Application of a Probabilistic Mechanics-based Methodology for the Seismic Risk Assessment of the Italian RC Bridges

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SUMMARY:
This study presents the evaluation of the seismic risk of the Italian multi-span simply supported and multi-span continuous RC bridges. The seismic fragility of the bridges is based on the computation of analytical fragility curves. A database is developed to collect the bridge information in terms of position, geometry and structural data. A complete set of data was available for about four hundreds bridges for which the fragility curves were computed considering two different limit states – damage and collapse. The results obtained for these bridges are then statistically extended to bridges for which the only known information is the georeferentiantion (more than 15000) thus allowing the computation of their seismic risk. A WebGIS platform has been developed for handling large-scale seismic risk assessment and real-time damage scenarios. The outcomes of this study can be extremely useful for prioritizing retrofit, pre- and post-earthquake planning and management as well as loss estimations.

Keywords: RC bridges, nonlinear time-history analyses, fragility curves, seismic risk assessment

1. INTRODUCTION

The majority of the Italian bridges and road networks was built more than 30 years ago and they result to be extremely vulnerable structures from a seismic point of view for two main reasons. First of all, these bridges were designed according to the regulations valid during their construction period and the latter usually appear to be non-conservative with respect to the current design code (D.M. 14.01.2008, herein called NTC08). Moreover, the classification of the seismic zonation was updated several times during the last decades. Hence, these bridges could not be designed for an adequate seismic resistance with respect to the recent prescriptions. The existing Italian reinforced concrete (RC) multi-span simply supported and multi-span continuous bridges are the object of this research related to the evaluation of their seismic risk.

The study is developed within the project "Seismic risk of the national road transport network", commissioned by the Italian Civil Protection Department. A probabilistic mechanics-based method has been applied for the computation of the fragility curves of the bridges performing nonlinear time-history analyses.

A number of projects have been carried out in the past for the evaluation of the seismic vulnerability of the road transport network, focusing the study at Regional scale or considering structures belonging to a specific managing authority (i.e., Pinto et al., 1996; Progetto V.I.A; Progetto S.A.G.G.I, Cardone et al., 2006). The aim of this research is the evaluation of the seismic risk of bridges at a national scale whose outcomes can be extremely useful for prioritizing retrofit, pre-earthquake planning, post-earthquake management and loss estimations.

2. AVAILABLE DATABASE

An ad-hoc database was developed for the collection of the data. This database has been generated to be as comprehensive as possible in order to create, whenever feasible, an accurate digital model of each bridge. The database is organised in several classes collecting detailed information on: materials, spans, expansion joints, bearing devices, columns and foundations. All these data collected in the database allow, for each bridge, the definition of a structural model ready to be used for performing nonlinear time-history analyses.

The available database is populated by a large number of RC bridges (more than 15000) considering those under the control of the national managing authorities (ANAS) and those that the national authority released to local administrations (e.g., Provincia Autonoma di Trento). Bridges along the motorways are not included yet because the national authority submitted them to private company control few decades ago, but they may be included in the future developments of the project.

In the database, there are 14966 ANAS bridges for which the only available information refers to the georeferenziation. In addition, there are 1000 bridges of the Provincia Autonoma di Trento for which the collected information is represented by the location and a reduced set of geometrical data.

A complete set of fundamental information (location, geometry and structural data) was available for about four hundreds ANAS bridges. Therefore, the classes of database are fully populated with the information required for the creation of the digital model for each one of these bridges.

3. SEISMIC FRAGILITY OF THE RC BRIDGES

In this research, the seismic fragility of the bridges is based on the computation of fragility curves which give the probability of reaching or exceeding a limit state given a ground shaking level. The parameter used for the definition of the different levels of ground shaking is the bedrock peak ground acceleration, a_g.

