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NON LINEAR PUSH OVER ASSESSMENT OF HISTORIC BUILDINGS IN ISTANBUL TO DEFINE VULNERABILITY FUNCTIONS.

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1. INTRODUCTION

The Government of Turkey (GOT) and the International Bank for Reconstruction and Development (World Bank) have agreed upon a loan to implement the Istanbul Seismic Risk Mitigation and Emergency Preparedness Project (ISMEP) objectives of which are to improve the city of Istanbul's preparedness for a potential earthquake. Within the framework of Component B, subcomponent B.3 "Risk Assessment of Cultural Heritage Buildings" supports a technical assistance program to address the vulnerability of cultural assets, specifically buildings with global cultural heritage value. The project has five components, from literature to field survey, to vulnerability and risk assessment of 170 buildings and the development of a GIS database. The locations of the major portion of these buildings are shown in Fig.1. The paper proposed here discusses in details the methodology adopted to estimate the expected performance of some of the buildings under analysis.
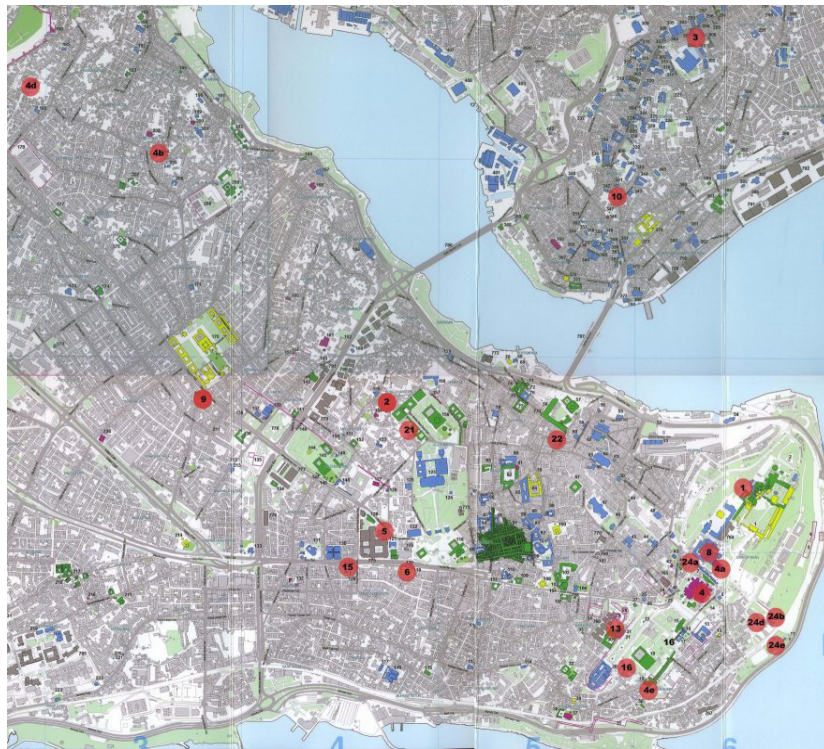

Figure 1: Locations of the major number of historical buildings evaluated in this study

Specifically the choice of the most appropriate earthquake scenario is first discussed then the methodology used for assessing the effects of local site conditions in earthquake performance of selected cultural heritage buildings is presented. The purpose is to estimate the earthquake characteristics on the ground surface based on the earthquake characteristics on the engineering bedrock outcrop obtained from the probabilistic and deterministic hazard studies conducted by Erdik et al. (2007).

The site specific elastic design spectra for each site are then further manipulated to obtain site specific non linear displacement spectra, so that these can be directly compared with capacity curves for the buildings obtained by using plasticity based limit state analysis. In particular the procedure for obtaining the bilinear curves is presented. A discussion on the choice of the most appropriate level of ductility and the equivalent reduction coefficient conclude the paper.

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2. CHOICE OF EARTHQUAKE SCENARIO

