Seismic damage assessment for six RC bridges in the Veneto region (NE Italy)

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ABSTRACT Seismic damage on six bridges in the Veneto region (NE Italy) is estimated by means of a probabilistic approach for seismic hazard assessment and by constructing the specific fragility curve for each bridge. Construction of fragility curves is based on various approaches from literature. With the application of the best-performing approach on the six bridges a comparative preliminary evaluation of the expected damage on the bridges was done. However, as the methodology is general, it could easily be applied to bridges of an extensive number of road networks.

1. Introduction

Bridges and viaducts are key structures in the communication network, and their efficiency is crucial in case of emergency. This is why the evaluation of their expected damage in the case of a natural catastrophe is important. When bridges and viaducts are located in a seismic area, definition of their operating efficiency is inadequate, as their seismic vulnerability must also be taken into account. The great number of structures and the often limited economic means require the application of a rapid, simple and reliable method, the results of which can be used to draw up maps that immediately identify the structures with a high seismic risk.

The Gruppo Nazionale per la Difesa dai Terremoti (National Group for the Defence against Earthquakes) financed a project aimed at defining scenarios of seismic damage in the Veneto and Friuli - Venezia Giulia regions (NE Italy). This work describes the results of a specific study involving a number of bridges. Part of this work was already presented in Franchetti *et al*. (2004, 2005).

The purpose of this work is to assess the expected damage caused by earthquakes on six bridges in the Veneto region. The fragility curves of the six bridges were created with different methods taken from the literature, and the results obtained were compared. The curves represent the probability of exceeding a fixed damage level according to the intensity of the earthquake's ground shaking. In this work, we calculated the fragility curves of the piers, because they are associated to the fragility of the bridge itself (Shinozuka *et al*., 2000). Computation included the complete hazard curve for the sites where the six bridges are located. Assessment of the expected damage is obtained by the convolution of the site hazard probability density function (PDF), calculated from the hazard curve, and the bridge fragility curve. The computation includes the mechanical characteristics of the structures investigated, as well as the uncertainty of their assessment.

An alternative deterministic way of evaluating the expected damage consists in using the maximum value produced by the largest earthquake for the study region as ground motion. In this case, the source parameters of the extreme event must be known and the damage estimate on the bridge only refers to that quake. Without any sure knowledge about future earthquakes, the intensity of ground shaking and its frequency contents can only be forecasted statistically on the basis of past events.

2. The six bridges

The analyses were conducted on the following six bridges in the Veneto region (web site: http://ibrid.dic.unipd.it), and more precisely in the Belluno and Treviso provinces (Fig. 1):

1. the Campelli bridge, in the Longarone area (Belluno province), has 8 PRC spans of maximum 30 m resting on RC piers; deck width is 10 m (Fig. 2a);
2. the Botteon viaduct, in the Fadalto area (Treviso province), has 6 PRC spans of maximum 24 m resting on RC piers; deck width is 11.5 m (Fig. 2b);
3. the San Vendemiano bridge, in the San Vendemiano area (Treviso province), has 17 PRC spans of maximum 12 m resting on RC piers; deck width is 10 m (Fig. 2c);
4. the Spresiano highway bridge over the River Piave, in the Spresiano area (Treviso province), has 25 PRC spans of maximum 24.75 m resting on RC piers; deck width is 9 m (Fig. 2d);
5. the Quero bridge over the River Piave, in the Quero area (Belluno province), has 12 PRC spans of maximum 34.5 m resting on RC piers; deck width is 10 m (Fig. 2e);
6. the Fener bridge over the River Piave, in the Fener area (Treviso province), has 24 PRC spans of maximum 24.75 m resting on RC piers; deck width is 9 m (Fig. 2f).

The analyses were carried out using 5 different values for concrete and 3 values for steel strength due to the variability of their properties, which provided 15 capacity curves for each bridge. The non-linear behaviour of the materials was considered.

3. Seismic hazard and artificial time histories for the studied sites

The computation of the ground motion at the sites where the studied bridges are located participates twice in the present damage assessment: first for the construction of the fragility curves of the bridges and, second, for the definition of the expected ground shaking at the bridge location. This ground motion computation has been made following a probabilistic approach in a complete agreement with the regional seismic hazard assessment for the broader Vittorio Veneto area (see Slejko *et al*., 2008).

Entering into details, the probabilistic seismic hazard assessment (PSHA) for the broader Vittorio Veneto area has been made according to the standard Cornell (1968) approach by using the computer formulation of Bender and Perkins (1987). This approach is based on two working hypotheses: the earthquake recurrence times follow a Poisson distribution (made up by independent, non-multiple events, and the process is stationary in time) and the magnitude is exponentially distributed (the Gutenberg - Richter relation holds). In addition, the seismicity is considered uniformly distributed over the seismogenic zone (SZ). The Cornell (1968) method, then, needs the following input data: the SZ geometry definition, the seismicity models (in terms