As stated by Choi et al. (2004), analytical fragility curves are developed through seismic response data from the analysis of bridges. The fragility analysis generally includes three major parts: (a) the simulation of ground motions, (b) the simulation of bridges to account for uncertainty in bridge properties, and (c) the generation of fragility curves from the seismic response data of the bridges. As previously introduced, the seismic response data are obtained from nonlinear time-history analyses, using OpenSees (http://opensees.berkeley.edu/) as finite element code. By means of an automatic procedure, BRIdge auTomatic NItha-based Earthquake fragilitY (BRI.T.N.E.Y.), n-suites of m-simulations are carried out for each bridge. The number n of suites herein considered corresponds to the 9 return periods, T_r, for characterising the ground shaking (from 30 to 2475 years). Each suite consists of m couples of artificial horizontal records, generated for being statistically independent in x and y directions and compatible with the elastic spectra associated to each bridge according to the NTC08 prescriptions. Soil dynamic amplification effects are included in the definition of each spectrum. The records are applied in the x and y directions of each bridge fully restrained at the base. A validation study has been carried out in order to verify the accuracy of the results as a function of the number of m-simulations and their duration. Considering the large amount of nonlinear time-history analyses to be carried out and the computational costs for performing them and manipulating the results, it was decided to use 10 couples of records per suite, and to generate records whose duration is equal to 15 seconds (with a length of the pseudo-stationary part equal to 10 seconds, as prescribed by NTC08). Therefore, 90 simulations have to be carried out for computing the fragility curve of each bridge. The evaluation of the effects of employing natural scaled signals instead of artificial records for computing fragility curves is carried out in Ceresa et al. (2012, in these Proceedings).

The limit states considered for the computation of the fragility curves are the damage and collapse limit states (named DLS and CLS). In order to evaluate the capacity, ductile and brittle mechanisms are taken into account as well as the potential span unseating induced by the horizontal ground motion. For the columns, the ductile mechanisms are checked considering the chord-rotation (θ) of the bottom segment of the pier whereas the brittle mechanisms are related to the shear strength (V) of the column. The unseating of the spans is evaluated computing the relative displacement (d) of the bearings, leading to the evaluation of the span unseating with respect to two limit excursions between the superstructure and the pier. The formulas for the computation of the pier capacity are those prescribed by the NTC08, very close to the Eurocode 8 provisions (2005). The capacity related to the unseating mechanism is computed from the geometry of the bridge. The capacities are always evaluated in the longitudinal and transversal directions and with respect to the considered limit states leading to vectors such as θ_{DLS}, θ_{CLS}, d_{DLS}, d_{CLS} and V_{CLS}. The shear failure of a column is associated to the collapse limit state. However, it is assumed that if a column is affected by shear failure, the structure reaches not only its collapse limit state but the damage limit state too.

The capacity (C) is then compared with the demand (D) computed from the peak values of the time-history results (considering the longitudinal and transversal response interaction). The probability of reaching or exceeding a limit state given a ground shaking (fragility curve) is derived from Monte Carlo analyses. Three demand-to-capacity ratios are computed at each time step and for each direction considering the pier chord rotation (θ), the shear (V) and the span unseating (d):

$$\rho_{\theta,x}= \theta_{D,x}/ \theta_{C,x} \qquad \rho_{V,x} = V_{D,x}/ V_{C,x} \qquad \rho_{d,x} = d_{D,x}/ d_{C,x} \tag{3.1}$$
$$\rho_{\theta,y}= \theta_{D,y}/ \theta_{C,y} \qquad \rho_{V,y} = V_{D,y}/ V_{C,y} \qquad \rho_{d,y} = d_{D,y}/ d_{C,y}$$

Random capacities are associated to these mechanisms taking into account the uncertainty of both the material properties and the analytical formulas. The ratios obtained in x and y directions are then combined according to the SRSS rule or computing their envelope as in the case of the span unseating:

$$\rho_\theta = (\rho^2_{\theta,x}+ \rho^2_{\theta,y})^{0.5} \qquad \rho_V = (\rho^2_{V,x}+ \rho^2_{V,y})^{0.5} \qquad \rho_d = \max(\rho_{d,x}, \rho_{d,y}) \tag{3.2}$$

An overall seismic demand-to-capacity ratio Y of the bridge is computed, for each one of the considered limit states, as the envelope of the three ratios computed for each time step. The Y variable expresses the global state of a structure according to the cut-set formulation, whereby a structural system is described as sub-systems in series of its components. In a series configuration, a failure of any component results in failure for the entire system. Y is the ratio between demand and capacity for the component that leads the system closer to considered LS, at the attainment of which Y equals unity:

$$Y_{DLS} = \max(\rho_{\theta,DLS}, \rho_{d,DLS}, \rho_{V,CLS}) \tag{3.3}$$
$$Y_{CLS} = \max(\rho_{\theta,CLS}, \rho_{d,CLS}, \rho_{V,CLS})$$

For each one of the 9 return periods, the peak values of Y are computed for each simulation, leading to 10 values of Y (Y_i). The Y_i values computed for each T_r are then fitted with a lognormal distribution as represented in Figure 3.1 (on the top).