Procedures to assess the vulnerability of existing and historic buildings are usually classified in relation to the dimension of the sample for which the vulnerability is considered and the number and detail of the structural parameters used to carry out the analysis (Bernardini, 1999, Coburn & Spence 2002). Typically a larger size of sample yields a smaller number of parameters and vice versa. Conversely studies move from purely statistical approaches for large samples to detailed structural analysis when a few buildings are considered. The project illustrated here however presented a specific challenge, as the number of buildings to be considered is relatively large, close to 200, while due to the nature of the sample is very difficult to identify common typologies and generalize behaviours within it, reducing the analysis to the choice of a small number of parameters. At the same time given the time frame of the project and its resources was unconceivable to run specific level 3 or above assessment analysis for each of the buildings. It was therefore decided to carry out a quantitative assessment using a numerical approach based on failure mechanisms and limit state analysis, called FaMIVE, whose robustness and reliability has already been proven in past studies for more homogeneous samples of historic masonry buildings (D'Ayala, Speranza 2003). This type of approach is intermediate between a first level of assessment based on scoring of a modest number of parameters, which are considered causes of damage to the buildings, and more sophisticated nonlinear analysis procedures which are lengthy and not necessarily feasible without very detailed data. The benefit of the FAMIVE method rests on the fact that although the calculations are relatively simple and use a small number of parameters easily obtainable from qualitative surveys, it produces a measure of vulnerability based on structural behaviour, expressed in terms of resultant capacity and displacement and hence directly comparable with demand spectra.
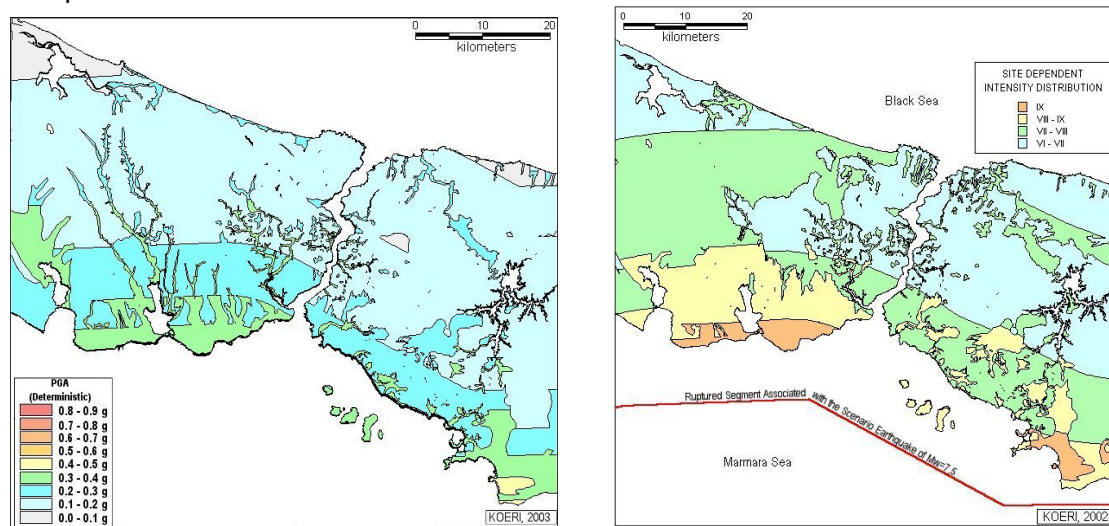

Figure.2: a)Site dependent deterministic peak ground accelerations, b) Site dependent deterministic intensity distribution (Erdik et al., 2007)

Once the vulnerability is quantified the risk of a given damage level associated with a given earthquake scenario can be calculated. Considering that the FaMIVE procedure is based on a deterministic approach and that the sample of buildings analysed is too small and too inhomogeneous for a rigorous and robust probabilistic treatment, a deterministic scenario for the rupture of the Marmara Fault is chosen. Specifically It is assumed that, based on the segmentation model developed, the scenario earthquake would occur on the unruptured segments of the Main Marmara Fault producing an Mw = 7.5 event. Compilation and interpretation of topographic, geologic and geotechnical data and the selection of the appropriate attenuation and site response models constitute the remaining main inputs of the earthquake hazard assessment. The distribution of the site dependent deterministic peak ground acceleration is presented in Figure.Fig. 2a and Fig.2b represents the intensity distribution resulting from the scenario earthquake. (Erdik et al., 2007)

3. EFFECT OF LOCAL SITE CONDITIONS ON DEFINITION OF ELASTIC SITE SPECTRA

Local geological and geotechnical site conditions were evaluated using available reports on geotechnical and geophysical investigations conducted close to the cultural heritage building sites. Most of the information came from previously conducted research, geotechnical investigations and few private geotechnical projects on the study area. Consequently, it should be noted that the results obtained for this study include some uncertainties concerning the compiled geotechnical data. Using all the relevant information, representative soil profiles with shear wave velocity profiles extending down to the engineering bedrock with estimated shear

wave velocity of 750m/s are determined for all building sites. Each soil profile contains information on soil stratification, depth of bedrock, ground water elevation, thickness and shear wave velocity with depth.

Site specific response analysis was carried out at each building site using representative soil profiles and regional seismic hazard parameters. Regional seismic hazard was evaluated using deterministic as well as probabilistic time dependent Poisson model by Erdik et al. (2007). Seismic hazard parameters in terms of PGA and spectral accelerations at periods T=0.2s and T=1s on the engineering bedrock outcrop were assigned at each site. 1-D equivalent-linear model (Shake91, Idriss and Sun, 1991) was used to conduct site response analyses.

Twenty four real acceleration time histories that are compatible with the earthquake hazard assessment in terms of probable magnitude, distance and fault mechanism were selected as the input rock outcrop motion (Ansal and Tönük, 2007). The main purpose was to account for the variability arising from the differences observed in the input motion acceleration time histories. The selected input acceleration time histories were then scaled with respect to the peak ground horizontal accelerations obtained from earthquake hazard study at each site and were used as outcrop input motions for site response analyses (Ansal et al., 2006).

