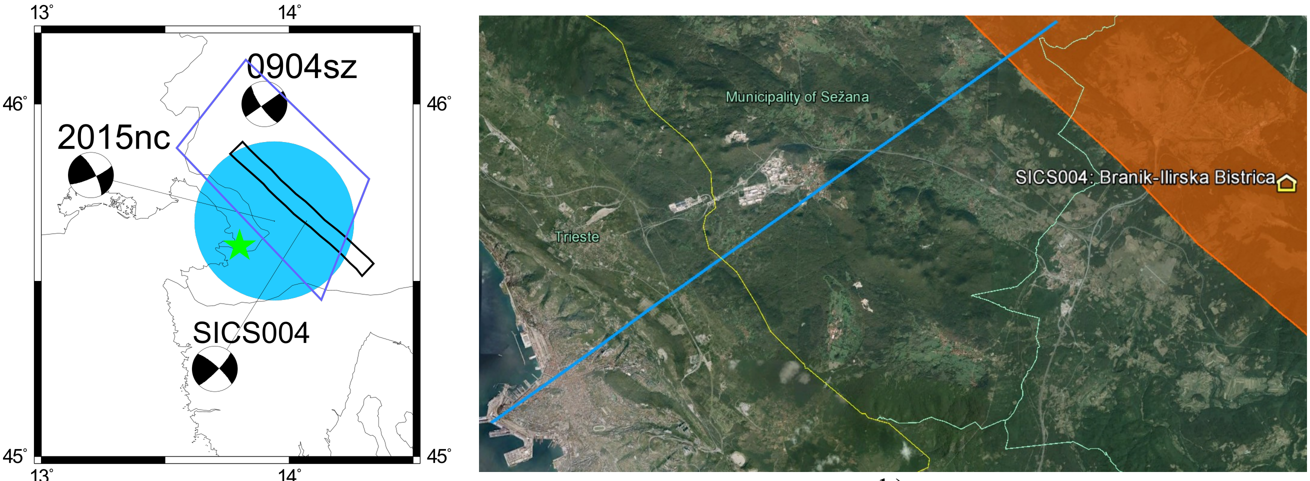
Fig. 1 – Definition of the resultant response spectrum

For each source within 150 km from the site of interest, the resultant response spectrum is computed for each NDSHA simulation (i.e. 300 for every source). $MDSI_{BD}$ should be set equal to their envelope (300 x number of sources). Alternatively, $MDSI_{BD}$ could be set equal to 84th percentile, as suggest by ASCE 7 [18] for the standard deterministic analysis, or equal to the median as done by several codes for the probabilistic analysis (e.g. Italian Building Code [11]), or equal to the 95th percentile, as done by codes for dead loads (Fig. 4).

A Regional Scale Analysis (RSA) provides the Maximum Deterministic Seismic Input at bedrock ($MDSI_{BD}$), without considering the site effects. The $MDSI_{BD}$ could be used, if corrected by means of the standard approximate soil coefficients, for a standard design for ordinary buildings and only for a preliminary design for hazardous buildings. Furthermore, a RSA allows for the identification of the sources responsible of the highest hazard at the bedrock, similarly to the disaggregation concept used in PSHA (Fig. 2a).

2.2 Site Specific Analysis (SSA)

The second step consists of a detailed Site Specific Analysis (SSA) which takes into account the site structural heterogeneities (whether topographical or due to the presence of soft-sedimentary soil) by adding a local finite difference model (Fig. 6a). The SSA allows determining the Maximum Deterministic Seismic Site Specific Input ($MDSI_{SS}$). In a Site Specific Analysis, the wavefield generated by the modal summation technique is introduced into the mesh that defines the local laterally heterogeneous area and it is propagated according to the finite-differences scheme shown in Fig. 6b [33]. The procedure to define the MDSI response spectrum is the same shown in section 2.1. However, to reduce the time costs the analysis is run only for the most hazardous sources (for the path source to site) for the site of interests as found by a Regional Scale Analysis (Fig. 2a). It is worth noting that a SSA supplies realistic accelerograms since MDSI relies on the computation of synthetic seismograms. These accelerograms are suitable to run accurate nonlinear dynamic analysis since they are consistent with the maximum magnitude, focal mechanism, epicentral distance and soil conditions of the site of interest.



a) b)

