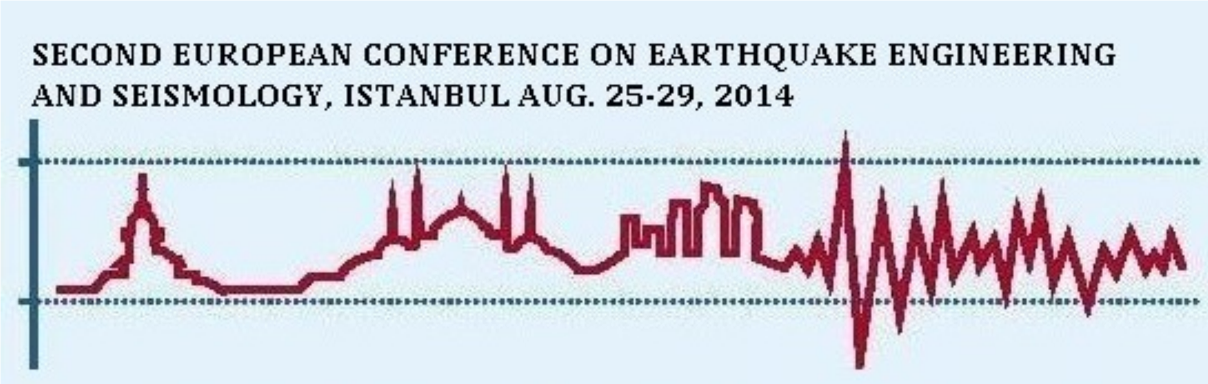

DIFFERENT APPROACHES TO DERIVE ANALYTICAL FRAGILITY FUNCTIONS OF BRIDGES

Ricardo MONTEIRO[1], Xiaoxuan ZHANG[2] and Rui PINHO[3]

ABSTRACT

Fragility functions play a critically significant role in the assessment of seismic loss thus the development of reliable procedures for their calculation has become increasingly popular. The existence of different analytical methodologies, as well as structure modelling approaches, has an important influence in the results of fragility functions. In addition it has been recognised that bridges represent one of the most vulnerable structural types during past seismic events and, depending on the seismic conditions of the local site, seismic vulnerability assessment of bridges can be carried out based on fragility curves. Many tools are currently available for calculating fragility curves, especially based on analytical approaches, which are many times preferred due to its scientific soundness and because they overcome the lack of data in empirical approaches. Both static and dynamic nonlinear analysis can be employed to derive fragility curves within an analytical approach and both feature, in turn, different possibilities of application as well as many sources of uncertainty. In this study, different nonlinear static procedures (NSPs) are considered to develop analytical fragility functions of RC bridges. The performance of the different NSPs, when applied to a large number of randomly generated bridge configurations, is compared with results of an extensive number of nonlinear dynamic analyses, considered as reference, in order to identify the most accurate and efficient method for calculating fragility functions. The results have shown that, among the seven selected nonlinear static procedures, N2 provides the closest to dynamic analysis estimations, featuring also with the lower computational onus efforts.

INTRODUCTION

Many tools are currently available for calculating fragility curves, especially based on analytical approaches, which has become increasingly popular in earthquake engineering community due to its scientific soundness and because they overcome the lack of data in empirical approaches. Both static and dynamic nonlinear analysis can be employed for deriving fragility curves within an analytical approach and both of them feature numerous different possibilities of application as well as many sources of uncertainty. In this study, different nonlinear static procedures (NSPs) are considered to develop analytical fragility functions of RC bridges. The performance of the different NSPs, when applied to a large number of randomly generated bridge configurations, is compared with results of an extensive number of nonlinear dynamic analyses, used as reference, in order to identify the most accurate and efficient method for calculating fragility functions.