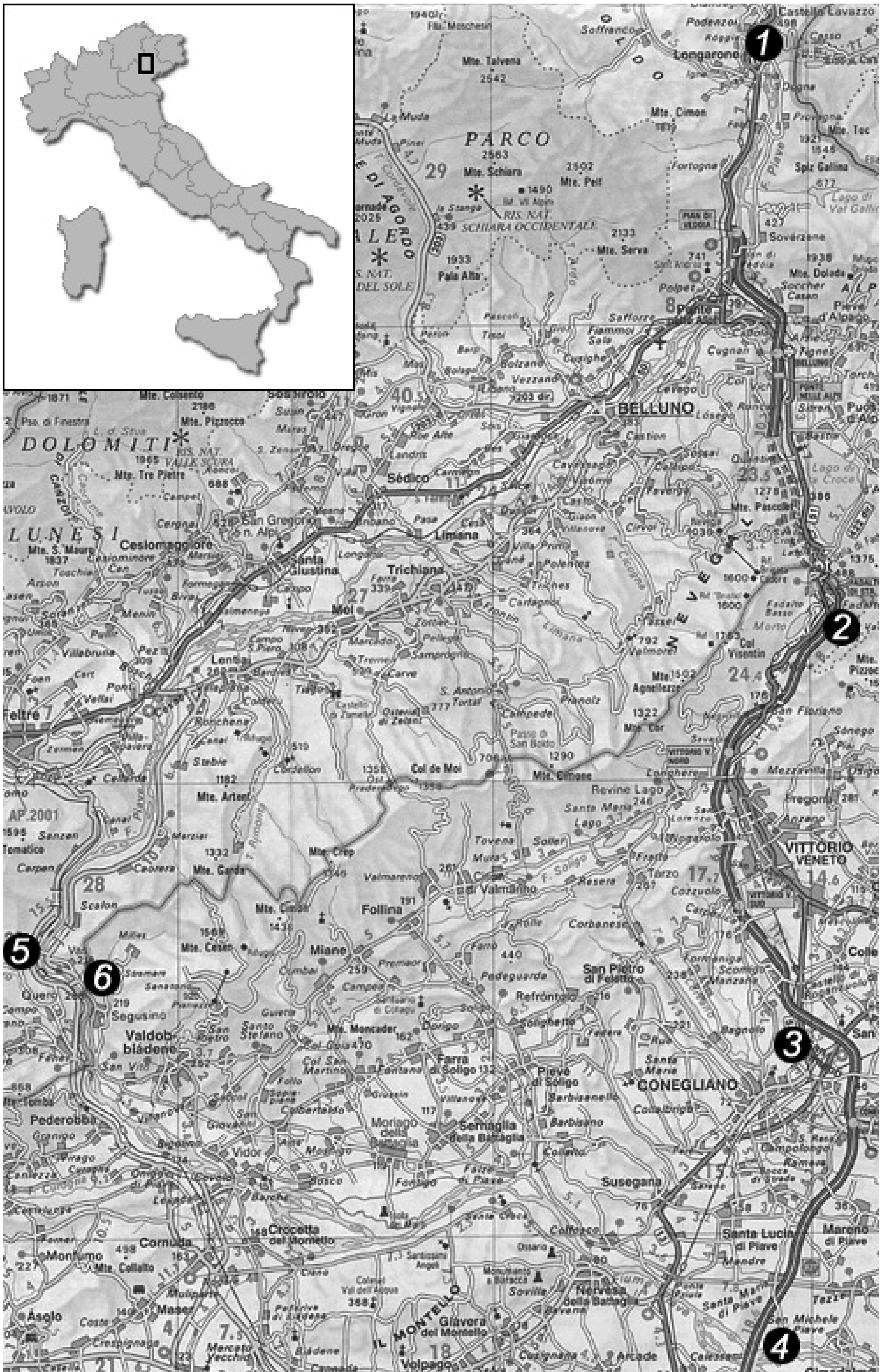

Fig. 1 - Location map of the six bridges: 1 = Campelli bridge, 2 = Botteon viaduct, 3 = San Vendemiano bridge, 4 = Spresiano highway bridge, 5 = Quero bridge, 6 = Fener bridge.

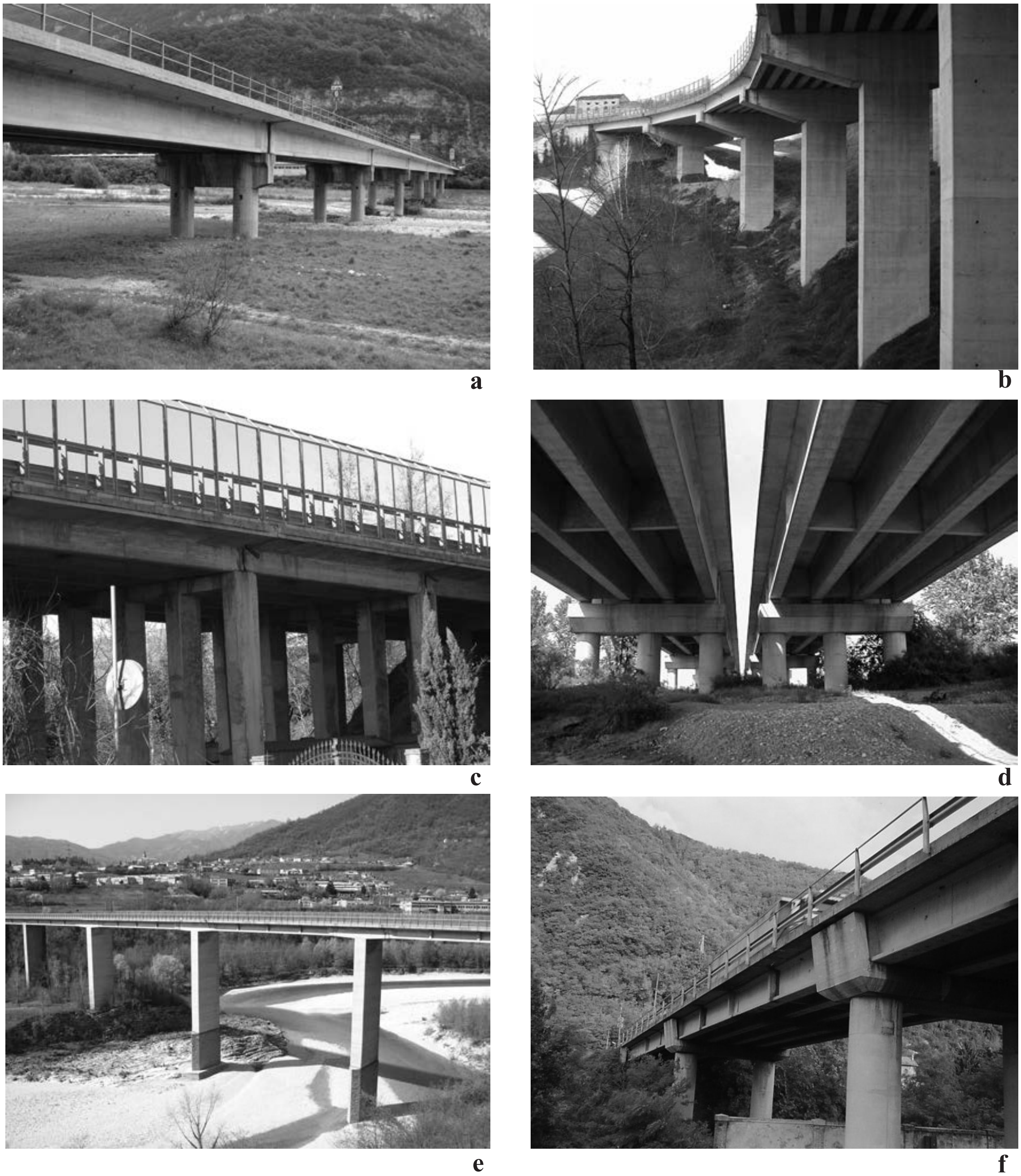

Fig. 2 - View of the six bridges: a) Campelli bridge; b) Botteon viaduct; c) San Vendemiano bridge; d) Spresiano highway bridge; e) Quero bridge; f) Fener bridge.

of average number of earthquakes per magnitude interval, and maximum possible magnitude), and the attenuation relation of the chosen ground motion parameter.