The probability of reaching or exceeding a limit state is then analytically computed from the probability distribution of the Y_i, conditioned by the considered T_r (i.e., a_g) of the ground motion:

$$p(T_r) = \Pr (Y_{Tr} > 1) = 1 - CDF_{Y|Tr}(1) \tag{3.4}$$
$$p(T_r) = 1 - \Phi[(\ln(1) - \mu_{Y|Tr})/ \sigma_{Y|Tr}] = \Phi [(\mu_{Y|Tr} - \ln(1))/ \sigma_{Y|Tr}]$$

which numerically corresponds to the red area plotted in correspondence to a given T_r in Figure 3.1.

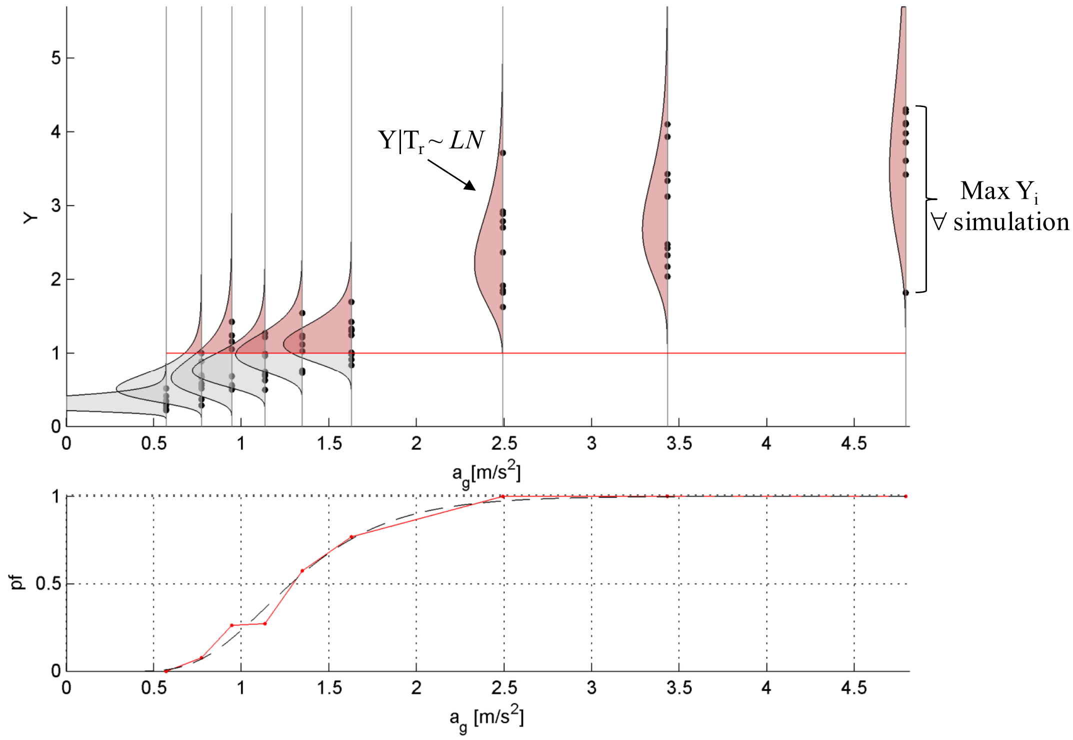


Figure 3.1. Demand-to-capacity ratio Y of the bridge (on the top) and computed fragility curve with its fit (on the bottom)

Therefore, the fragility curves associated to the considered limit states are derived as follows:

$$p_{DLS,Tr} = \Phi\left[\mu_{DLS|Tr} / \sigma_{DLS|Tr}\right] \qquad (3.5)$$
$$p_{CLS,Tr} = \Phi\left[\mu_{CLS|Tr} / \sigma_{CLS|Tr}\right]$$

