SEISMIC RISK ASSESSMENT OF HIGHWAY BRIDGES

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Abstract: In this paper a numerical procedure for the evaluation of seismic risk and vulnerability of highway bridges is described. First, the pushover curve of each structural subsystem (i.e. pier/abutment + bearing/isolation devices) is determined. The contributions of the subsystems are then properly assembled to provide the Capacity Curve of the entire bridge, both in the longitudinal and transverse direction. The Capacity Curve is then step-by-step converted into an equivalent SDOF Adaptive Capacity Curve and intersected with the Demand Curve, represented by an over-damped normalised response spectrum, to provide the PGAs associated to specified damage states for piers and bearing/isolation devices. Based on the PGA values thus obtained, fragility curves (seismic vulnerability) and annual probabilities of exceedance (seismic risk), for a bridge located in a given site, are obtained. The method also gives the possibility to consider possible modifications of strength and ductility, due to decay of materials and/or rehabilitation interventions and/or seismic retrofit interventions.

Key words: Bridge assessment, Seismic vulnerability, Pushover analysis, Fragility curves.

1. INTRODUCTION

The Italian motorway network mainly consists of bridges built between 1960 and 1980. The seismic safety of the majority of the existing bridges is rather uncertain, being based

on old seismic codes, relied upon elastic design philosophy. Recent earthquakes, indeed, have demonstrated the seismic vulnerability of existing bridges, also increased by the slow degradation of the bridge structures which can significantly change their strength and ductility. In order to make a rational decision about the need of retrofitting or replacing an existing bridge, the development of advanced tools for the seismic assessment of highway bridges, which define the seismic risk associated with given performance levels, is needed,. In this paper the background and implementation of a procedure for the seismic assessment of existing bridges is presented. It is based on Adaptive Pushover Analysis for the characterization of the seismic resistance of the structure. The end result of the procedure is a series of Fragility Curves, which describe the seismic vulnerability of the bridge under a probabilistic perspective. Seismic risk is then obtained from hazard maps combined with fragility curves. The proposed procedure can be applied in different conditions, taking account of the current degradation state of the structure, natural evolution of the decay process, programmed maintenance and/or seismic upgrading measures.

2. NUMERICAL PROCEDURE

Figure 1 shows the flowchart of the proposed procedure. Basically, it consists of three phases: (i) derivation of pushover curves, taking into account possible structural decay scenarios, (ii) evaluation of the structural vulnerability and seismic risk and (iii) design and implementation of possible retrofit measures. The procedure has been developed in Visual Basic environment, by exploiting an electronic spreadsheet as graphical interface. As general input data, (i) bridge location (GPS coordinates), (ii) bridge structural typology (simply supported, continuous, Gerber or frame) and (iii) normalized reference response spectrum are required. Subsequently, the bridge geometry and the bridge mass are specified. The deck mass is lumped at the top of the piers, based on tributary areas. If the mass of the piers is large, a tributary mass from the mass pier is considered. For piers with monolithic superstructure connection the two contributions of mass are simply summed. For bridges where the superstructure is supported on bearings, reference is made to a two-mass model to derive the participating mass of the pier-deck system. The algorithm assumes the deck as infinitely rigid. Appropriate geometric constraints between pier/abutment displacements are then imposed to simulate the presence of a rigid deck. Piers and bearing devices are considered to be the critical structural members of the bridge, i.e. those responding inelastically under an earthquake. Abutments and foundations, on the contrary, are assumed to be infinitely rigid and resistant. In the proposed procedure, the following types of piers have been implemented: (i) single shaft, (ii) simple portal, (iii) double portal, (iv) simple frame, (v) interconnected frame, (vi) simple wall and (vii) double wall. As far as the shape of the cross section of the pier columns is concerned, the following options are available: (i) solid or hollow circular section, (ii) solid or hollow rectangular section and (iii) generic section. For this latter, the moment-curvature diagram is uploaded directly from an external file.