In order to quantify the epistemic uncertainties related to the regional PSHA (McGuire and Shedlock, 1981; Toro *et al*., 1997), the logic tree approach (Kulkarni *et al*., 1984; Coppersmith

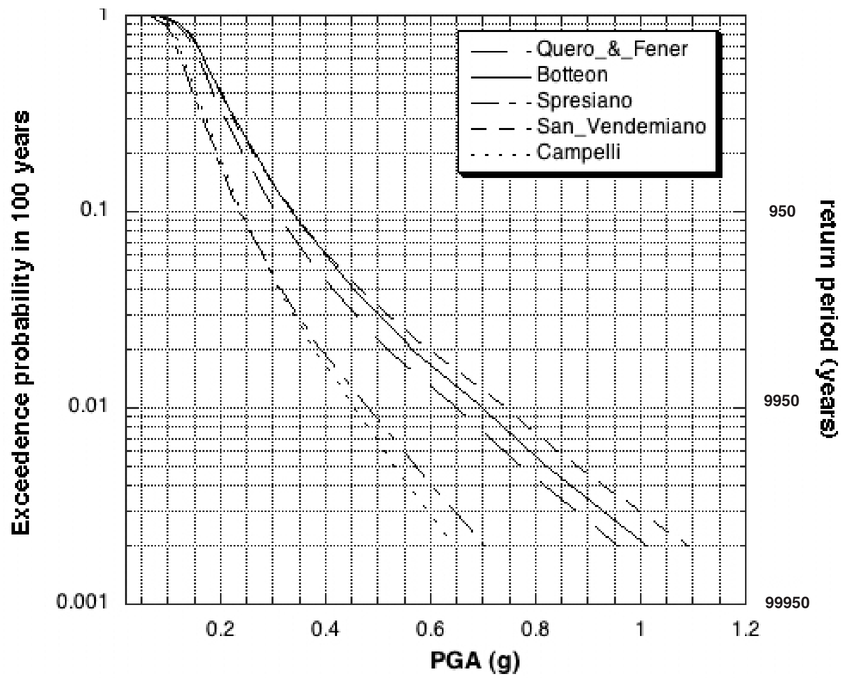

Fig. 3 - Hazard curve for the six bridges: solid line = Botteon viaduct, long dashed line = Quero bridge and Fener bridge, shord dashed line = San Vendemiano bridge, dotted line = Campelli bridge, dotted-dashed line = Spresiano highway bridge.

and Youngs, 1986) has been followed. In our case, for the rock hazard computation (the bridges have foundations on rock), the logic tree consists of 54 branches: we used three seismogenic zonations, representing various levels of our seismotectonic knowledge, three methods for the seismicity rate computation, three statistical approaches for the maximum magnitude estimation, and two attenuation relations of different spatial relevance (Italian, European). All the details about the components of the logic tree and about the elaboration done are reported in Slejko *et al*. (2008). The result of the elaboration is given by the complete hazard curve [properly averaged from the 54 hazard curves, see for details Slejko *et al*. (2008)] of the hazard parameter [peak ground acceleration (*PGA*) or spectral ordinates] for each studied site. This hazard curve accounts, then, for all the uncertainties (aleatory and epistemic) related to the hazard computation. Fig. 3 shows the hazard curves for the six bridges under study: the standard deviation of the attenuation relations (representing the aleatory variability) has been taken into account in the elaboration. For the scope of the present study, the hazard is not represented, as it usually is, by the annual exceedence probability but by the exceedence probability in 100 years.

For the computation of the fragility curve of each analysed bridge, we computed some acceleration time histories, with spectral contents very similar to those of the uniform hazard response spectrum of the site where the bridge is located. More precisely, we constructed the response spectra at a 2% damping corresponding to eight values of *PGA* (0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, and 0.8 g) and from each response spectrum we derived five accelerograms in agreement with it. The random vibration approach (Vanmarcke, 1976) has been used for the artificial motion generation. According to this method, any periodic function can be expanded into a series of sinusoidal waves of variable phase and amplitude. The artificial motion generation is given by superposition of sinusoids having random phase angles and amplitudes derived from a stationary power spectral density function of motion. The final simulated motion is stationary in frequency content with a peak acceleration close to the target peak acceleration. The computer code SIMQKE (Gasparini and Vanmarcke, 1976) is based on this approach and can: 1) compute a power spectral density function from a specified smooth target response spectrum; 2) generate statistically independent artificial acceleration time histories and try, by iterations, to match the specified target response spectrum. To simulate the transient character of real earthquakes, the steady-state motions are multiplied by a deterministic trapezoidal or exponential envelope function.

Ten spectral acceleration (*SA*) values of the uniform hazard response spectrum were taken as the target spectrum, and several trials were done using different intensity envelope functions and by changing the number of cycles to fit that target spectrum. The response spectrum of the artificial time history shows a good agreement with the target spectrum with the exception of some oscillations which are smoothed by definition in the uniform hazard target response spectrum [for more details, see also Slejko and Rebez (2004)].

4. Analysis methods for constructing fragility curves

Three methods for constructing fragility curves have been examined in the present study.

a) The first method was proposed in Hazus99 (FEMA, 2001). It has a very fast application because it avoids numerical analyses, and it can therefore be used for a large number of structures.

b) The second analysis method examined here includes an approximate procedure described in the ATC-40 document (ATC, 1996) to calculate structural deformation due to an earthquake. This procedure is considered approximate because it avoids the dynamic analysis of the inelastic system by replacing it with the analysis of a sequence of equivalent elastic systems. Each deformation value, belonging to the plastic branch of the initial system, corresponds to a different elastic system with a distinct period and viscous damping values. An iterative process is required to reach the intersection between the capacity and demand curves, because the curve of demand depends on the system's viscous damping, which changes at each iteration. However, some authors (Chopra and Goel, 1999) showed that the results provided by the ATC-40 procedures (ATC, 1996) are incorrect because based on simplistic hypotheses, and thus proposed a series of changes. As regards the "Capacity Spectrum Method" (CSM), the approximation in estimating seismic deformation as a sequence of equivalent linear systems, which avoided the analysis of the inelastic system, was criticised. It was also noted that simplified procedures do not always converge. For the construction of the demand diagram, these procedures use the inelastic spectrum at a constant ductility, instead of the elastic project spectrum used in the ATC-40 methods (ATC, 1996).

c) The last analysis method examined is the non-linear dynamic analysis.

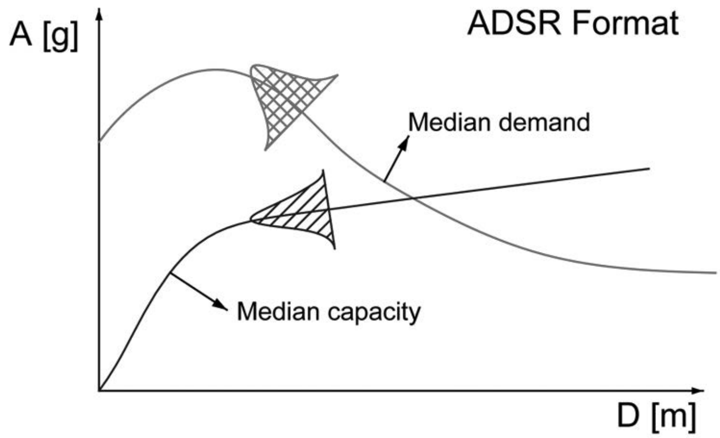

Fig. 4 - Intersection of capacity and demand curves.