In the past few decades there has been a significant research effort regarding fragility functions for buildings whereas, clearly, less thorough investigations have been carried out for the

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study of bridges. Fragility functions of bridges is thus a topic that still undergoes significant development and improvement, particularly in what concerns the classes of bridges under scrutiny and subsequent loss assessment studies. Generally, there are several methodologies for calculating fragility functions and the resulting curves are conditional on the assumptions and approaches followed in the process, which includes the parameters used in the modelling, definition of the damage states, computational approaches, etc. These difference caused by different methodologies will lead to critical discrepancies in the seismic assessment, even when considering the same region. In addition, the limitations of the previous studies include the use of a unique methodology or a reduced number of structural configurations.

In this study, in order to improve such drawbacks, several analytical methodologies are employed to calculate fragility functions for the same bridge type in the same region. Nonlinear dynamic analysis is used as benchmark for the comparison with seven different NSPs (Capacity Spectrum Method, N2, Displacement Coefficient Method, Modified Modal Pushover Analysis, Adaptive Capacity Spectrum Method, Adaptive Modal Combination Procedures and Modified Adaptive Modal Combination Procedure), which make use of conventional and adaptive pushover analysis. The selected case study is made up of 3D analytical RC bridge models, generated using OpenSees (2013), based on statistical distributions for the different material and geometrical parameters. A sufficiently large, thus representative, ensemble of RC bridges is considered in order to duly account for the uncertainty associated to the use of such methodologies. Nonlinear analyses are conducted for each bridge model under different ground motion records and curvature ductility is employed as the engineering demand parameter to determine the bridge damage limit states in terms of structural capacity.

CLASSIFICATION AND GENERATION OF BRIDGE POPULATIONS

The first step to adopt when calculating fragility curves for seismic loss assessment of a population of bridges can be their classification into typological classes, based on simple parameters that characterize their configuration and thus their seismic response. Past studies have selected and categorized bridges based on the characteristics of the piers, bearings and pier-to-bearing connections (Moschonas et al., 2009) or other parameters with significant influence on seismic response, such as skew angle, number of spans and number of bent columns (Avsar, 2009). Afterwards, for each chosen parameter, a statistical probabilistic distribution can be defined and used for random generation of bridges. Typically used distributions are normal, log-normal, gamma, beta or exponential distributions. In this study, in order to keep the computational time and effort within a sustainable range, a single geometrical configuration of bridges with four bays was considered however the length, height and configuration of the piers were all submitted to variation, according to pre-defined distributions. In addition, uncertainty was also considered to come from the material and other geometrical properties of the bridges, according to the distributions presented in Table.1. The distributions assumed for the different variables intend to represent a scenario of typical RC bridges and viaducts in Italian territory.

Table 1. Distribution of material and geometrical properties

Parameters	Mean	COV (%)	Lower bound	Upper bound	Type of distribution
Steel modulus (GPa)	200	3	–	–	Normal
Steel yield strength (MPa)	371.1	11	250	–	Normal
Concrete strength (MPa)	40	20	20	70	Gamma
Column height (m)	15	15	8	25	Normal
Column diameter (m)	2	14	1.2	4.79	Lognormal
Span length (m)	50	20	30	80	Normal
Beamcap width (m)	5	4	2	20	Lognormal
Beamcap height (m)	1	2	1	4	Lognormal

Once the statistical distributions for all the relevant parameters have been defined, the bridge models can be sampled either following Monte Carlo or Latin Hypercube approaches. In order to get a reliable sample, able to represent the probability distribution as good as possible, a proper simulation procedure should be applied. Furthermore, the size of the sample will play a significant role towards the appropriateness of the simulation procedure. Among the available solutions, the most commonly used method is the Monte Carlo approach based on pure random simulation whereas the Latin Hypercube sampling scheme is a more recent and innovative technique, initially proposed by Mckay et al. (1979), and further developed by Iman et al. (1980). One of the most important claims of the Latin Hypercube approach is that it reduces the size of the sampling efficiently. In this study, an automatic framework was created to randomly simulate the selected parameters (material and geometrical) for each individual bridge. As a first approach, parameters are randomly generated based on the distributions defined above using the Monte Carlo simulation scheme.