Fig. 2 – a) Controlling seismic sources resulting from a RSA; b) Source to site path used in the SSA

3. Performance Based Seismic Design

As previously said, structural performance levels are function of the damage (usually defined in terms of acceptable storey drift or acceptable rotation of plastic hinges) that is accepted to occur in the elements of a building when subjected to a certain level of ground motion. While the Collapse Prevention Level refers to a specific physical phenomenon and thus can be recognizable (collapse), the other performances represent exclusively a conventional state of damage. The definition of the ground motion used to check if a performance level has been reached is a crucial step. In the introduction it has been explained how it is unreasonable to design against the collapse using a probabilistic seismic input, in this section we go further. Imagine having to design a building of strategic importance, like a hospital. In case an earthquake occurs the hospital must be able to receive and treat injured. Therefore, is it reasonable to accept that it may be unusable after an earthquake? We believe not. In light of the considerations made so far (moved by common sense) we propose to identify, according to the importance of the structure, the Target Performance Level (TPL) that is the highest level of damage acceptable for the building. Achieving this level of performance will always be verified using the Maximum Deterministic Seismic Input. In this way, the seismic input used to check the TPL becomes independent from the choice of the reference average life and the probability of exceedance which are rather arbitrary. It is clear that once the TPL is chosen, levels of performance that involve a lower percentage of damage assume a minor importance in terms of potential adverse consequences. These levels of performance are defined Lower Performance Levels (LPLs). By definition of LPLs, it is acceptable that they may be exceeded during the life of the structure, as they involve less damage than TPL. Consequently, the acceleration response spectra associated with them must be less than MDSI (which should be a reasonable upper limit) and, given the conventionality of this procedure, their choice is completely arbitrary, therefore not unique. In a first approximation, the use of probabilistic values may be acceptable from an engineering perspective even though it is based on the non-physically routed concept of return period [34]. Alternatively and equally arbitrarily, such levels could be defined as a fraction of $MDSI_{SS}$ response spectra (for example 2/3 of $MDSI_{SS}$ for medium seismic input level and 2/5 of $MDSI_{SS}$ for low seismic input level). The procedure proposed is summed up in Fig. 3 whereby two values are suggested as examples. It can be summarized in the following three steps: 1) identification of the Risk Category of the building (e.g. Ordinary Building, Essential Building or Hazardous Building); 2) as a consequence of step 1, choice of the Target Performance Level associated with the $MDSI_{SS}$ response spectrum; 3) as a consequence of step 2, choice of the Lower Performance Levels and the associated ground motions.