Two approaches were adopted to evaluate site-specific free field earthquake characteristics on the ground surface. In the first approach 1-D site response analysis using Shake91 (Idriss and Sun, 1992) was conducted and average from all 24 analyses at each site was computed to determine average peak ground acceleration (PGA), average peak ground velocity (PGV) and average elastic acceleration response spectrum on the ground surface at each site. In the second approach empirical relationship proposed by Borcherdt (1994) was employed to determine peak spectral accelerations corresponding to 0.2s and 1s using equivalent (average) shear wave velocities at each site.
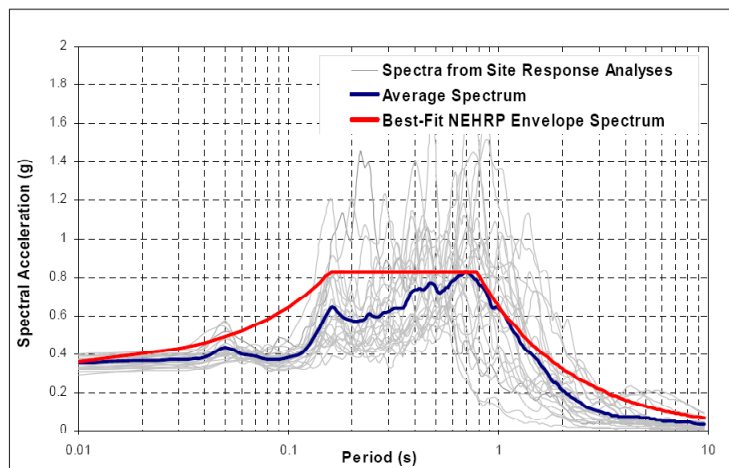

Figure 3: An example best envelop NEHRP spectrum fitted to average acceleration response spectrum calculated from site response analyses.

In order to be used for seismic vulnerability assessments of buildings two more parameters; site-specific short period (corresponding to 0.2s) and long period (corresponding to 1s) spectral acceleration values were evaluated. The average acceleration response spectrum obtained for each site from site response analyses were used to determine spectral accelerations for the short period (Ss) and for the long period (S1). An approach was adopted to determine the best fit envelope to the calculated average acceleration response spectra (Ansal et al. 2006). All the requirements of the NEHRP design spectra were applied in obtaining the short (Ss) and long (S1) period spectral ordinates. The two independent variables in the developed optimization algorithm were (Ss) and (S1). An example of the best fit NEHRP envelope obtained by this approach with respect to average elastic acceleration response spectra is shown in Fig. 3. The NEHRP design spectrum is preferred because of its flexibility in defining short period spectral accelerations and for vulnerability assessment of the building stock (Erdik and Fahjan 2005).

As shown in Fig. 4 even though the range of the peak ground accelerations on the engineering bedrock outcrop (0.234g - 0.348g) is relatively narrow, the calculated peak ground accelerations on the ground surface have much larger range (0.171g- 0.595g) indicating the importance of site conditions. In the case of deterministic earthquake scenario, the probable fault rupture will take place in the near field most likely within 20-50km distance. As a result the calculated peak ground acceleration levels are relatively high.
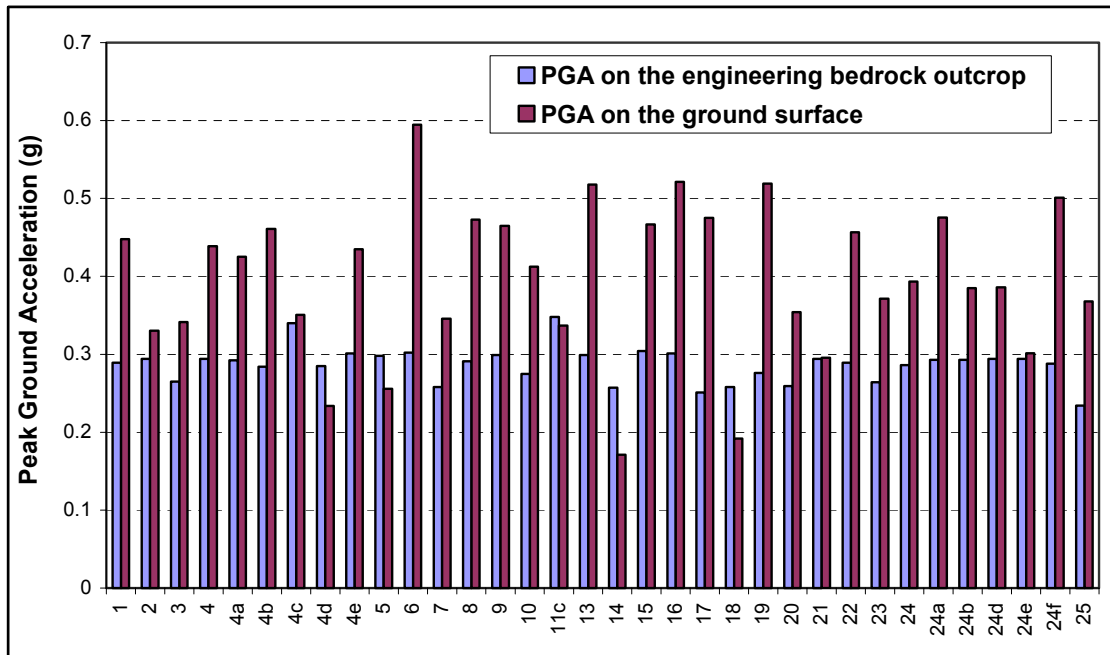
Figure 4: Peak ground accelerations on the engineering bedrock outcrop (Vs ≥750m/s) and on the ground surface calculated by site response analysis

4. CAPACITY CURVES FOR HISTORIC BUILDINGS

The first step in any analytical vulnerability assessment is to categorise the buildings, by structural system, construction material, but also layout, typology and period of construction. It is difficult to reduce the ISMEP sample to only few typologies as they vary greatly by all criteria identified above. However the large majority are built in masonry and have wall box behaviour, which lend itself to be analysed by using the FaMIVE approach.