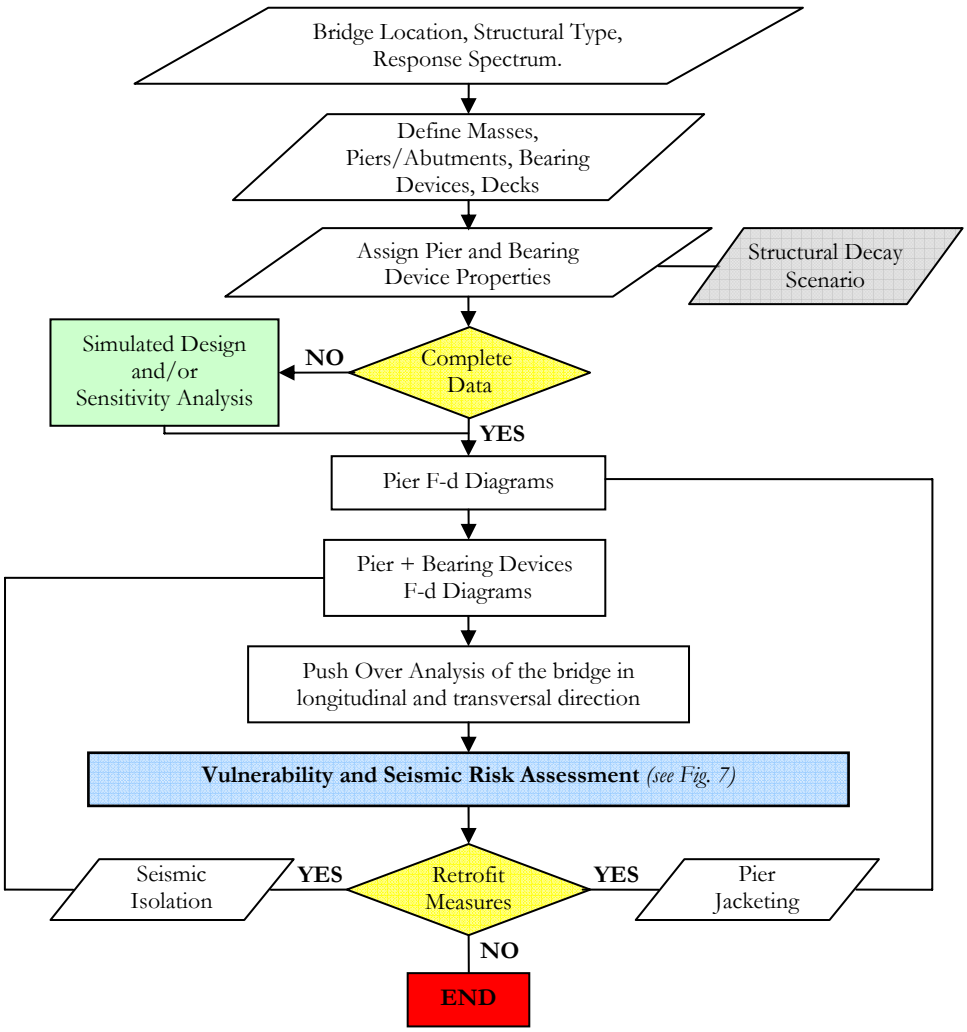

Figure 1. Flowchart of the proposed procedure.

In the input process, the characteristics of piers and bearing devices (including stress-strain relationships of materials, steel reinforcement amount and arrangement, device mechanical properties, etc.) are specified. A routine for managing situations of incompleteness of input data is being implemented. Two different strategies are pursued: either simulated design, when the reinforcement of the piers is unknown, or sensitivity analysis, when the mechanical properties of the devices are unknown. The simulated design is carried out according to the design codes enforced at the construction era of the bridge. In the sensitivity analysis, the bridge assessment is repeated by changing the device parameters within reasonable ranges.