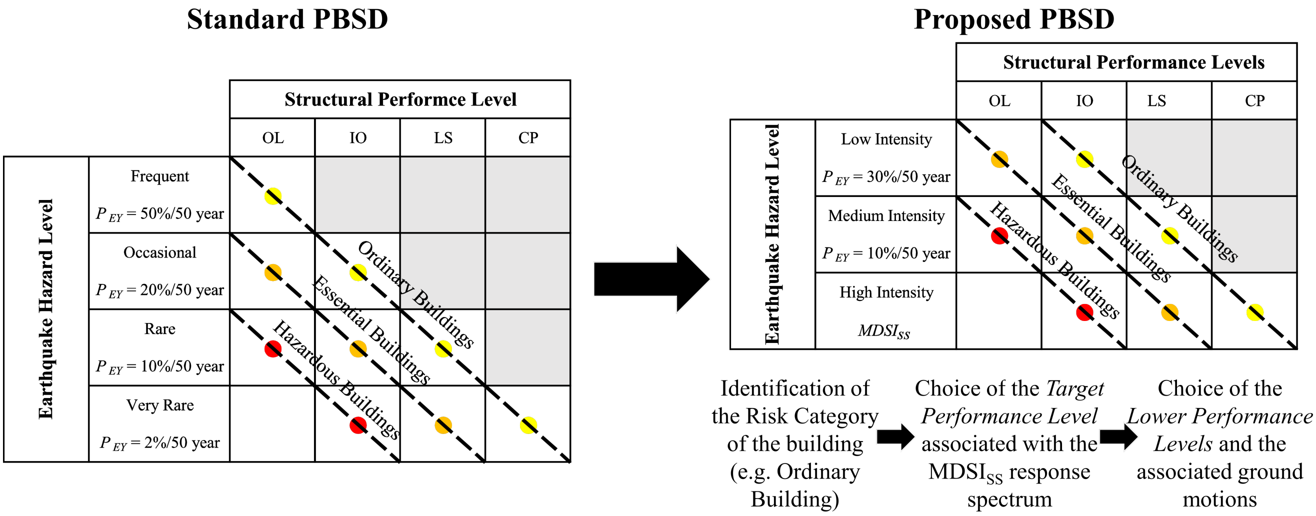


Fig. 3 – Proposed PBSD procedure considering the MDSI

According to this procedure, a residential building should be designed at the CP level for the $MDSI_{SS}$, while at the LS and IO levels a reduced seismic input could be used. An essential building could be designed for the $MDSI_{SS}$ at the LS, while the IO and OL levels should be assessed with a lower value of the seismic input. It is worth noting that the uncertainties, both structural and related to the seismic input, are such that it is impossible to predict exactly the seismic behaviour of a structure. This is the main aspect that should lead engineers to design by means of envelopes instead of using probabilistic calculations. The procedure proposed in Fig. 3 should be used as a minimum requested performance to assess the building during its design stage.

4. Examples

As an example, the procedure has been applied to the city of Trieste (Italy). Both resultant ("*Res*") and maximum between the orthogonal ground motion components ("*Max_xy*") have been calculated. This has been done to show the dependency of the *Max_xy* response spectrum from the choice of the reference system. Furthermore, it is necessary for a comparison with the response spectra of the Italian Building Code that represents the *Max_xy* and not the resultant.

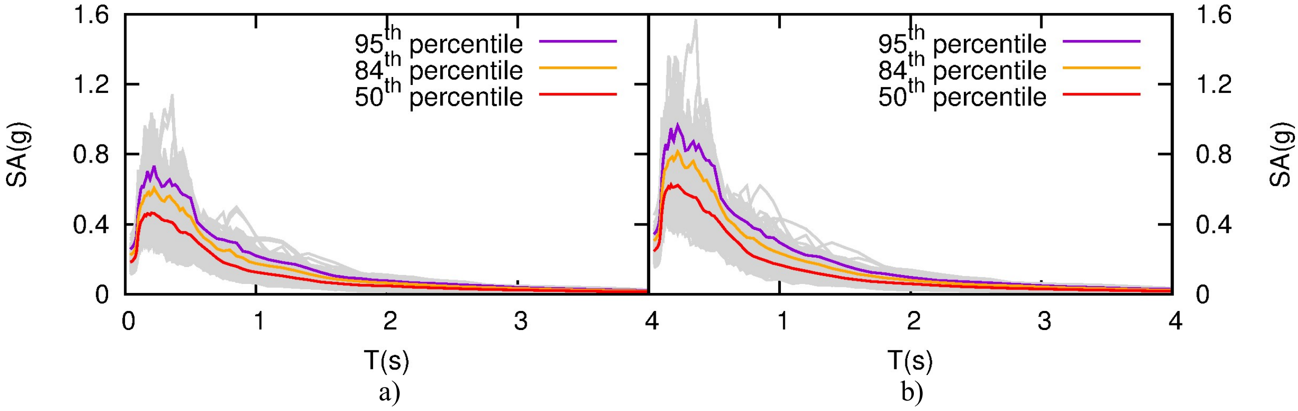
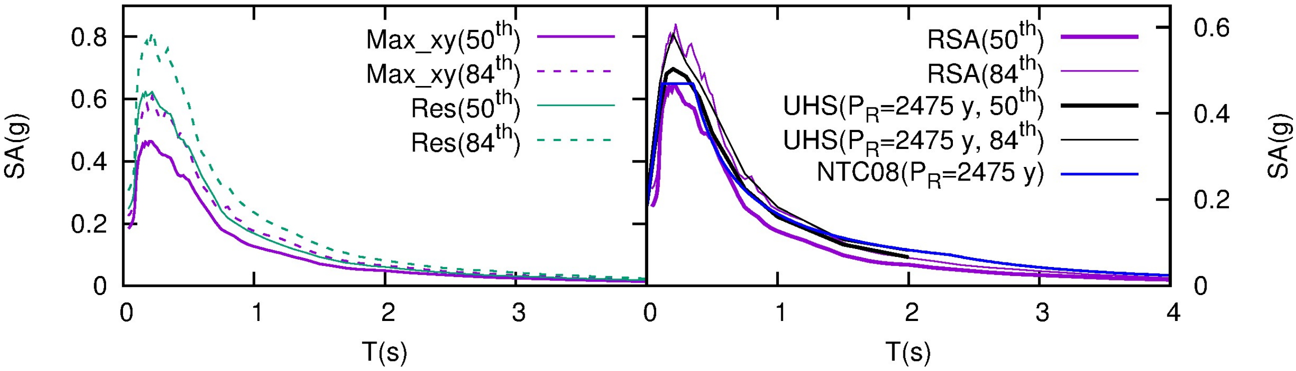