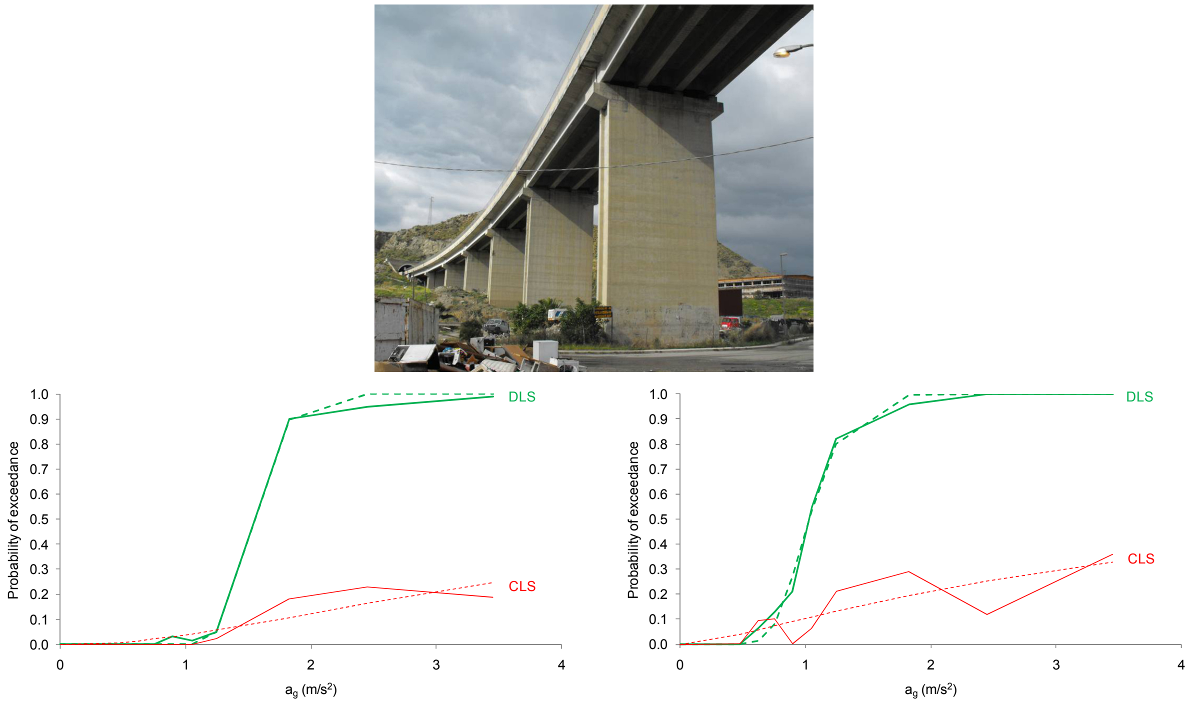
where Φ [·] denotes the cumulative distribution function (CDF) of the standard normal distribution, μ is the mean and σ the standard deviation of the logarithm of the Y values.

4. FRAGILITY CURVES

Applying the probabilistic method described in Section 3, the fragility curves of 389 bridges of the ANAS database have been computed using the application BRI.T.N.E.Y.

The 336 bridges belonging to the 389 ANAS structures are characterised by the presence of rubber bearing devices, whereas the remaining 53 bridges have different bearing devices (such as, fixed, seismic devices and so on). These rubber bearing devices are the most representative bearings used in bridges built between '60 and '70 in Italy. The application always models the behaviour of these rubber bearings with a friction constitutive law according to the study of Capozzi (2009). However, very few are the studies describing the failure of these bearings and the constitutive law to be adopted after the rubber failure. The failure phenomenon is complex and can lead to the direct contact between the span and the pier cap. When contact happens, there are no relative excursions between the adjacent spans and the pier as in the case of fixed restraint between them. In order to model the described post-failure behaviour and to take into account the uncertainties related to the right definition of the friction coefficient to be associated to these devices, a second analysis is carried out for all the bridges with the rubber bearings modelling them with a rigid constitutive law in order to have nil relative excursions.