The nonlinear behaviour of the piers is obtained based on moment-curvature analyses of their critical cross sections, taking into account the axial load due to gravity loads and the effects of concrete confinement and steel strain-hardening. Reference to the model of Mander et al. [1988] has been made for confined and cover concrete. The procedure permits to consider different structural decay scenarios, through the use of proper reduction factors, which are applied to concrete strength, diameter of reinforcement bars, thickness of cover concrete, steel resistance and steel ultimate strain, respectively. In the moment-curvature analysis, the pier cross section is divided into a number of fibers, in order to distinguish steel, cover concrete and confined concrete. The curvature of the section is then step-by-step increased and the strain of each fiber evaluated. Values of bending moment and axial load at each step of the analysis are obtained through the Newton-Raphson iterative process. The collapse of the section takes place when concrete or steel ultimate strain is attained. The moment-curvature diagram thus obtained is then properly bilinearized (see fig. 2(a)). In this phase, possible premature failure due to lap-spliced or buckling effects are considered (see fig. 2(a)).

The lateral force-displacement relationship of the pier is derived from the moment-curvature diagram of its critical sections, based on an elasto-plastic pushover analysis, in which the pier is modelled as an elastic beam with plastic hinges at the ends. More precisely, for the cantilever scheme, one plastic hinge at the base of the pier is considered, while, for the shear-type scheme, two plastic hinges are supposed to occur, one at the base and one at the top of the pier. Reference to the equation provided by Priestley et al. [1996] has been made for the evaluation of the plastic hinge length. In this phase, P-Δ effects due to gravity loads are taken into consideration. The shear strength of the pier is then computed, based on well-known equations [Priestley et al., 1998]. It is expressed as a function of the pier top-displacement and compared to the flexural force-displacement behaviour previously obtained (see Fig. 2(b)).

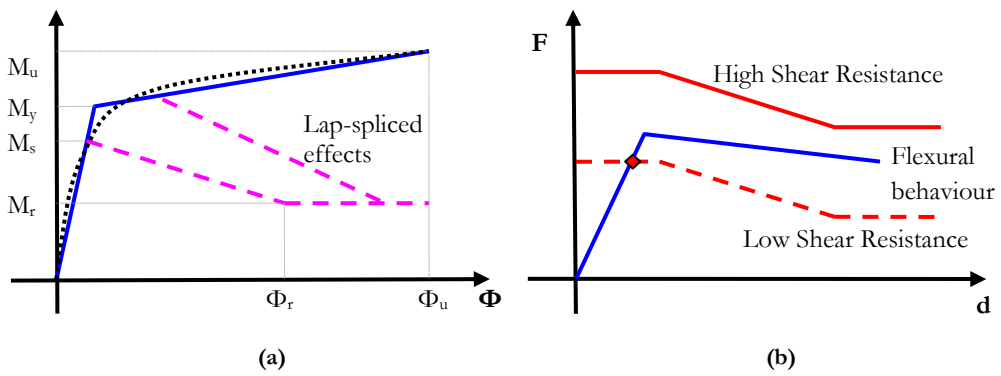

Figure 2: (a) Schematization of the moment-curvature diagram of a flexural plastic hinge, which account for premature failure due to lap-spliced effects, (b) comparison between flexural and shear strength-displacement relationships of a pier.

The nonlinear behaviour of the bearing devices is defined according to the selected typology (see Fig. 3(a)). Five different types of bearing devices are considered, namely: (i) steel hinges, (ii) steel rollers, (iii) neoprene pads, (iv) RC/steel pendulums and (v) steel-PTFE sliders, which can realise three kinds of pier-deck connection, i.e.: fixed hinge, trasversal/longitudinal hinge and multidirectional sliding, respectively. In the model of the pier-deck connection, shear keys and cable restrainers are also considered (see Fig. 3(b)-(c)). Their force-displacement behaviours are combined in parallel with those of the bearing devices, separately in the transverse and longitudinal direction. As device failure occurs, a frictional force-displacement behaviour is employed, up to bridge collapse due to span unseating. The next step of the procedure goes through the assembling of the pier-bearings systems. The force-displacement relationship of each pier-bearings system is derived, by summing up the displacements of pier and bearings under the same horizontal force (see fig. 4).