Fig. 4 – Variability of response spectra shape at the site of interests: a) *Max_xy*; b) *Res*

As previously said, the first step consists in a Regional Scale Analysis. Fig. 4a and 4b show the variability of the spectral accelerations respectively for the *Max_xy* and *Res* response spectra, as resulting from the Regional Scale Analysis. This variability is due to the contribution of different sources, magnitudes, focal mechanisms and rupture process. The MDSI should be defined as their envelope. In fact, MDSI represent a sort of Uniform Hazard Spectra (UHS), where the hazard is identified by the maximum magnitude expected for every potential source that could affect the sites of interest.

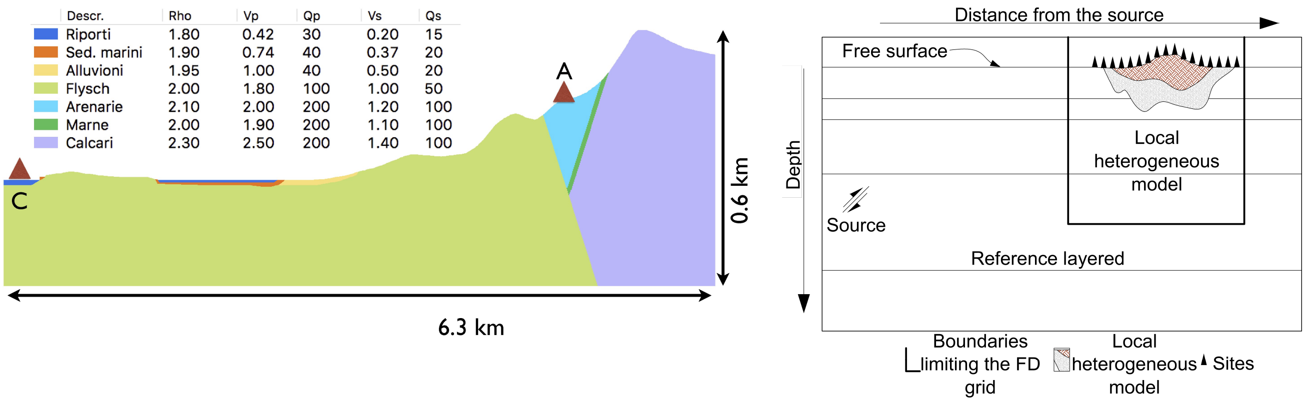
A Regional Scale Analysis allows for the identification of the seismic input at the bedrock (i.e. soil class "A" as per Italian Building Code). The Uniform Hazard Spectra (UHS) given by the Italian seismic hazard map [35] represent the *Max_xy* response spectra at bedrock for a given probability of exceedance. In Fig. 5b it is shown a comparison between the *Max_xy* response spectra resulting from the Regional Scale Analysis (50^{th} and 84^{th} percentile) and the Uniform Hazard Spectra of the Italian seismic hazard map for a return period P_R=2475 years (50^{th} and 84^{th} percentile). The return period of 2475 years is the highest value used to calculate the Italian seismic hazard map. Consequently, it is assumed that for this value of probability of exceedance (2%/50years) the resulting spectral accelerations are still realistic. As it can be seen MDSI caps the spectral acceleration when using small values of probability of exceedance. In addition, in Fig. 5b the response spectrum of the Italian Building Code for a return period P_R of 2475 years is reported too. This response spectrum represents a "code fit" of the median UHS.



a) b)

Fig. 5 – a) Comparison between *Res* and *Max_xy* (RSA); b) Comparison between *Max_xy* resulting from a RSA and the Italian building code response spectra

As a second step a Site Specific Analysis has been performed. To take into account the effects of soil and topographic characteristics on both vertical and horizontal components of the earthquake ground motion, a laterally heterogeneous profile representative of the local conditions has been composed (Fig. 6a), using data from literature and field work.