Hence, it can be stated that two different assumptions are introduced for modelling the same phenomenon. All the RC bridges of the database with rubber bearings have been analysed twice. The first analysis, with friction devices, allows the computation of the maximum relative excursions between the spans and the bents and it is important for the evaluation of the span unseating. A friction coefficient equal to one has been assumed. The second analysis, with fixed devices, leads to catch the maximum feasible forces acting on the columns and plays a fundamental rule in the evaluation of the ductile and brittle mechanisms of the columns. The RC bridge shown in Figure 4.1 has rubber bearing devices and the fragility curves are computed taking into account the double assumption for modelling these devices.



(a) (b)

Figure 4.1. Fragility curves computed for a RC multi-span simply supported bridge of the database with rubber bearings modelled with a) friction and b) fixed devices

As introduced in Section 2, the database collects 14966 ANAS georeferenced bridges since the only available information is their position. With a statistical analysis of the results computed for the 389 ANAS bridges, "mean" fragility curves could be defined in order to allow the seismic risk evaluation of these structures. A least-squared regression analysis with a cumulative log-normal distribution of the discretized fragility curves computed for these 336 ANAS bridges with rubber bearings is carried out. The derived "mean" fragility curves are plotted in Figure 4.2 and they are associated to all the bridges of the database for which an ad-hoc computation was not possible with the available data.

An additional statistical manipulation of the computed fragility curves refers to the computation of the fractile curves which give extremely useful information on the distribution of the probability of reaching or exceeding the damage and collapse limit state, respectively. As described in Section 2, each fragility curve, computed with the application BRI.T.N.E.Y, is described by nine points, each one corresponding to a given return period (from 30 to 2475 years). Starting from the fragility curves computed in their discrete form, the fractile curves are calculated for the overall 389 bridges for a given value of a_g. This computation has been carried out considering the fragility curves obtained as a function of the different assumptions introduced for modelling the bearing devices and for the two considered limit state conditions. In the plots of Figure 4.3, only the fractiles of the fragility curves of the bridges with rubber bearings modelled with fixed devices are presented since the computed fragility curves are usually characterised by higher probability values (as shown in Figure 4.2), mainly affecting the seismic risk evaluation. The 5, 10, 25, 50, 75, 90 and 95% fractile curves have been computed. The dots in Figure 4.3 are the points of all the fragility curves computed for the 336 bridges modelled with fixed devices. Knowing the fractile curves and for a given return period T_r of the ground motion (or a given a_g value), it is possible to associate to a bridge belonging to the same sample of bridges a probability distribution of exceeding the DLS and CLS limit state, respectively.

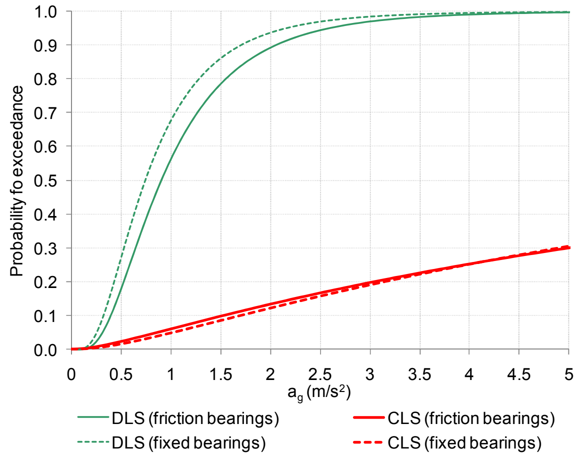
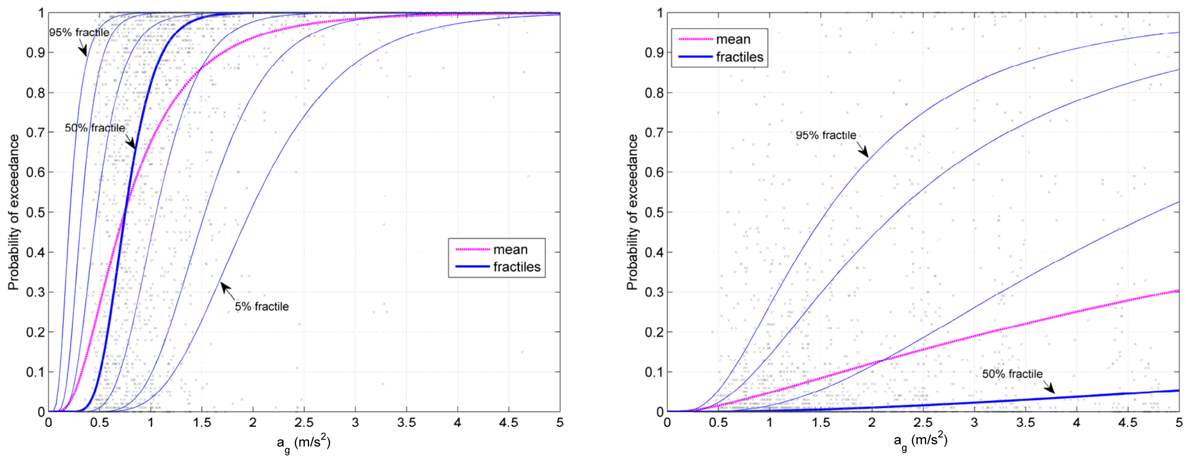