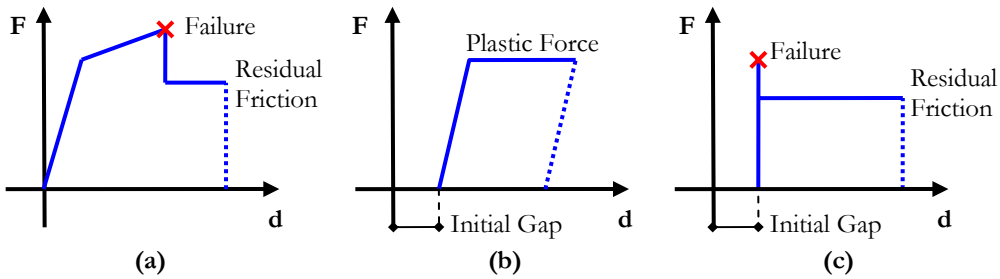

Figure 3: Typical nonlinear force-displacement behaviour of (a) bearing device, (b) cable restrainer, and (c) shear key.

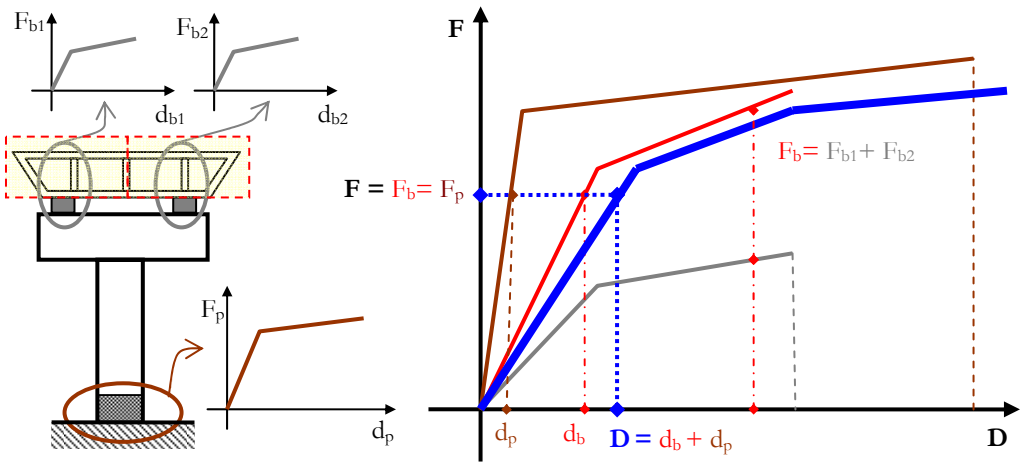

Figure 4: Assembling of pier + bearing devices systems.

Once the lateral force-displacement behaviour of each pier-bearings system has been identified, the response of the bridge in the considered direction (longitudinal or transverse) is examined. The pier-bearings systems are represented by simple inelastic springs with effective stiffness equal to the secant stiffness at the current displacement. During the pushover analysis, the displacement of the stiffness centre of the deck is step-by-step increased (see fig. 5). At each step of the analysis, the spring displacements and associated forces are computed. The effective stiffness of the springs and the position of the centre of stiffness are then updated and a new step of analysis is performed.

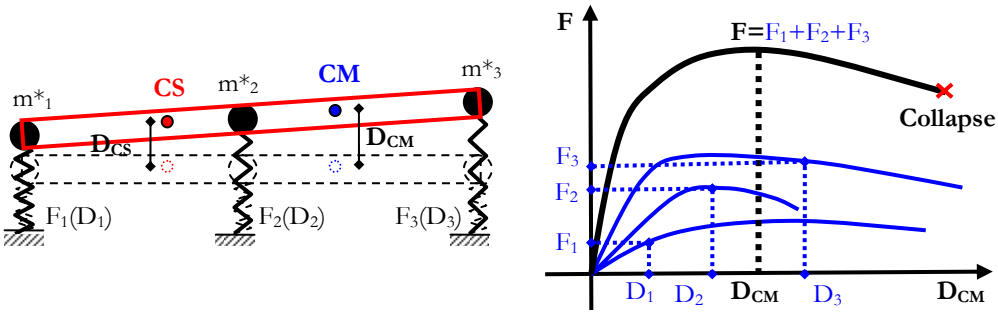

Figure 5: Pushover analysis of the bridge in transversal direction.