5. Fragility curves

Fragility curves are efficient, intuitive tools for evaluating the seismic vulnerability of bridges and viaducts. They are made up of a series of diagrams representing the intensity of ground motion, expressed in terms of *PGA* or *SA* on the *x*-axis, and, on the *y*-axis, the probability of exceeding the damage level (*DL*) to which the curve refers. The probabilistic nature of the issue is due to the randomness of some variables, such as the intensity of expected ground shaking and the values of the actual strength of the materials used.

Fig. 4 shows these uncertainties, indicating that the diagrams of demand and capacity can be obtained by using probability distributions. The obvious conclusion is that the performance level required is not represented by a point but by an interval of intersecting points.

Structural capacity and seismic demand are random variables which adapt to a probabilistic distribution of the log-normal type, and their intersection point can also be represented by a log-normal type distribution. This representation provides a cumulative probability function, i.e. a fragility curve. This curve is described by two parameters: the median value (value with a 50% probability of occurrence) and its standard deviation. The normalised cumulative probability function is given by:

$$F(a) = \Phi\left[\frac{1}{\beta_c}\ln\left(\frac{a}{a_i}\right)\right] \qquad (1)$$

where Φ is the normal and normalised cumulative distribution function; a is the intensity of the ground motion; a_i is the median (or expected value) intensity of ground motion necessary to cause the L_i DL to occur; β_c is the normalised composite log-normal standard deviation which incorporates the aspects of uncertainty and randomness of both capacity and demand. The value of this last parameter was investigated by several authors: the value proposed by Mander (1999) is β_c =0.6, the one used in the HAZUS99 (FEMA, 2001) system is β_c =0.4.

The median values a_i of ground motion required to cause a certain *DL* can be determined after having defined the *DLs*, which are taken approximately as multiples of the limit values of elasticity (ductility demand).

5.1. First approach for constructing the fragility curve: the Hazus99 method

The advantage of the Hazus99 method (FEMA, 2001) lies in its ability to construct the fragility curves of a specific bridge using limited data and avoiding complex structural analyses. A large number of structures can therefore be evaluated in a short time.

Only three types of data are needed to develop the curves: 1) data relating to the geometric-structural characteristics of the bridge and its geographical location; 2) data on the design earthquake for that site; 3) information on the type of ground where the structure is built.

In this method, the construction of the curves is based on a prior classification of the bridges with respect to their geometric–structural characteristics.

The construction of the fragility curve of a specific bridge is performed by adapting the curve for a standard bridge, i.e. a bridge long enough to disregard the three-dimensional effects present. The median ground shaking values (a_1, a_2, a_3, a_4) required to produce certain *DLs* (*SD*, *MD*, *ED*, *CD*) are obtained for the standard bridges belonging to each category.

The four *DLs* taken into account are:

a) *SD*: slight/minor damage is defined as minor cracking and spalling to the abutment, cracks in shear keys at abutments, minor spalling and cracks at hinges, minor spalling at the column (damage requires only cosmetic repair) or minor cracking to the deck;

b) *MD*: moderate damage is defined as any column experiencing moderate (shear cracks) cracking and spalling (column structurally still sound), moderate movement of the abutment (< 2”), extensive cracking and spalling of shear keys, any connection having cracked shear keys or bent bolts, keeper bar failure without unseating and rocker bearing failure or moderate settlement of the approach;

c) *ED*: extensive damage is defined as any column degrading without collapse-shear-failure (column structurally unsafe), significant residual movement at connections or major settlement approach, vertical offset of the abutment, differential settlement at connections and shear key failure at abutments;

d) *CD*: complete damage is defined as any column collapsing and connection losing all bearing support, which may lead to imminent deck collapse and tilting of substructure due to foundation failure.

The *DLs* correspond to:

$SD : \mu_1 = 1;$

$MD : \mu_2 = 2;$

$ED : \mu_3 = 4;$

$CD : \mu_4 = 7;$

where μ is the ductility of the section. These values are taken from a study carried out on true post-earthquake damage affecting U.S. bridges.

Five of our bridges (Botteon, Campelli, Spresiano, Quero, Fener) belong to the same Hazus99 (FEMA, 2001) class HWB17; the sixth (San Vendemiano) belongs to the Hazus99 (FEMA, 2001) class HWB5. The two Hazus99 (FEMA, 2001) classes HWB5 and HWB17 have the same median

Table 1 - Median ground shaking values [defined in terms of *SA* (0.1)] required to cause certain *DLs* for the HWB5 and HWB17 Hazus99 (FEMA, 2001) classes.

a_1 [g]	a_2 [g]	a_3 [g]	a_4 [g]
0.26	0.35	0.44	0.65

ground shaking {defined in terms of *SA* at 01.s [*SA* (0.1)]} values required to cause certain *DLs* (Table 1).

The median ground mean values are then modified by factors that take into account both the angle of skew (K_{skew}) and three-dimensional effects (K_{3D}) related to the specific bridge.

The curve of a specific bridge is constructed as follows:

1. identification of the bridge characteristics: location, structural type according to predefined classes, number of spans (N), angle of incidence (α), deck width (W), total length (L), maximum length of the spans (L_{max});
2. evaluation of the soil-amplified shaking at the bridge site: that is, get *PGA*, *SA*(0.3) and *SA*(1.0);
3. evaluation of the three modification factors:

$$K_{skew} = \sqrt{\sin(90° - \alpha)} \tag{2}$$

$$K_{shape} = 2 \times \frac{SA(1.0)}{SA(0.3)} \tag{3}$$

$$K_{3D} = 1 + \frac{A}{N - B} \tag{4}$$

where A and B are tabulated in Hazus99 (FEMA, 2001);

4. modification of the ground shaking median values for the "standard" fragility curves (Table 1);
5. use of the new median values along with dispersion $\beta_c = 0.4$ to evaluate the ground shaking related *DL* probabilities.

The fragility curves for our six bridges are very similar to each other (Fig. 5); in particular, those for the L_1 *DL* agree. This is due to the fact that the bridge types are similar (RC simply supported bridges).

5.2. Construction of fragility curves through non-linear static analysis

The fragility curves of the six bridges were analytically obtained using the structure seismic displacements, according to the method proposed by Shinozuka *et al*. (2000). This method considers both the variability of the expected earthquakes and the characteristics of the materials used in the structure.