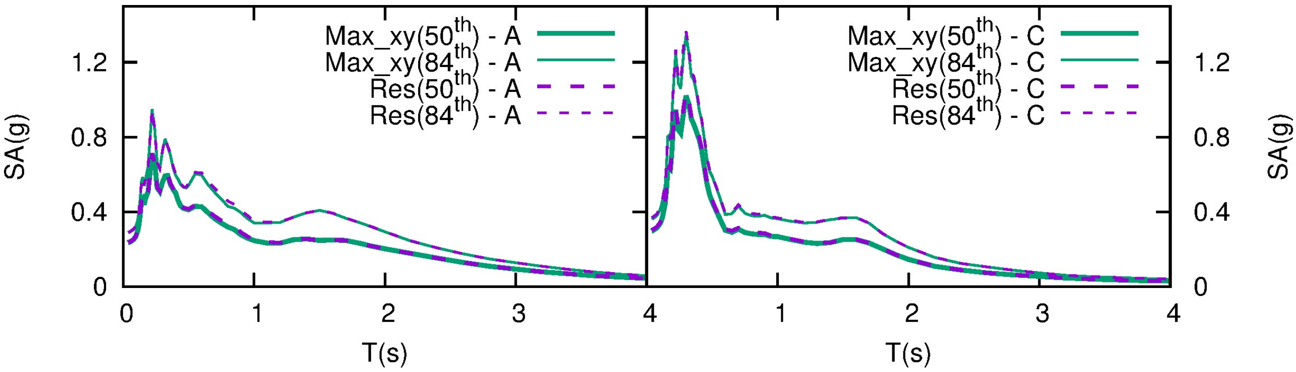


a) b)

Fig. 6 – a) Profile and sites of interest used for the SSA; b) Schematic diagram of the hybrid method (SSA)

To save computer time costs, the scenario used for the SSA has been chosen from the disaggregation of the $MDSI_{BD}$ response spectrum calculated with the Regional Scale Analysis (i.e. the source-distance combination that gives the largest spectral acceleration at periods of interest). The controlling event, for the range of periods from 0 to 4 seconds, has been found to be a magnitude 6.5 earthquake at a distance between 15 to 20 kilometres (Fig. 2). This scenario is coincident with the seismic potential of the Branik-Ilirska Bistrica fault (SICS004) [36] and falls within a seimogenic node (Fig. 2). The assumed focal mechanism parameters are: depth 10 km, Strike 281°, Dip 79°, Rake 16°, in accordance with the local dominant tectonic style [36].

Two sites have been selected as representative for the analysis of the results of the entire study: A representative of a soil type “A” ($V_{S,30} \geq 800$ m/s) and C representative of a soil type “C” ($180 < V_{S,30} \leq 360$ m/s) as defined by the Italian Building Code. Fig. 7a and Fig. 7b represent a comparison between the *Max_xy* response spectrum (50th and 84th percentile) and the resultant response spectrum (50th and 84th percentile) for the aforementioned sites resulting from the Site Specific Analysis. The same comparison has been done in Fig. 5a for the response spectra resulting from the Regional Scale Analysis. As it can be seen, the ratio between the two response spectra varies from about 1.4 in the RSA (Fig. 5a) to about 1 in the SSA (Fig. 7) where *Res* and *Max_xy* are almost overlapped. This comparison confirms how the *Max_xy* response spectrum is dependent on the orientation of the reference system and therefore it isn’t a valuable tool for seismic design.



a) b)

Fig. 7 – Comparison between *Res* and *Max_xy* (SSA): a) Site A; b) Site C

Fig. 8a and Fig. 8b show a comparison between the maximum response spectrum provided by the Italian code (P_R=2475 years), the input associated with the code to the Collapse Prevention Level for a standard residential building (P_R=975 years) and the response spectrum adopted by the code for a standard design associated with the Life Safety Level (P_R=475 years). As it can be seen, both for the site A and for the site C the structural lateral heterogeneities has a strong effect on the shape and amplitude of the response spectrum. In particular, the use of standard soil coefficient provided by codes can strongly underestimate the local amplification. In fact, even though the median UHS is very close to the median *Max_xy* response spectrum resulting from the Regional Scale Analysis (i.e. the response spectrum at the bedrock without considering local soil and topographic conditions) (Fig. 5b), the differences increase when adopting a Site Specific Analysis. The Site Specific Analysis, differently from the standard procedure used to calculate the soil amplification effects (e.g. H/V method), takes into account the relevance of the incidence angle of the radiated wavefield, consistently with the tensor nature of earthquake ground motion [17].