Bridge modelling

Proper analytical modelling of bridges is of great importance when calculating seismic fragility curves and it still encloses a number of challenging issues, among which some need to be further developed and others call for reasonable assumptions. Special attention should be paid when making simplified assumptions since any simplification will have a direct impact on the analysis and, consequently, on the reliability of resulting fragility curves. On the other hand, an excessively refined or complex model will lead to higher computational demand or even unreachable results. In this study, OpenSees has been employed to build all the models (and to carry out all the static and dynamic nonlinear analysis required by the analytical procedures), using force-based fibre elements with distributed nonlinearity. The reason for choosing OpenSees to perform this study is that it enabled the settlement of an automatic computation platform, allowing the calculations to be performed automatically for a large number of records and bridge configurations.

With respect to the bridge structural model itself, a brief description of the assumptions made for specific components are described next. Abutments are commonly modelled using equivalent linear springs that simulate the deck restrains provided by the abutments. The characterization of the equivalent springs should represent the dynamic behaviour of the soil behind the abutments, the structure components of the abutments and the interaction between them. The work presented herein did not have the scope of going deep into abutment design but rather choosing a reasonable model to define the abutments of the bridges used in the extensive fragility curve parametric study. Indeed, no uncertainty related to the abutments has been considered within the random simulation phase (see Table.1), although it should be seen as a future development. Accordingly, for this study, the type of abutments considered has been the one sitting on top of pot bearings, allowing a maximum elastic deformation of 500mm. The stiffness of the abutments was set as 26’329kN/m, according to what proposed in the study of Casarotti et al. (2005). Decks are typically modelled as linear elastic elements, which represent the expected behaviour of this structural component under seismic actions. Given that it is often made up of pre-stressed, rather than regular reinforced concrete, no damage or nonlinear deformation is expected to occur. In this study, the deck was modelled with a 3D elastic element, whose sectional properties are described in Table.2.

Table 2. Deck cross section geometric and material properties

EI2 (kNm^2)	EI3 (kNm^2)	GJ (kNm^2)
$1.32x10^8$	$1.32x10^8$	$4.75x10^9$

GROUND MOTION RECORDS

Ground motion records are generally recognized as the essential input for any seismic assessment based on nonlinear dynamic analysis. Typically, real records are employed in dynamic analysis supporting seismic assessment studies. It is generally recommended that a sufficiently large number of different records is used in order to duly represent an intended seismic hazard scenario, characterized by predefined ranges of parameters such as magnitude and peak ground acceleration (PGA), most common fault failure mechanism, frequency content, duration and epicentre distance. To achieve the goal of this study, which did not refer to a specific region or specific bridges, the ground motion records were collected in terms of peak ground motion acceleration (PGA), as well as the distance and magnitude, taking into account the historical seismicity in Italy. The majority of the strongest earthquakes that occurred in Italy featured dip-slip mechanism and some of those events were associated to strike faults. The average depth of an Italian damaging earthquake is 10km. Based on the data from Catalogue of Strong Earthquakes in Italy (Boschi et al., 2000) from 461 B.C. to 1997, the simplified chart in Fig.1 shows the distribution of those historical earthquakes, in terms of magnitude.