Exceptions are represented by a modest number of timber clad timber frame buildings supported on masonry basement walls. As the lower part of the structures is made of masonry and at this level of assessment without a detailed survey of the connection of the timber superstructure is difficult to quantify reliably the effectiveness of the frame action of the timber portion, it is assumed that the overall structures behave in a manner similar to a masonry building as far as the triggering of the mechanism is concerned, while in the post peak phase the timber structure will be able to show greater ductility than typical masonry structures. Hence FaMIVE has also been applied to these classes of buildings. A second exception is constituted by a few gigantic load bearing masonry structures as in city walls and defensive constructions: these structures often lacking a masonry box layout are not suitable for FaMIVE and they have been analysed using a number of global indicator such as total shear base capacity and walls or columns slenderness, following guidelines from EC8. Finally the FAMIVE procedure was interfaced with a procedure able to calculate the trust produced by vaulted structures on the bearing walls in presence of gravity loads and lateral action, and referred to in the following as VULVAULT, developed at purpose for the project.

4.1. Definition of quantitative vulnerability judgement for each building

The FAMIVE procedure is based on the lower bound approach of limit state analysis and identifies all possible mechanisms that can occur for an elevation in a building given its connections to other elevations and horizontal structures and the layout of openings (D'Ayala, Speranza 2003). Among all possible mechanisms for each wall the one that has the worst combination of load factor and damage extension, is considered as defining the vulnerability of the wall. These results can be presented in terms of synthetic plan maps in which the values of collapse factor, type of mechanism and extent of wall failure are plotted, or can be further elaborated to give an overall judgement of the vulnerability of the building as shown in Fig. 5.

To include the results in the GIS system for each building in each unit a global, prevalent and local vulnerability judgement are provided with the following definitions:

- *Global vulnerability* refers to weaknesses which are either distributed in various and several parts of the building or characterise a most relevant part/section of it, such as more than one entire façade and

involving in their collapse floors and roofs, so that possible damage would affect a large portion of the building. In terms of collapse load factor and class attribution the global vulnerability is calculated as the weighted average of the collapse load factors and class of each of the critical collapse load factor for each elevation, while as far as the mechanism is concerned the most common is chosen as the building mechanism, or the one associated with the highest extension.

- *Prevalent vulnerability* (class, load factor, failure mechanism) is neither the worst nor the average vulnerability, but the most significant i.e. that which best characterise or refer to the possible most significant damage, i.e. is either the most common or the one with the most serious consequences. In this respect in some cases global and prevalent might coincide in terms of class or mechanism, but it is unlikely in terms of collapse load factor.

- *Local vulnerability* refers to the most vulnerable element/section/part of the building where possible damage affects a limited part of the building. It highlights a localised weakness where possible damage can occur for considerably lower collapse load factor than the rest of the building. Also usually these are vulnerability that can be easily identified and mitigated with ordinary, conventional strengthening. For instance, mechanisms G and M (see Fig.1) are usually considered as local.
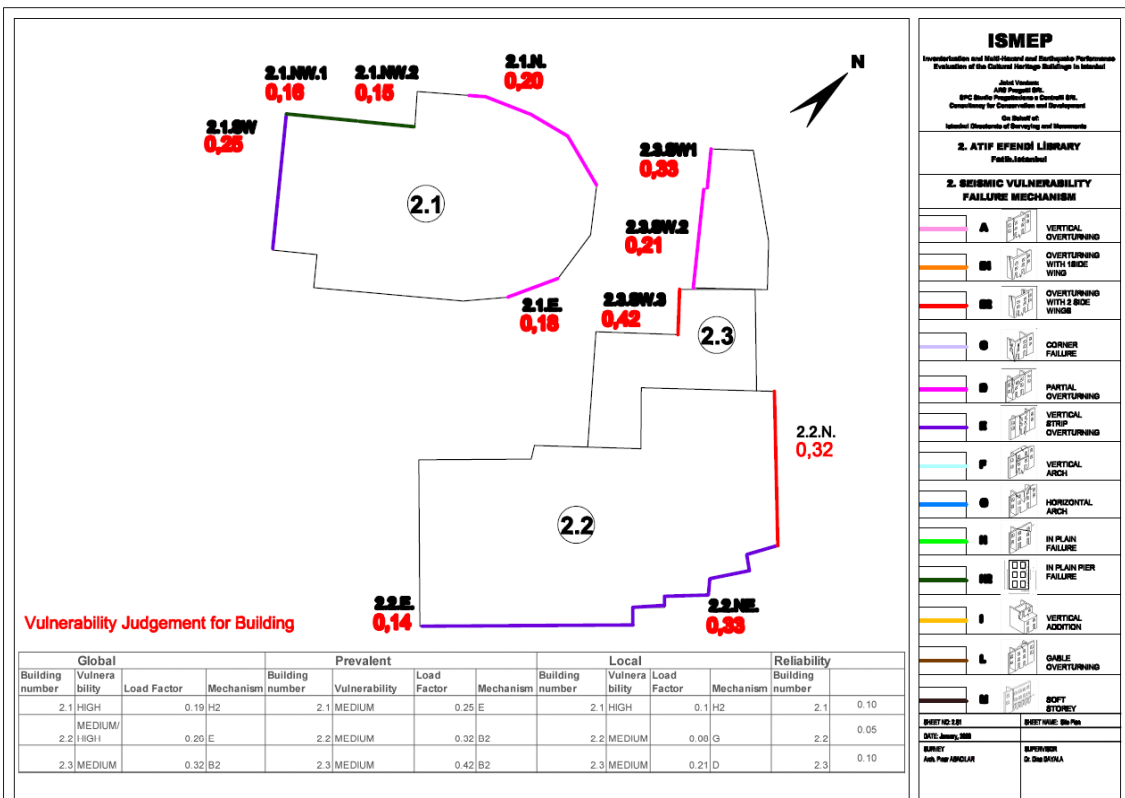