As far as the PBSD design is concerned, from the analysis of the previous comparisons it is evident how the proposed procedure represents somehow an increment in the design criteria. However, as long as the design of residential buildings is concerned the $MDSI_{SS}$ response spectrum is associated with the Collapse Prevention Level (Fig. 3). Since the behaviour factor is usually related with the check of the Life Safety Level this means a higher behaviour factor could be used to check the Collapse Prevention Level (because it involves a higher percentage of damage). This increasing could vary between 1.3 to 1.5 times the standard behaviour factors.

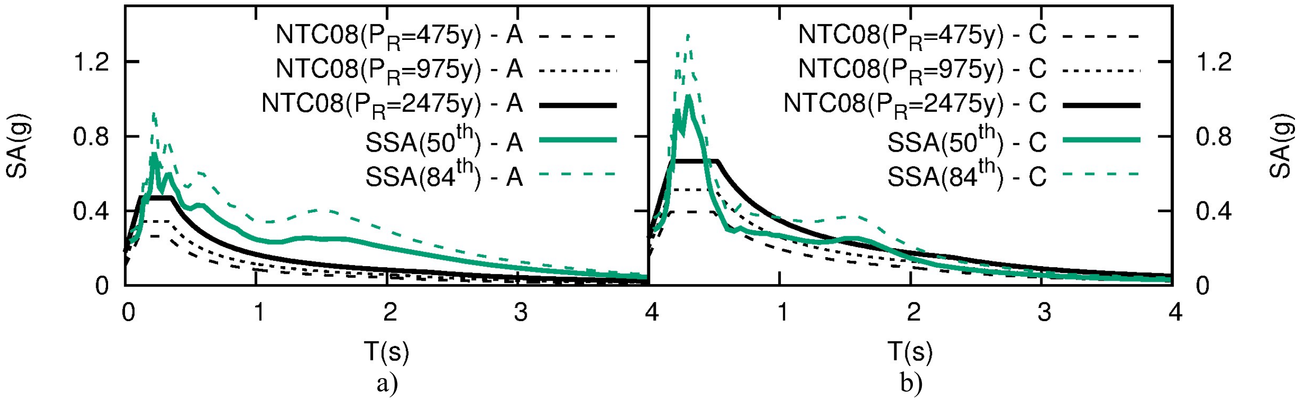


Fig. 8 – Comparison between *Max_xy* resulting from a SSA and the Italian code response spectra

5. Conclusions

A new design procedure has been proposed, whereby the seismic input is calculated by means of envelopes on a wide range of Neo Deterministic Seismic Hazard Assessment (NDSHA) simulations. The "Maximum Deterministic Seismic Input" (MDSI) has been defined at bedrock ($MDSI_{BD}$) and considering site specific characteristics ($MDSI_{SS}$). $MDSI_{SS}$ is always associated with structural performance that involves the highest level of damage acceptable for a building, called Target Performance Level (TPL). The importance of the structure can be taken into account increasing or decreasing the maximum acceptable level of damage, while keeping unchanged the seismic input that, as required by basic physics, becomes independent from the choice of the reference average life and the probability of exceedance, which are rather arbitrary. The performance levels that involve less percentage of damage with respect to the TPL are called Lower Performance Levels (LPL). Given the conventional nature of LPLs, from an engineering perspective the seismic input level associated with them can be found either using probabilistic values (even though they are based on the non-physically routed concept of return period) or reducing the $MDSI_{SS}$ spectral accelerations. In a nutshell, on account of the large uncertainties about the structural properties of the build environment, the detailed characteristics of seismic input and to avoid the illusory idea of optimizing costs probabilistically reducing the earthquake ground shaking, a reliable approach to be followed is believed to be that based on the use of the MDSI response spectrum, that certainly represents a lower limit of the worst possible case consistent with present day knowledge. The proposed procedure should be used as a minimum requested performance.

4. Acknowledgements

This paper has been supported by the "*Fondazione Cassa di Risparmio di Gorizia*" and FRA 2014 - University of Trieste.

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