Figure 4.2. "Mean" fragility curves



(a) (b)

Figure 4.3. "Mean" curve and fractiles of the fragility curves for (a) DLS and (b) CLS (analysis with fixed bearings)

In order to evaluate the accuracy of the applied methodology and the obtained results, the latter should be compared, whenever feasible, with the ones derived from other studies. In the works published in literature related to the computation of fragility curves for RC bridges, no enough data have been found for modelling the presented cases-of-study. Hence, it was not possible to directly repeat the computation of the published fragility curves. Figure 4.4 shows the comparison between the "mean" fragility curves (Figure 4.2 - fixed bearings) and the curves published by Nielson and DesRoches (2007a, b) for the bridges commonly found in the Central and Southeastern United States (MSC stays for multi-span continuous girder and MSSS for multi-span simply supported girder). The comparison has to be evaluated taking into account two main factors: i) the limit states considered are not the same; ii) the databases are different and, in particular, the Nielson and DesRoches' fragility curves are computed considering nine US bridges. Therefore, it could be stated that the comparison is satisfactory and the higher probabilities of exceedance related to the "mean" fragility curves could be justified by the lower reinforcement ratios characterising the Italian bridges with respect to the US ones.

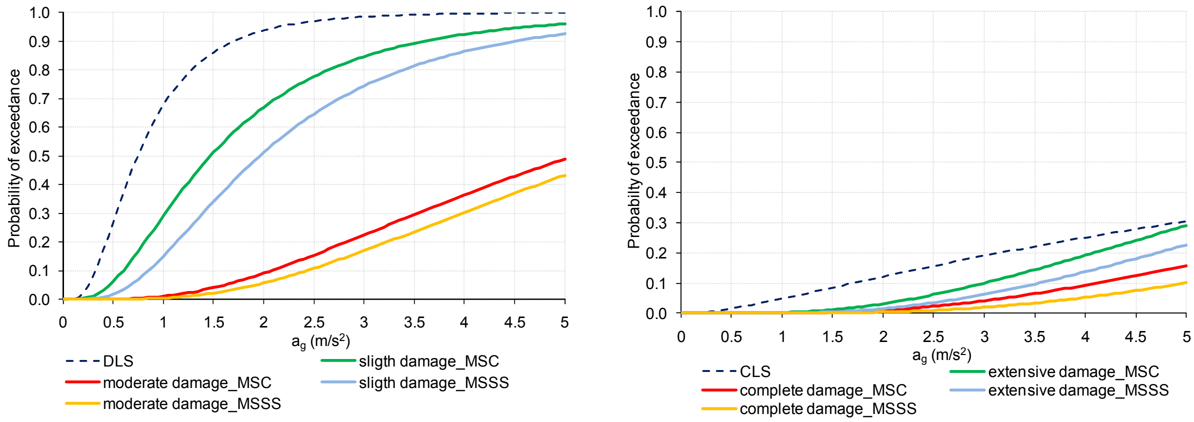


Figure 4.4. Comparisons of the "mean" fragility curves and the results of Nielson and DesRoches (2007a, b)