Figure 5: mapping of results on one of the units of assessment, by means of mechanism, collapse load factor and reliability at global, prevalent and local level

The reliability judgement, relating to the input of the data provides the range of confidence with which the central value of collapse load factor is arrived at. In this respect only one value is given for each building indicative of the overall quality of the assessment. It is worth notice that besides the final values described above and directly implemented in the GIS, for each elevation analysed results are provided for all feasible mechanisms, together with the extension of wall involved in terms of storeys and the consequence for horizontal structures. This allows the operator to compare results for a given elevation and quickly consider what would be the expected collapse load factor if the current most likely mechanism is prevented. (See Fig. 2)

4.2. Definition of capacity

The fundamental assumption of the work developed in this project is that the seismic capacity of masonry walls is highly reliant on the possibility of considering post elastic behaviour, and specifically extensive cracking under relatively stable loading conditions. The frictional model introduced to describe the structural behaviour, at the basis of both FaMIVE and VULVAULT, indeed assumes that any mechanism is stable until the "plastic hinges" needed to define the mechanism are fully developed, i.e for in plane mechanisms, for instance, failure will not occur until the width of the crack is greater than the staggering of the units. This

allows considering capacity curves for each wall which are characterised by post peak behaviour. From a mechanical point of view the collapse load factor as calculated in FaMIVE and assumed as the peak capacity of the element, corresponds to the inception of collapse and hence to a condition of "repairable damage", characterised by damage level D2 to D3 depending on the type of structural typology and the mechanisms considered.

The capacity is measured in two ways: a) as ratio of non linear acceleration demand obtained from a NHERP spectrum against acceleration capacity measured as the collapse load factor; b) as a comparison between capacity and demand in spectral displacement terms by checking the performance point on the non linear displacement spectrum obtained starting with the NHERP equivalent spectrum.

The capacity curves are defined for each facade on the basis of 3 parameters. The strength capacity, a_y, is identified on the basis of the limit state analysis and hence coincides for each façade with the collapse load factor obtained with FaMIVE. This quantity is adimensionalised and represent the ratio of horizontal to gravity acceleration a/g. The elastic limit displacement, at the top of each façade, is estimated with the following simple relationship:

$$\Delta_y = \frac{a_y}{4\pi^2}T^2 \qquad \text{with} \qquad T = \sqrt{\frac{m_{eff}}{K_{eff}}} \qquad (1)$$

where T, natural period of the façade, is calculated on the basis of the mass m_{eff} of the façade activated by the failure mechanism, and K_{eff} the elastic stiffness relevant to the specific mechanism (e.i in plane or out of plane stiffness of the façade with specific constraint conditions and cracked cross section).

The ultimate displacement Δ_u is defined in a manner coherent to the mechanism approach at the basis of the FaMIVE procedure, and calculated as the displacement that determines the geometric instability of the facade and hence its collapse. Hence for each wall, in relation to its slenderness and its constraint conditions a different value of out-of-plane or maximum lateral displacement can be calculated beyond which equilibrium is not recoverable. Following this method the capacity curve for each façade can be obtained and the available ductility for each one can be calculated.
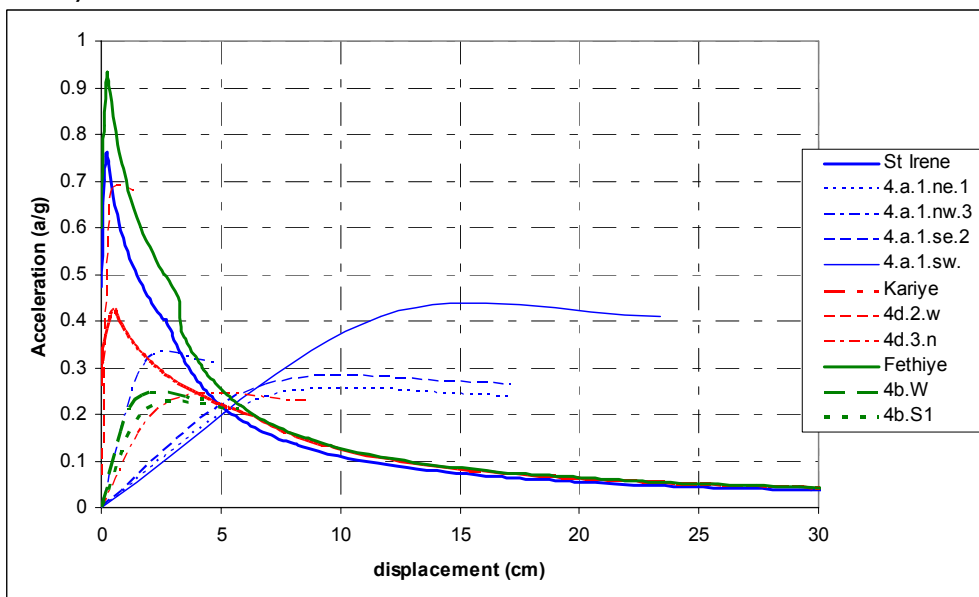