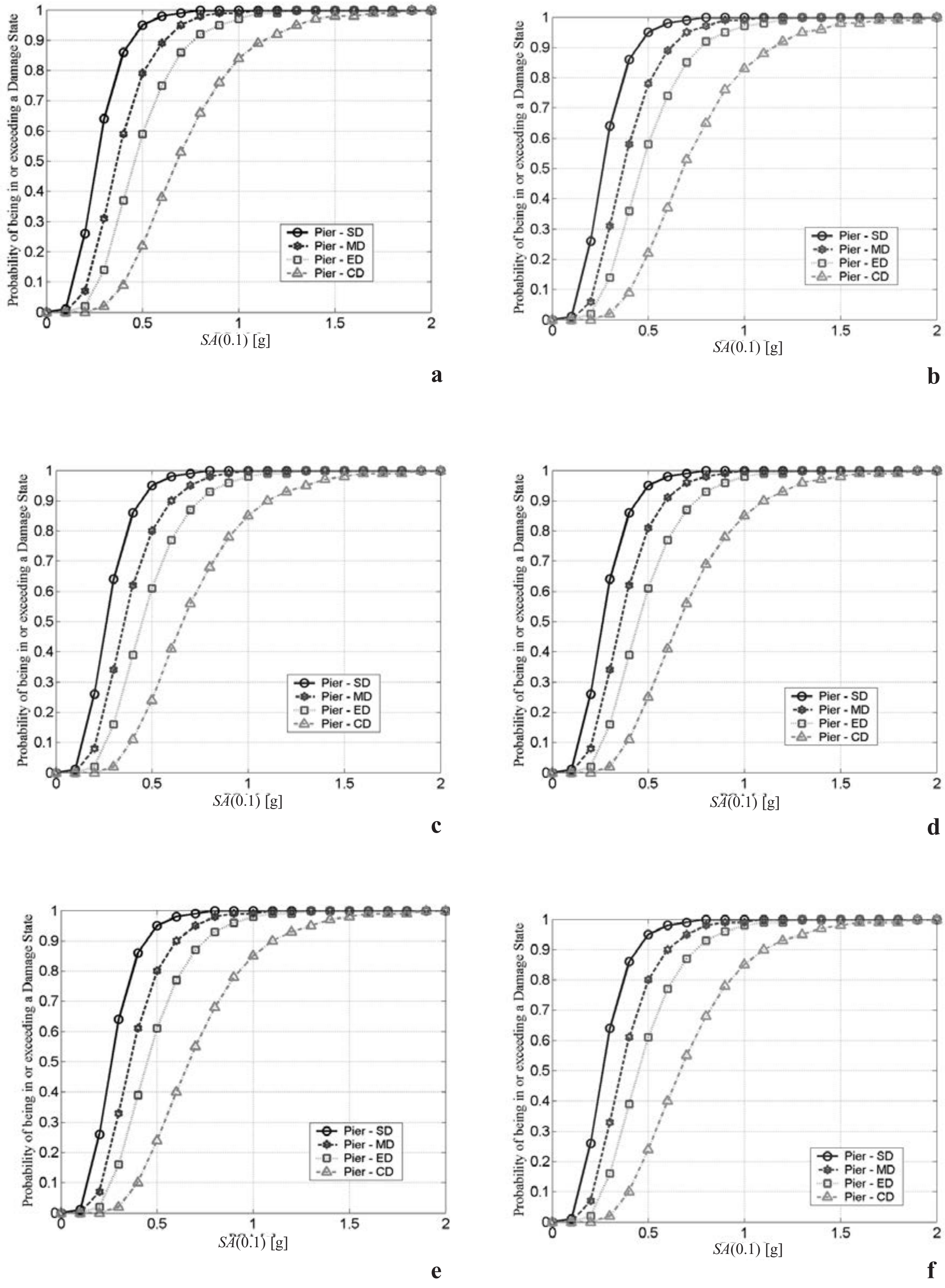

Fig. 5 - Fragility curves for the different *DLs* according to the Hazus99 (FEMA, 2001) procedure: a) Campelli bridge; b) Botteon viaduct; c) San Vendemiano bridge; d) Spresiano highway bridge; e) Quero bridge; f) Fener bridge.

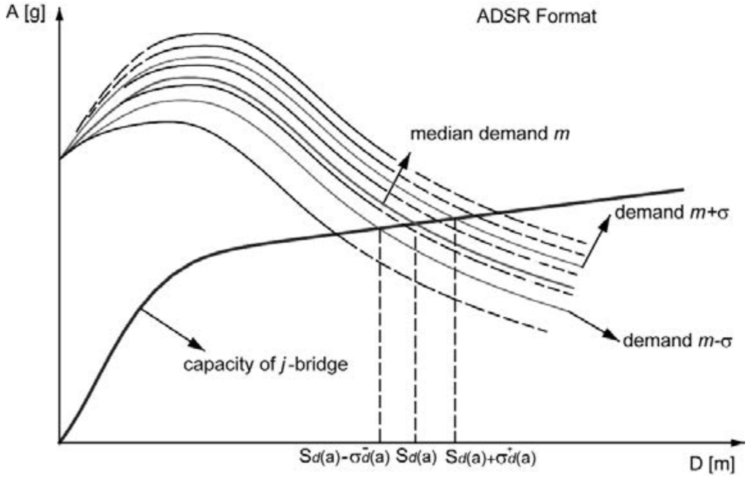

Fig. 6 - Demand spectra in ADRS format for a set of *PGAs*: average spectrum and average spectrum +/- one standard deviation.

Due to the type of bridge analysed, piers were considered to be the elements characterising bridge vulnerability. In order to determine their structural capacity, a non-linear static analysis providing a force-displacement curve was carried out. Transversally, the piers were schematized as:

- cantilever beams (Botteon, Quero);
- framed beams (Fener, Spresiano, Campelli, San Vendemiano).

While, in the longitudinal direction, a cantilever model was always adopted. The piles-foundation lean on rock soil. Therefore, soil structure interaction could be neglected: the piers were schematised as ideal cantilever, or frame, rigidly clamped at the base and free at the top.

This work evaluates pier vulnerability by means of its flexural and shear fragility.

In order to account for any possible performance difference of the types of concrete and steel actually used in the structure, different material strengths were considered. Hypothesising different combinations of material strengths for each bridge, "push-over" curves were obtained: they represent the possible structural behaviour. "Push-over" curves were obtained using the SAP2000 (2002) finite elements code, hypothesising the formation of a plastic hinge for rotation at the base of the pier. The model used was, therefore, made up of "beam" elements with a plastic hinge at the base and a mass at the top, equal to the mass of the deck related to the pier.

By changing variables, the curves issued the capacity curves required for the evaluation of the "performance point". Forty accelerograms (5 time histories for each of the 8 *PGA* values) were used to describe the expected ground shaking in probabilistic terms. They were divided into 8 groups scaled on the *PGA* values 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, and 0.8 g. The relative response spectrum was obtained from each accelerogram to determine the "performance point" using CSM. For each *PGA* value, 5 spectral responses were therefore available, from which the average spectrum and its standard deviation were calculated. Three performance points were obtained from the intersection of these three spectra, which were appropriately reduced following procedure B proposed by ATC-40 (ATC, 1996), with a capacity curve of the pier (for concrete and steel strength values assigned). The values thus obtained were called $S_d(a)$ for the intersection of the diagram of capacity with the average spectrum m, and $S_d(a)+\sigma^{+}_{d}(a)$, $S_d(a)-\sigma^{-}_{d}(a)$ for the intersection with $m+\sigma$ and $m-\sigma$, respectively (Fig. 6).