5. EVALUATION OF SEIMIC RISK

The vulnerable structures to be studied are the RC bridges of the Italian road transport network. The seismic vulnerability is the probability of reaching or exceeding a limit state given a ground motion intensity level:

$$P_{ik} = P\,[D \geq d_i \mid E = e_k] \tag{5.1}$$

where P_{ik} is the probability that the damage corresponding to the limit state d_i is reached or exceeded for a ground shaking level e_k; D and E are the variables of damage and ground shaking. The fragility curves are the mathematical expression of the vulnerability and, in this research, they are expressed as a function of the peak ground acceleration at bedrock, a_g. The probabilities of exceeding a limit state have been obtained for 9 return periods. These probabilities can be expressed in terms of conditional or unconditional seismic risk. The first takes into account fragility curves as function of a_g values, while the second one accounts for many possible future earthquake scenarios that could impact upon bridges during a given time window.

The seismic hazard is based on the probabilistic study carried out in the INGV-DPC S1 project (http://esse1-gis.mi.ingv.it; Montaldo et al., 2007) and adopted in the NTC08 allowing the computation of the mean spectral accelerations for a grid of points (each one at distance of 0.05 degrees) and for seismic events characterised by a return period T_r varying from 30 to 2475 years. In the seismic risk evaluation of the RC bridges, the least-square interpolation method has been applied to the median acceleration spectrum (50th percentile) plus and minus one standard deviation for deriving the parameters that allow the computation of the 84th and 16th percentile spectral accelerations, respectively. Therefore, the seismic demand imposed by the ground motion to the bridges is computed with respect to the 16th, 50th and 84th percentile acceleration spectra. With the fragility curves associated (for DLS or CLS) to the bridges of the database, the conditional seismic risk is computed deriving the probability of damage or collapse for a given a_g (i.e., T_r) and a selected percentile. The unconditional seismic risk considers the probability of occurrence of a ground shaking level for a given T_r in exposure periods of 1 year, 50 years and 100 years. This probability is expressed as a function of a_g and the Annual Frequency of Exceedance (AFE = $1/T_r$). Figure 5.1(a) shows the conditional seismic risk computed for the collapse limit state, a return period of 475 years and the 50th percentile. From the results plotted in the map, the bridges most at risk appear to be located in central regions of Italy. The map in Figure 5.1(b) gives the unconditional seismic risk for the CLS in the exposure period of 50 years, showing values always less than 50% even in regions with high seismicity.