Figure 6 Byzantine buildings

In Fig. 6 the performance curves for 3 buildings are compared with their respective design response spectra. The buildings chosen are Byzantine buildings of different periods and size, but very similar construction techniques, material and layout. Each capacity curve is associated to a specific façade or portion of the facade for a building and in this case the curve representative of the prevalent vulnerability as defined in 4.2. are represented here. It can be noted that 3 recurring values of stiffness are identifiable, corresponding to the type of mechanism and to the construction proportions which are fairly constant throughout the 3 buildings. A ductility value of 3 has been assumed to obtain the non linear displacement spectra.

InFig. 7 three 18th century buildings of similar layout and construction techniques are also compared. Here also the stiffness is essentially constant, but peak capacity and ductility show greater variability. In both

diagrams it is worth noticing also the variability in the spectral curves and hence the importance of local soil conditions. Where the capacity is greater than 0.4, it is because anchoring metallic ties are present and have been assumed to be effective. Besides these cases it can be noted that the capacity of the various elevation are fairly similar, what changes is the stiffness and ductility depending on the type of mechanism that is triggered.
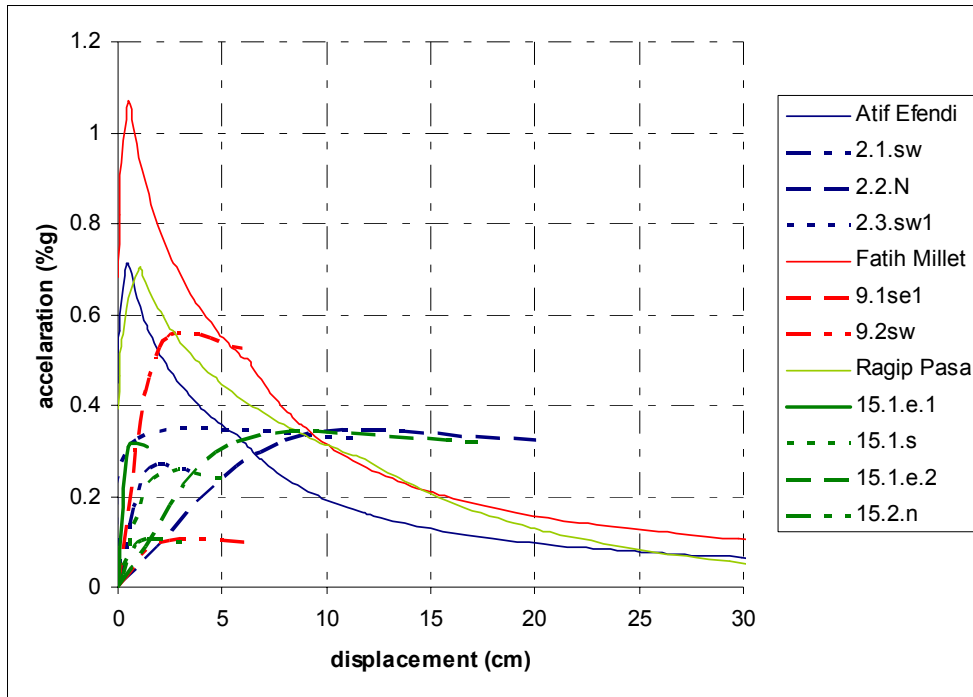

Figure 7: 18th Century libraries with central plan layout

5. DEFINITION OF RISK BY COMPARISON OF CAPACITY CURVES WITH NON LINEAR SPECTRA

The evaluation of seismic risk, in the ISMEP project, is based on the comparison between the vulnerability of each building and the seismic hazard of its site. For buildings assessed with FaMIVE the numerical global collapse load factor has been compared with the non linear spectral acceleration obtained from site specific spectral analysis for the deterministic earthquake. Comparison can occur in terms of spectral acceleration for a given natural period or in terms of damage level corresponding to the performance point identified by the intersection of the capacity curve with the relevant spectrum.
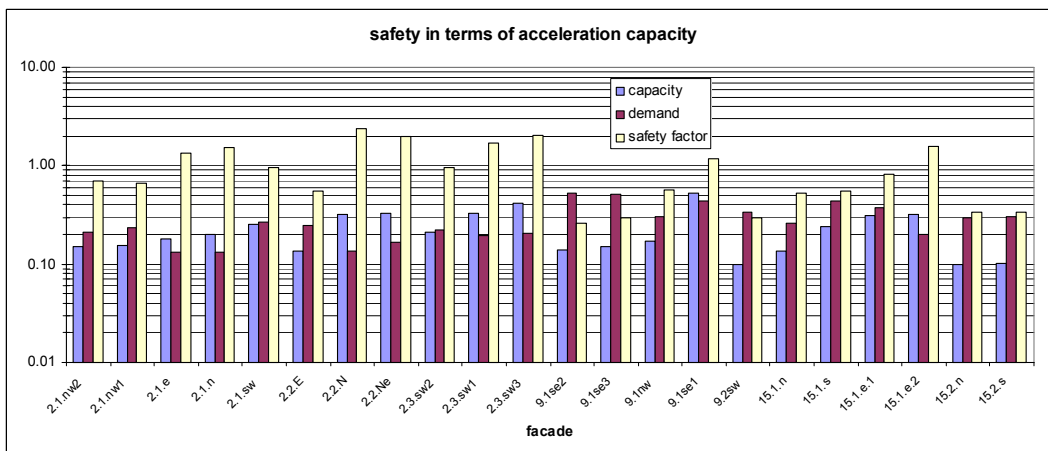