As the $\sigma^{+}_{d}(a)$ and $\sigma^{-}_{d}(a)$ values do not usually coincide, the standard deviation $\sigma_d(a)$ is redefined as follows to obtain a probability distribution:

$$\sigma_d(a) = \sqrt{\sigma_d^{+}(a) \cdot \sigma_d^{-}(a)} \tag{5}$$

The parameters needed for the log-normal distribution can be obtained from the following equations:

$$\bar{S}_d(a) = c(a)\exp\left[\{\zeta(a)\}^2 / 2\right] \tag{6}$$

$$\{\sigma_d(a)\}^2 = \{\bar{S}_d(a)\}^2[\exp(\{\zeta(a)\}^2) - 1]. \tag{7}$$

The next step to obtain fragility curves was to set the damage levels by means of displacement values.

In this work, four flexural levels were set and identified as multiples of the displacement value at yielding (value directly correlated with ductility of the pier). The element ductility values are directly associated with the section ductilities (μ_1, μ_2, μ_3, μ_4) described in the previous chapter. The ductility parameter shows various increasing values of pier flexural damage.

Shear failure (*SF*) is identified by intersecting the capacity curve with shear strength. The latter can be estimated with the formulas proposed by the seismic code (OPC 3431, 2005). The displacement of the performance point is compared with the displacement of this intersection point.

The definition of these elements enabled the calculation of the probability of the *j*-th bridge, i.e. the probability that a predetermined bridge having the characteristics of the *j*-th material may reach or exceed a flexural or shear damage level identified by d_l using the following equation:

$$P_j\left[S_d(a) \geq d_l\right] = P_j(a, d_l) = 1 - \Phi\left[\frac{\ln\left(\dfrac{d_{l,j}}{c_j(a)}\right)}{\zeta_j(a)}\right] \tag{8}$$

where $c_j(a)$ and $\zeta_j(a)$ are extrapolated from Eqs. (6) and (7). The index *j* indicates that parameters $d_{l,j}$, $c_j(a)$, and $\zeta_j(a)$ depend on the specific bridge analysed (intended as the same bridge physically, but with materials having different characteristics). Different probability values were thus obtained with the varying of the material characteristics, and the final fragility value was extrapolated with an arithmetic mean as follows (Shinozuka *et al.*, 2000):

$$F(a, d_l) = \frac{\sum_{j=1}^{K} P_j(a, d_l)}{K} \tag{9}$$

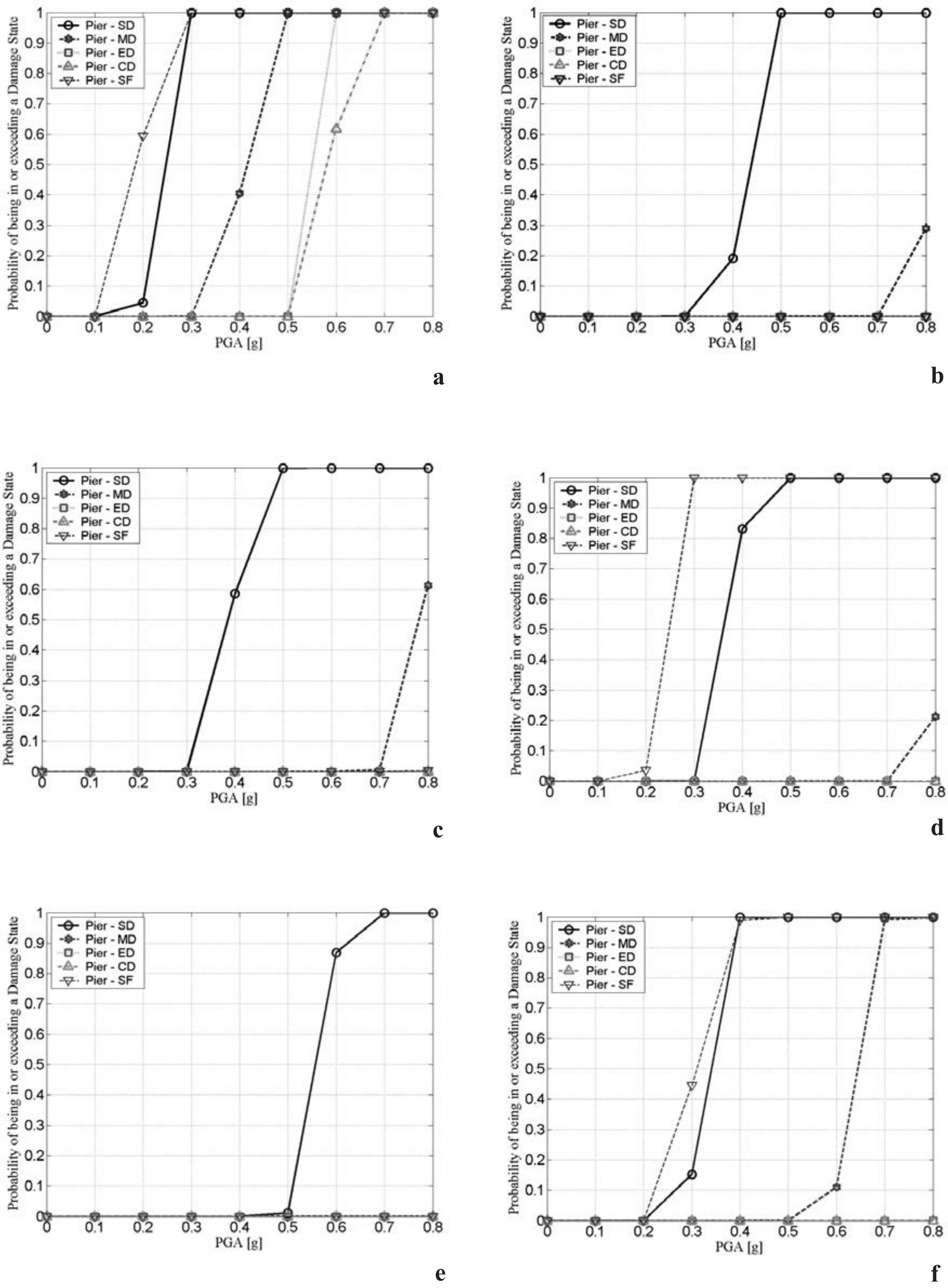

Fig. 7 - Fragility curves for the different *DLs* performed by non-linear static analysis (ATC-40 procedure): a) Campelli bridge; b) Botteon viaduct; c) San Vendemiano bridge; d) Spresiano highway bridge; e) Quero bridge; f) Fener bridge.

where K is the number of variations of material considered.

The fragility curves obtained for the six bridges studied with procedure B are reported in Fig. 7. The figure shows that the fragility curves obtained with the ATC (ATC, 1996) procedure move to the right, providing non-conservative results, and the method is unreliable.