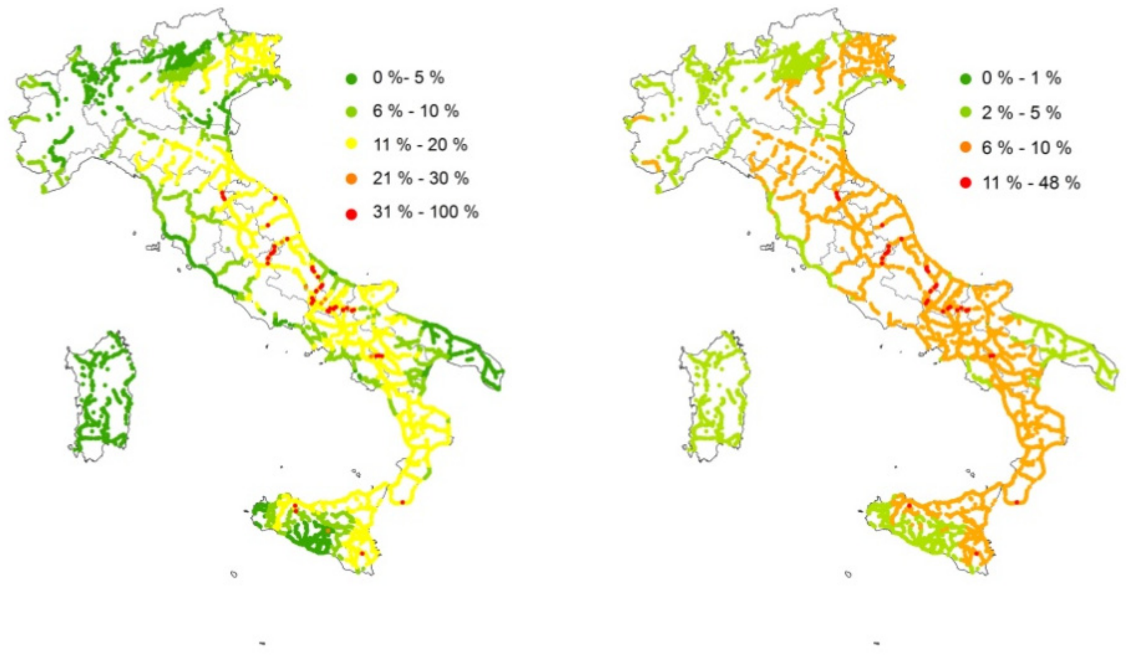


Figure 5.1. Maps showing (a) the conditional probability of exceeding the collapse limit state (CLS) for a return period of 475 years and the 50th percentile; (b) the unconditional seismic risk of exceeding the CLS in a time window of 50 years

The seismic risk maps of Figure 5.1 are computed with the WebGIS (Web Geographical Information System) platform developed within this research. Furthermore, the WebGIS platform allows, by means of interactive tabs and maps, to view the location of the bridges, the data collected in the database and the computed fragility curves.

The routines developed for the calculation of the seismic risk also allow the computation of a real-time damage scenario, using the seismic input obtained from a selected attenuation relationship for given values of magnitude and distance. Three attenuation laws have been implemented (Cauzzi and Faccioli, 2008; Boore and Atkinson, 2008; Akkar and Bommer, 2010) for i) their simple form, which allows the generation of a damage scenario with a small amount of input data; ii) good performance when compared with the spectra derived from the records of several real Italian earthquakes. A direct application of the developed tools was the Civil Protection drill, carried out in Calabria Region from 25 to 27 November 2011 (http://www.protezionecivile.gov.it). A strong earthquake has been simulated (M_w= 6.49) and a real-time damage scenario for the bridges located at a distance less than 50 km from the epicentre was computed in a very short time (11 minutes).

6. CONCLUSIONS

The evaluation of the seismic risk of the existing RC multi-span simply-supported and multi-span continuous bridges has been carried out at a national scale. The data of more than 15000 bridges were collected in the developed database. The detail of the available information is not the same for all the bridges and a complete set of data was available for about four hundreds bridges. With the application of a probabilistic methodology, the fragility curves of 389 RC bridges were computed for the damage and collapse limit states. Artificial records were generated for the input ground motion simulation. The uncertainty related to the bridge properties was taken into account and the seismic responses of the bridges were obtained by means of nonlinear time-history analyses. Monte Carlo simulations led to the computation of analytical fragility curves for each one of these bridges.

The results obtained for the four hundreds bridges were then statistically extended to bridges with an incomplete set of data in order to allow the computation of the seismic risk considering all the bridges of the database. In particular, a “mean” fragility curve has been computed for the bridges for which the only available information refers to the georeferentiation. The data of the bridges and their fragility curves, as well as the procedures for handling large-scale seismic risk assessment and real-time damage scenario, capable to define the safer way to be followed by rescue teams in case of an earthquake, were integrated in a WebGIS platform developed during this project.

The obtained results may be improved upon as more information is collected on the bridges of the database. Moreover, the developed tools are arranged to collect the data of new bridges, such as the ones along the motorways. Further developments can be introduced in the application BRI.T.N.E.Y in order to account for soil-structure interaction and multi-support excitation.

Additional improvements of the presented results should be obtained with the statistical characterisation of the computed fragility curves developing a probabilistic model for the fragility curves conditioned to a small set of data of the bridges that should give a better estimate of the “mean” fragility curve without performing finite element analyses or simulations.

ACKNOWLEDGEMENT

This work was supported by the Italian Civil Protection Department within the framework of the "Seismic risk of the national road transport network" Project (Progetto Operativo d4). This support is gratefully acknowledged. Any opinions, findings or conclusions expressed in this paper are those of the authors and do not necessarily reflect those of the Department. The authors would like to acknowledge the valuable input of Prof. Paolo Emilio Pinto, Prof. Gian Michele Calvi and Dr. Paolo Franchin to the work described in this paper.

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