Figure 8: Safety factors in terms of acceleration capacity for the buildings in Fig. 6

Such intersection can be defined in terms of drift or in terms of expected damage. The drift is defined as the displacement corresponding to the intersection of the capacity and spectral curve, over the height of the façade. The expected damage is defined in terms of % of ultimate displacement. For simplicity, if the capacity

curve meets the spectral curve for a value smaller or equal than the limit elastic displacement, it is assumed that the damage will be non structural, corresponding to level D2; if the performance point is between this limit and 50% of the ultimate displacement, the damage level is D3, if it is between 50% and 75% the damage level is D4, if greater D5. The corresponding classes of limit drift assumed are respectively smaller that .3%, between, 3% and .5%, between .5% and .75, or greater.

As it can be seen in Fig. 8, the classes so defined correspond well in terms of sample percentage, not just between damage level and drift, but also between these two and the corresponding acceleration capacity classes. These values have been obtained using throughout a ductility factor of 3 and a corresponding maximum reduction factor $R_{max}=3$. In reality the ductility of some of the façades can be greater, so the value chosen represent a minimum threshold rather than an upper limit. These assumptions are in line with tests reported in Tomazevic 2008 aimed at testing actual values of ductility, and in the absence of direct tests on the sample, are certainly conservative.

The results shows that under these conditions more than 40% of the sample has a capacity comprised between .08 and .18 , a required drift greater than .75% and collapse is expected for the earthquake scenario chosen.. Buildings to suffer non structural damage are 25% of the sample, while building suffering structural damage pr partial collapse are 13% and 16% respectively.

For the purpose of the project and in agreement with the limited reliability of the actual figures involved, and the accuracy of the analysis, it is more appropriate to express the risk in qualitative terms defining classes of risks as follows:
- LOW for buildings of low vulnerability for both judgements which show a safety factor $\Lambda > 1.5$
- MEDIUM/LOW for buildings of either low or medium vulnerability with safety factor $1.15 < \Lambda < 1.5$
- MEDIUM for buildings of medium vulnerability for both judgements with $.85 < \Lambda < 1.15$
- MEDIUM/HIGH for buildings of either medium or high vulnerability with $0.6 < \Lambda < 0.80$
- HIGH for buildings of high vulnerability for both judgements with $0.4 < \Lambda < 0.6$
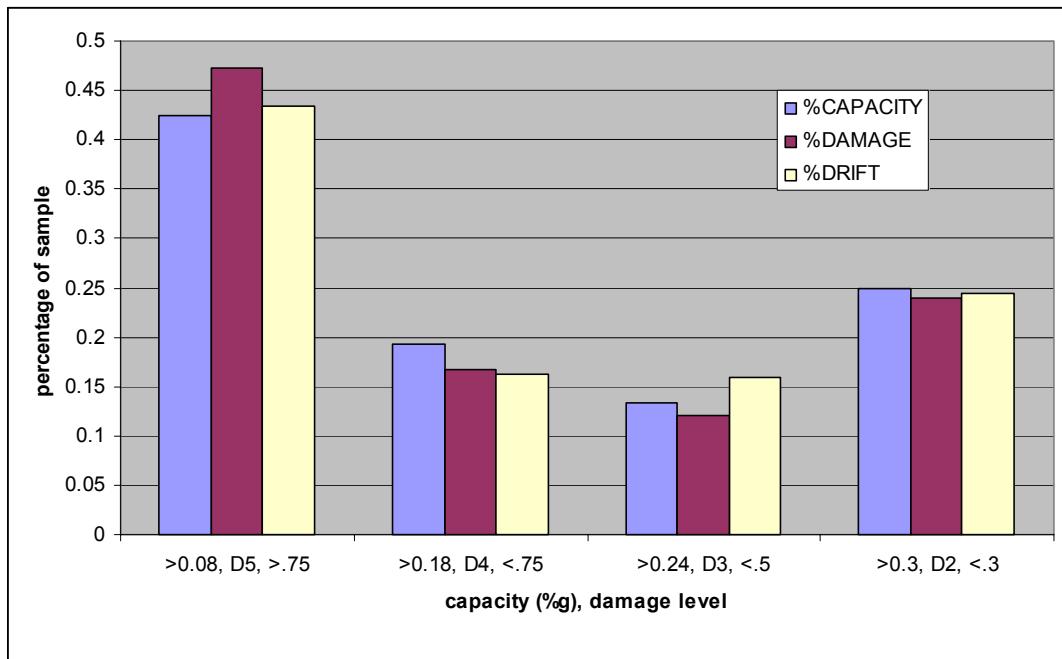- VERY HIGH for buildings of very high vulnerability for either judgement with $\Lambda < 0.4$.