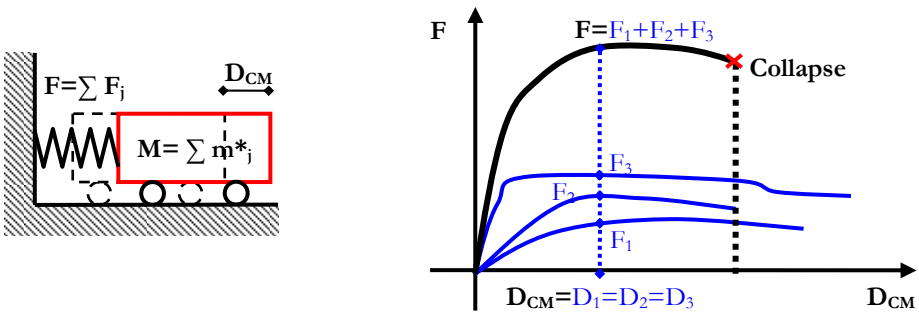

Figure 6: Pushover analysis of the bridge in longitudinal direction.

Actually, the pushover curve of the bridge in the longitudinal direction is simply obtained by summing up the forces of each pier-bearings system F_i at the same displacement D_{CM}. As a matter of fact, indeed, the eccentricity between centre of mass (CM) and centre of stiffness (CS) is zero in the longitudinal direction. The pier-bearings systems work in parallel under the same displacement D_{CM} and the bridge can be modelled as a SDOF system with mass (M) equal to the sum of the participating masses (m_j^*) of the single pier-bearings systems (see Fig. 6). For simply supported bridges, the pushover analysis in the tranverse direction is carried out on independent stand-alone spans, considered as completely separated from the adjacent spans at the separation joints (see Fig. 5). At the

end of the analysis, a diagram showing the total base shear reaction (V) as a function of the displacement of the centre of mass of the deck (D_{CM}) is derived.

The methodology for the evaluation of the seismic vulnerability and seismic risk of bridge structures is schematically summarised in Fig 7. The starting point is represented by the lateral force-displacement relationships obtained from pushover analysis of the bridge (see Figs. 5 and 6).

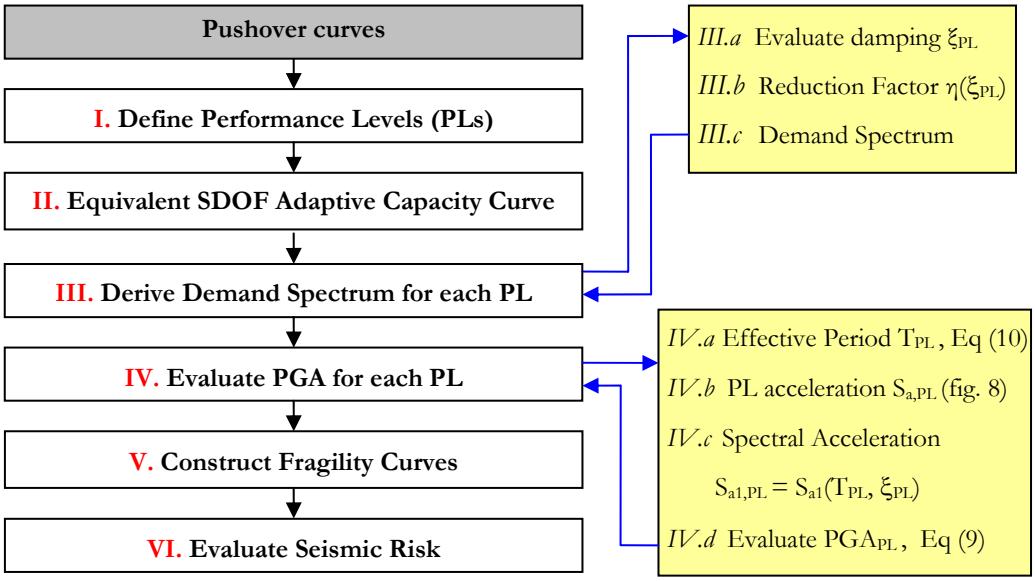

Figure 7: Flowchart of the algorithm for the evaluation of the vulnerability and seismic risk.