5.3. Construction of fragility curve through a non-linear dynamic analysis

The displacements were obtained directly from SAP2000 (2002) with a more suitable, accurate model specially designed for this type of analysis. Although this model is very similar to that of the static equivalent analysis, a non-linear element of the "non-linear link" type is added, simulating the formation of the plastic hinge. The characteristics of the non-linear element are introduced according to the type of concrete and amount of reinforcement in the section. The non-linear dynamic analysis for each of the 40 (8 *PGAs* × 5 time histories) accelerograms available, and the different material strengths, provided the maximum displacements required, then grouped according to the *PGA* value. For a set value of *PGA*, the Monte Carlo simulation performs 75 (5 time histories × 15 capacity curves) analyses. Then the final fragility curves were obtained at last.

6. Expected damage on the six bridges

Seismic risk is the probability of observing a certain economic loss within a defined time period at the studied site (Ambraseys, 1983). It depends on seismic hazard, vulnerability, and exposed value. Seismic hazard is the probability of observing a certain level of ground shaking in a defined time period at the studied site. Vulnerability is the likelihood that the studied structure may be affected by a certain damage. Exposed value is an economic estimate of the studied structure; it is generally difficult to quantify and, consequently, seismic risk is often replaced by the probability of observing damage of a certain level and is computed considering only seismic hazard and vulnerability. The expected damage is, then, computed by the convolution of the hazard PDF (obtained from the hazard curve) and the fragility curve [for further details see Codermatz *et al*. (2003)].

In this case, the operation was carried out considering both the hazard results of the sites where the six bridges are located (Fig. 3) and the "exact" fragility curves, i.e., fragility curves obtained by non-linear time history analysis of the six bridges (Fig. 8). These fragility curves were obtained for a single structural member, which in bridges is the pier. If all piers are independent, a bridge made up of N piers may exceed a DL with a probability estimated as:

$$P_{f,PL,system}(a) = 1 - \prod_{pier=1}^{N} \left[1 - P_{f,PL,pier}(a) \right]. \tag{10}$$

This hypothesis applies to bridges with simply supported spans, for which each pier can be modelled as a system independent of SDOF.

The expected damage was computed for all the four *DLs* for which the fragility curves were calibrated (Fig. 9).

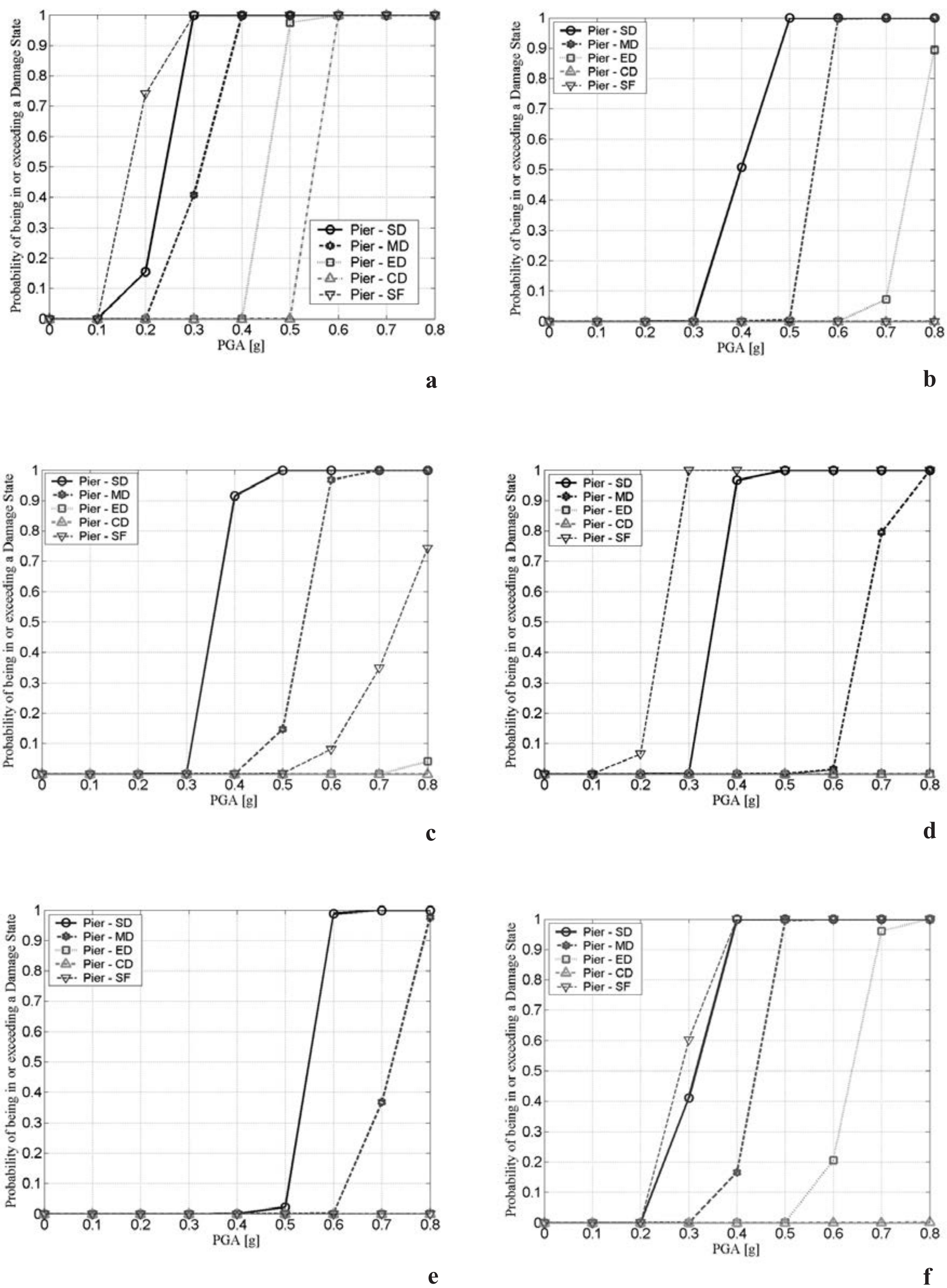

Fig. 8 - Fragility curves for the different *DLs* performed by non-linear dynamic analysis: a) Campelli bridge; b) Botteon viaduct; c) San Vendemiano bridge; d) Spresiano highway bridge; e) Quero bridge; f) Fener bridge.