Figure 9: Comparison of force capacity, damage level and drift

Moreover the judgement is weighted according to the protection level of which the building needs to be endowed. The expected performances of any building is generally diversified according to its importance and use, and therefore to the more or less heavy consequences of damage due to a seismic event. The level of protection depends, therefore, on the historical and architectural value of the building and of its contents, as well as on its strategic importance and its level of use. In practice with the definition of a level of protection the conventional evaluation of the seismic risk is tailored to the expected performances of the assessed building. Specifically the protection level rating is: LOW for buildings of relative recent construction or that have undergone substantial alteration, i.e of low architectural value, whit modest content and function; MEDIUM for buildings of some architectural value with content and function of secondary importance; HIGH for buildings which for either their artistic or architectural value or for their strategic use or for the value of

their content are considered of primary importance. Correspondently the safety factor as defined above is weighed by a factor of 1.2, 1 and 0.8 respectively for the 3 above classes of protection.

The reliability of the risk assessment depends on the reliability of the vulnerability assessments and of the evaluation of on site seismic hazard. As a reliability judgement has been associated to each step of the assessment the risk reliability is the compound value of the reliability of all components.

According to this definition for the subsample in Fig. 7 30% of the structures are either low risk or very high risk, while 20% and 16% are respectively medium or high risk, having combined the intermediate classes, given the small size of the subset used..

6. DISCUSSION AND CONCLUSIONS

A non-linear pushover analysis procedure has been applied to study the seismic vulnerability and quantify the risk of a large sample of historic buildings belonging to or in use by the Ministry of Cultural Heritage. This project is part of the wider ISMEP initiative to reduce the seismic risk of Istanbul. The study has entailed defining the most appropriate earthquake scenario, further evaluate the earthquake spectra depending on local site condition and finally modifying the equivalent NERPH spectra to take into account the non linear behaviour of the structures.

The procedure allows results to be derived in terms of acceleration capacity, performance points and expected damage. The results produced show that there is good correlation between the three parameters. It also show that the numerical results can be usefully manipulated to obtain classes of buildings with associated different levels of risk, taking into account both different levels of protection and different level of reliability, depending on the quality of the data available.

This approach is particularly useful to inform conservation and upgrading policies as intervention priority can be established on the basis of the quantified risk.. Moreover the numerical output and the identification of all possible mechanism of collapse for each structures also allows to forecast the expected behaviour if the current most likely mechanism is prevented by a strengthening intervention.

REFERENCE

A.Ansal, G.Tönük (2007) "Source And Site Effects For Microzonation", Earthquake Geotechnical Engineering, 4th International Conference on Earthquake Geotechnical Engineering-Invited Lectures, Springer, Editor: K.Pitilakis, Chapter 4, pp.73-92

Ansal, A., Durukal, E., Tönük, G. (2006) "Selection and Scaling of Real Acceleration Time Histories for Site Response Analyses", Proc. of ETC12 Workshop, Athens, Greece.

Ansal, A., Tönük, G., Demircioglu, M., Bayraklı,Y., Sesetyan,K., Erdik,M (2006) "Ground Motion Parameters for Vulnerability Assessment", Proceedings of the First European Conference on Earthquake Engineering and Seismology, Geneva, Paper Number: 1790

Borcherdt, R.D. (1994) "Estimates of Site Dependent Response Spectra For Design (Methodology And Justification)" Earthquake Spectra, 10(4): 617-654, 1994

D'Ayala DF. (2008) *Seismic vulnerability and risk assessment of cultural heritage buildings in Istanbul, Turkey*. in 14th WCEE, Beijing October 2008

D'Ayala DF (2005). *Force and Displacement Based Vulnerability Assessment for Traditional Buildings*. Bulletin of Earthquake Engineering, 3(3): 235-265.

Erdik,M., E.Durukal, K.Şeşetyan, M.B.Demircioğlu (2007) Assessment Of Site-Specific Earthquake Hazard At Reference Ground Conditions, Istanbul Seismic Risk Mitigation and Emergency Preparedness Project (ISMEP) Consultancy Services for Inventorization and Multi-Hazard and Earthquake Performance Evaluation of the Cultural Heritage Buildings in Istanbul under the Responsibility of the Ministry of Culture and Tourism

Erdik M, Fahjan Y (2006) Damage Scenarios and Damage Evaluation, Assessing And Managing Earthquake Risk Geo-scientific and Engineering Knowledge for Earthquake Risk Mitigation: developments, tools, techniques, Ch.10, Book Series: Geotechnical, Geological, and Earthquake Engineering, V2, Oliveira CS, Roca A, Goula X (Eds.)

Idriss, I. M. and Sun J. I. (1992) "Shake91, A Computer Program for Conducting Equivalent Linear Seismic Response Analysis of Horizontally Layered Soil Deposits Modified based on the original SHAKE program Published in December 1972 by Schnabel, Lysmer and Seed"