The first step of the method (see fig.7) is to define a number of Performance Levels (PLs), for which seismic risk and vulnerability will be evaluated. The PLs are automatically defined on the force-displacement curve of each structural member (piers, bearing devices, shear keys and restrainers), based on predetermined values of the ratio d/d_y. Five PLs are identified for the piers, corresponding to different Damage States (DSs), ranging form no damage ($d = d_y$) to structural collapse ($d = d_u$). The PLs for bearing devices, shear keys and restrainers are selected based on their mechanical behaviour, taking into account their force and displacement capacity. The last PL for the bearing devices corresponds to bridge failure due to span unseating. During the pushover analysis, the displacements of each structural member are monitored. As soon as a given damage state is reached in the first structural member, a point is determined on the pushover curve and a new damage state considered in the continuation of the analysis.

The second step of the method (see fig.7) is to convert the pushover curve of the nonlinear MDOF model of the bridge into an equivalent SDOF "adaptive" pushover curve, referred to as "Adaptive" Capacity Spectrum of the bridge. To this end, the approach recently proposed by Casarotti et al. [2006] has been followed. It combines elements from the Direct Displacement-Based Design (DDBD) Method [Priestley et al. 2003] and the Capacity Spectrum Method (CSM) [ATC-40, 1996]. The Adaptive Capacity Spectrum of the bridge is step-by-step derived by calculating the equivalent system displacement $S_{d,k}$ and acceleration $S_{a,k}$ based on the actual deformed shape of the bridge at each analysis step k, according to equations (1) and (2), where $V_{b,k}$ is the base shear of the bridge, $m^*_{j,k}$ the participating mass of the j-th pier-deck sub-assemblage, $D_{j,k}$ the horizontal displacement of the j-th pier-bearings system at the analysis step k and $M_{e,k}$ the effective mass of the bridge as a whole, calculated according to equation (3).

$$S_{d,k} = \frac{\sum_j m^*_{j,k} D^2_{j,k}}{\sum_j m^*_{j,k} D_{j,k}} \tag{1}$$

$$S_{a,k} = \frac{V_{b,k}}{M_{e,k} g} \tag{2}$$

$$M_{e,k} = \frac{\sum_j m^*_{j,k} D_{j,k}}{S_{d,k}} \tag{3}$$

The aforesaid approach can be viewed as an adaptive variant of the CSM method, because all the equivalent SDOF quantities, even though formally identical to the corresponding modal quantities, are calculated step-by-step, based on the current deformed shape of the bridge, rather than on invariant elastic modal shapes as in traditional CSM. The PLs previously identified on the pushover curves are automatically transferred on the adaptive capacity spectrum.

The third step of the method (see fig. 7) is to determine the seismic demand associated to each PL. Similarly to CSM, the Demand Spectrum is represented by over-damped acceleration-displacement elastic response spectra. This requires the evaluation of the equivalent viscous damping of the bridge associated to each PL. To this end, the following routine has been implemented: (i) choose a given PL, (ii) go back to the pushover database and determine the actual displaced shape of each structural member (basically piers and bearing devices), (iii) evaluate the equivalent damping of each member, based on the Jacobsen's equation [Priestley et al., 2003] specialised to the actual mechanical behaviour of each structural member (see Eqs. (4)-(5)), (iv) combine the contributions of each structural member to get the equivalent viscous damping of the

bridge as a whole (see Eqs. (6)-(7)). The equivalent damping of the bearing devices is calculated based on the following equation:

$$\xi_{b,j} = \frac{E_{visc} + E_{hyst} + E_{fr}}{2\pi \cdot F_{PL} \cdot d_{PL}} \tag{4}$$