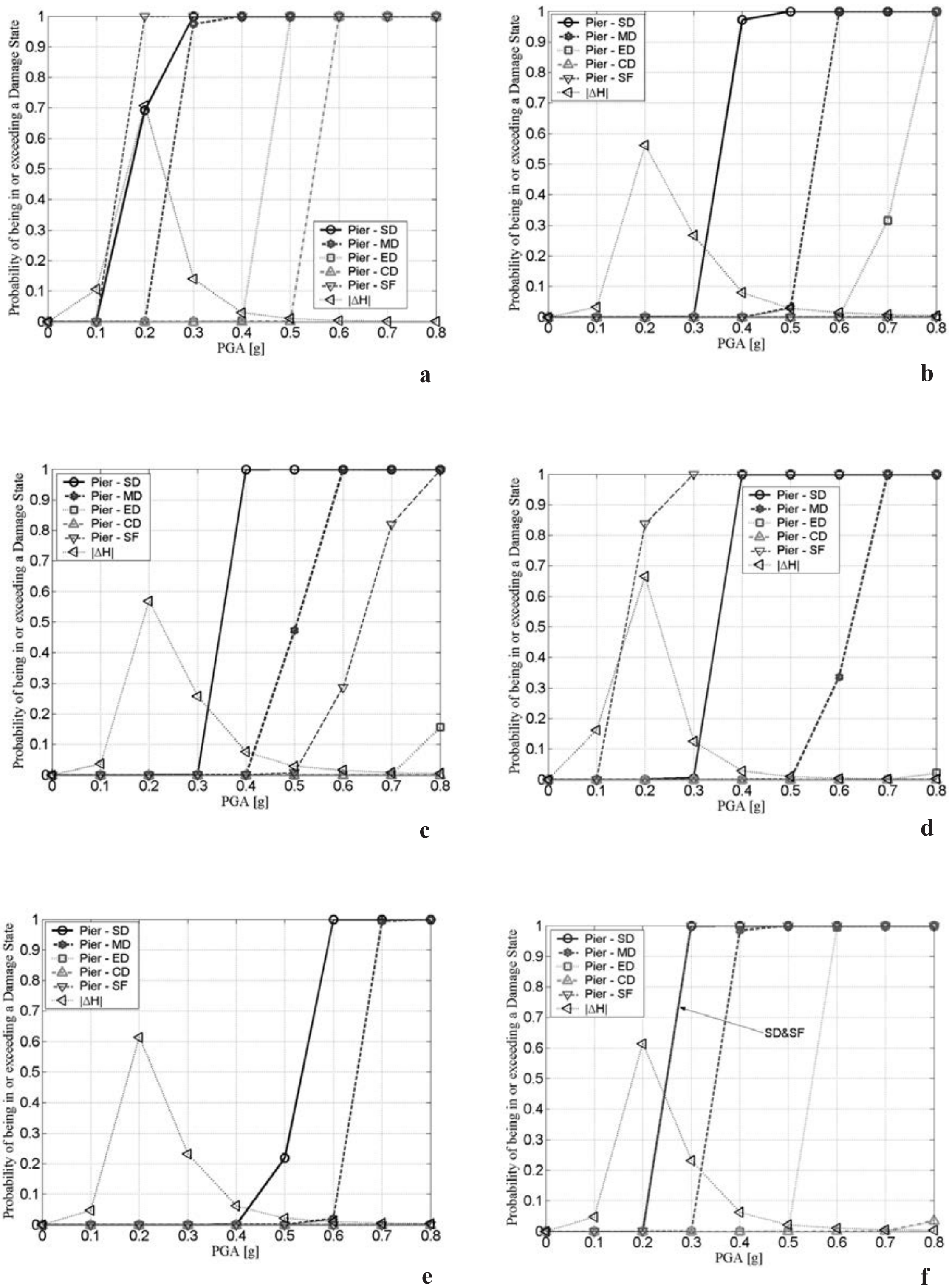

Fig. 9 - Damage assessment. Convolution between the PDF of the hazard curve ($|\Delta H|$) and fragility curves for: a) Campelli bridge; b) Botteon viaduct; c) San Vendemiano bridge; d) Spresiano highway bridge; e) Quero bridge; f) Fener bridge.

Table 2 - Probability of structural damage on the six bridges analysed.

	SD	MD	ED	CD	SF
Fener bridge	0.3340	0.1012	0.0187	0.0001	0.3340
San Vendemiano bridge	0.1303	0.0401	0.0007	10e-9	0.0150
Spresiano highway bridge	0.0461	0.0081	0.0000	10e-8	0.7273
Campelli bridge	0.6748	0.1813	0.0153	0.0051	0.8924
Botteon viaduct	0.1312	0.0266	0.0061	10e-8	10e-9
Quero bridge	0.0234	0.0088	0.0000	10e-7	10e-8

The range of probability obtained is reported in Table 2: it refers to a 100-year time period. The table shows that the range of probability for low damage spans between 0.67 and 0.02 (Grendene, 2006).

7. Conclusions

In this work, the fragility curves for six bridges in the Veneto region were defined considering different approaches. The seismic hazard of the sites where the six bridges are located was computed. Combination of the fragility curve of the bridge and the complete hazard curve of the site where the bridge is located provides the expected damage. The results show that in one case (Campelli bridge) the probability of structural damage is very high and, consequently, reinforcement should be planned.

In addition, the study pointed out a number of interesting aspects related to the construction of fragility curves.

- These applications confirm the limitations of the simplified ATC-40 (ATC, 1996) procedures. Despite being quick in evaluating post-earthquake deformation, they are not sufficiently accurate, leading to underestimation errors of up to 60%. In addition, procedure A does not converge in many cases, and cannot be used as an evaluation method within a system of bridge management. These errors are due to the substitution of the non-linear system with a series of linear systems, that lead to iterate with overly high damping values (up to 40%). It should be noted that transformation of the response spectra into a pseudo acceleration-displacement format is only possible if a simplification is made, which leads to minor differences for damping values of up to 20%, but not more.
- The variation proposed by Chopra and Goel (1999) to iterate on the ductility value instead of on the damping value as provided by ATC-40 (ATC, 1996) is possible only by using simplified response spectra, like those proposed in Eurocode 8 (CEN, 2002), which can be reduced according to ductility. For response spectra obtained as an integration of accelerograms, no reduction method depending on ductility is yet available, and iteration to determine the demand curve is therefore impossible.
- Although determination of the required displacements through non-linear time history analyses is always the most accurate method, it involves careful construction of the model

to be analysed. Particular attention must be paid to the non-linear elements that simulate the formation of plastic hinges.
- As regards fragility curves, the method proposed in HAZUS99 (FEMA, 2001) is extremely fast and easy to apply. These characteristics are essential for a system of bridge management which plans the classification of many structures. However, this method was developed and calibrated on U.S. bridges.
- The construction of fragility curves, based on real accelerograms for the investigated area is, perhaps, the most accurate procedure, although it requires a large number of accelerograms.

In specific cases, the proposed procedure could be adapted for the vulnerability analysis of other bridge typologies. In the case of continuous deck RC bridges, for example, if the so-called structural regularity criteria are satisfied (i.e. high value of relative stiffness deck-pier, symmetric stiffness pier distribution, etc.), the seismic response could be accurately described by an equivalent SDOF system. In this case, fragility curves could be easily obtained by the methods described here, paying attention to some specific peculiarities, i.e the degree of correlation between piles.

In all the other cases, an onerous MDOF system must be performed, but this is beyond the objectives of this paper.

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