in which E_{visc}, E_{hyst} and E_{fr} indicate the energy loss in the device, through its viscous, hysteretic or frictional behaviour, in a cycle of amplitude d_{PL}, being d_{PL} the displacement of the device at the considered PL and F_{PL} the corresponding force level. As far as piers are concerned, reference has been made to the following relationship:

$$\xi_{p,j} = \xi_0 + \xi_{eq} = 0.05 + \frac{1}{\pi}\left(1 - \frac{(1-r)}{\sqrt{\mu}} - r\mu\right) \tag{5}$$

which relates the equivalent hysteretic damping of the pier (ξeq) to its displacement ductility μ and strain-hardening ratio r. The aforesaid relationship has been derived by Kowalski et al. [1995], by applying the Jacobsen's approach to the Takeda degrading-stiffness-hysteretic model. In Eq. (5) a viscous damping ξ_0 = 5% has been assumed. The equivalent damping of each pier-bearings system is then computed, by combining the damping values of pier and bearing devices in proportion to their individual displacements:

$$\xi_j = \frac{\xi_{b,j} d_{b,j} + \xi_{p,j} d_{p,j}}{d_{b,j} + d_{p,j}} \tag{6}$$

Finally, the equivalent damping values of the pier-bearings systems are combined to provide the total equivalent damping of the bridge, for the selected PL. The approach followed in the proposed method is to weigh the damping values of the single pier-bearings systems in proportion to the force acting in each of them:

$$\xi_{PL} = \frac{\sum_{j=1}^{n} \xi_j F_j}{\sum_{j=1}^{n} F_j} = \frac{\sum_{j=1}^{n} \xi_j F_j}{V} \tag{7}$$

Once the equivalent damping of the bridge at each PL is determined, the corresponding demand spectrum is derived from the 5%-damped normalized response spectrum defined at the beginning of the analysis (see Fig. 1), by means of proper damping reduction factors $\eta(\xi)$. The reduction factor to be used can be selected by the designer among different relationships, having the following general form:

$$\eta(\xi_{PL}) = \sqrt[a]{\frac{a}{(b+\xi_{PL})}} \tag{8}$$

The fourth step of the method (see Fig. 7) is to determine the PGA values associated to each PL. From a graphical point of view, this can be done by a translation of the normalised demand spectrum to intercept the capacity spectrum in the performance point (see Fig. 8). From an analytical point of view, the PGA associated to each PL can be determined as the ratio between the acceleration of the capacity curve $S_{a,PL}$ corresponding to each PL (see fig. 8) and the spectral acceleration $S_{a1,PL}$ at the effective period of vibration T_{PL} and total equivalent damping ξ_{PL} associated to each PL (fig. 8):

$$PGA_{PL} = \frac{S_{a,PL}}{S_{a1}(T_{PL}, \xi_{PL})} \tag{9}$$

being:

$$T_{PL} = 2\pi\sqrt{\frac{M_{PL}}{K_{PL}}} = 2\pi\sqrt{\frac{S_{d\,PL}}{g \cdot S_{a,PL}}} \tag{10}$$

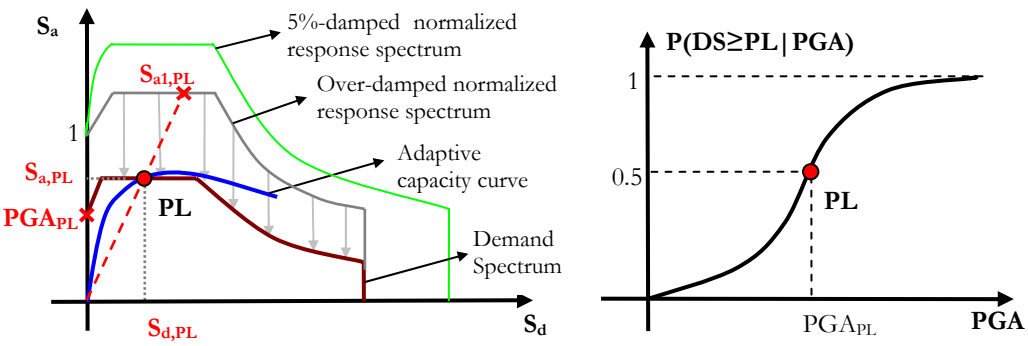

Figure 8: (Left) Evaluation of PGA associated to a given PL and (right) corresponding fragility curve.

The PGA values thus obtained represent an estimate of the median threshold value of the peak ground acceleration associated to each performance level. Starting from these values, a series of fragility curves are produced, one for each PL. The fragility curve of each PL provides the probability of exceedance of that PL, as a function of the PGA of the expected ground motion. In line with other similar proposals [Kappos et al., 2006], in the proposed procedure fragility curves are expressed by a lognormal cumulative probability function:

$$P(DS \geq PL | PGA) = \Phi\left[\frac{1}{\beta_c}\ln\left(\frac{PGA}{PGA_{PL}}\right)\right] \tag{11}$$

in which P(·) is the probability of the Damage State (DS) being equal to or exceeding the selected Performance Level (PL) for a given seismic intensity (PGA), Φ is the standard normal cumulative probability function, PGA_{PL} the median threshold value of PGA associated to the selected PL, as obtained from the previous step of the analysis (see Eq. (9)), and βc the total lognormal standard deviation which takes into account the uncertainties related to the input ground motion, bridge response, etc.. According to previous studies [Kappos et al., 2006]], a value of βc equal to 0.6 has been assumed in this method.

The last step of the procedure consists in the evaluation of the seismic risk through the use of hazard maps, which provide the PGA values at the bridge site having a given probability of exceedance (e.g. 10%) in a given interval of time (e.g. 50 years). The measure of the seismic risk for the bridge under consideration is then given by the probability of exceedance of a given PL conditioned to the local return period hazard. If needed, at the end of the analysis, retrofit measures can be taken. In the current version of the procedure, two different seismic retrofit techniques have been implemented (see Fig. 1): (i) seismic isolation, realised by substituting the existing bearing devices with a suitable isolation system and (ii) confinement of pier through steel, concrete or composite-material jackets [Prietley et al., 1996]. Different types of isolation systems can be chosen in the current version of the procedure. They include: (i) Lead-Rubber Bearings, (ii) High-Damping Rubber Bearings, (iii) Friction Pendulum Bearings, (iv) Combinations of either Low-Damping Rubber Bearings or Friction Pendulum Bearings with Viscous Dampers, (v) Combinations of flat Sliding Bearings and Low-Damping Rubber Devices, (vi) Combinations of flat Sliding Bearings and Elasto-Plastic Devices, (vii) Combinations of flat Sliding Bearings, SMA-based re-centring Devices [Dolce et al., 2000]] and Viscous Dampers. The preliminary design of the isolation system (or pier jacketing) is carried out through an auxiliary routine, which provide the target value of the period of vibration of the isolated bridge (or displacement ductility of the piers) to satisfy a given PL under a reference PGA (e.g. that provided by the national seismic code for that seismic zone, with a given probability of exceedance in 50 years).

3. CONCLUDING REMARKS AND FUTURE DEVELOPEMENTS

A numerical procedure for the seismic assessment of existing bridges has been presented. It is inspired to the principles of the Capacity Spectrum Method, reviewed under an "adaptive" perspective. The most important features of the procedure are as follows: (i) great versatility in the consideration of structural types of decks, piers, pier-deck connections and bearing devices, (ii) use of accurate models to describe the mechanical behaviour of the structural elements which are more vulnerable from the seismic point of

view (i.e. piers and bearing devices), (iii) use of adaptive pushover analysis for the evaluation of the seismic resistance of the bridge in the longitudinal and transverse direction, (iv) ability to operate for different performance levels, (v) derivation of fragility curves for the probabilistic description of the seismic vulnerability of the bridge, (vi) possibility to account for different structural decay scenarios, (vii) possibility to employ different strategies for the seismic risk reduction. A routine for dealing with cases of incompleteness of input data, through two different approaches (i.e. simulated design or sensitivity analysis) is being implemented. At the moment, the procedure is going to be applied to a number of existing bridges and the results compared to those provided by nonlinear time-history analyses.

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