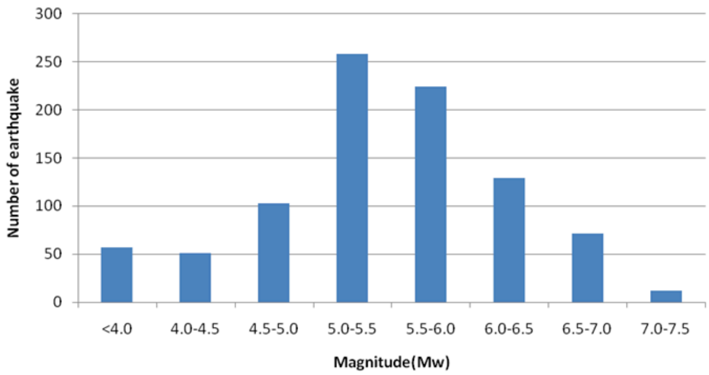

Figure 1. Distribution of magnitude of Italian historic earthquake

The parametric study presented herein considered non-scaled records, directly chosen from one of the available ground motion databases, restricted to the predefined ranges of magnitude and epicentre distance. A sufficiently large number of records (50) were considered, so as to duly incorporate the epistemic/aleatory uncertainties. In addition, a uniform distribution of the peak ground acceleration, within a representative range (e.g. 0 – 1g), was sought. When following this sort of approach, each record is used only once, given that the entire range of ground motion intensity (measured in terms of peak ground acceleration) is covered by the different time-histories. Given that there is no need for scaling, the fact that the records keep their original form can be seen as a major advantage. On the other hand, each ground motion intensity level is somewhat represented by a single accelerogram hence not taking into account record-to-record variability.

DAMAGE LIMIT STATES

A large number of studies have addressed the definition of damage states for the specific seismic assessment of bridges. For example, according to Hwang et al. (2001), element-level limit state criteria are more suitable when detailed seismic damage assessment is required, such as the assessment of the seismic retrofit scheme of a bridge. For other purposes, such as seismic fragility analysis of bridge populations, as intended in this study, other structure-level approaches, based on the global damage of the bridge, could be applied to define damage states. The chosen engineering demand parameter, used for quantification of damage of each pier, was ductility in displacements. On a simplified fashion, the highest ductility demand among the different bridge piers was taken as representative of the behaviour of the entire bridge. In this study, the definition of each damage limit state followed the description of damage states proposed in HAZUS (2003) thus four damage states were considered – slight, moderate and extensive damage and collapse. The use of strain measurements from nonlinear numerical analysis can be seen as of questionable accurateness hence curvature, a parameter far more stable, has been chosen to check damage limit states occurrence. The approach by Neilson (2005), who used curvature ductility to check each limit states, was adopted to define each of the four damage states. The corresponding ductility in curvatures is presented in Table.2.

Table 2. Four damage limit states from Neilson et al. (2005)

	Slight	Moderate	Extensive	Collapse
Ductility in curvature	1.29	2.10	3.52	5.24

Accordingly, displacement ductility can be calculated based on the corresponding observed curvature ductility using Eq.(1) and Eq.(2), as proposed by Priestley et al. (1996).

$$\mu_{\Delta} = 1 + 3\left(\mu_{\phi} - 1\right)\cdot\left(l_p/l\right)\cdot\left[1 - 0.5\cdot\left(l_p/l\right)\right] \quad (1)$$

$$l_p = 0.08l + 9d_b \quad (2)$$

When the different damage states are very distant from each other, in terms of the corresponding values for ductility, the allowance range for each damage state will be larger, and so will be the tolerance for differences between each method, which means that the comparison between methods, in terms of fragility functions, could conceal important differences.

NONLINEAR STATIC PROCEDURES

Nonlinear static procedures (NSP) represent a simplified approach for the assessment of the seismic behaviour of structures based on pushover analysis and have become very popular within the engineering community as they have proved, when compared to nonlinear dynamic analysis, to provide reasonable estimates for the seismic response of bridges, requiring less time and computational effort (Pinho et al., 2009; Casarotti et al., 2009; Monteiro, 2011). NSPs can currently be found in codes and guidelines, such as ATC-40 (ATC, 1996), FEMA-273 (ATC, 1997) or the European Code (CEN, 2005a, 2005b). There is a group of pioneering methods, corresponding to the first proposals of nonlinear static analysis based procedures, which have led to reasonably accurate results. Capacity Spectrum Method (CSM), introduced by Freeman et al. (1975) and implemented in ATC-40 guidelines, is one of those. Similarly, the N2 method, has been proposed by Fajfar and Fischinger (1988) and included afterwards in the recommended simplified procedures in European Code (CEN, 2005a). Recently, an improved version of CSM has been presented in FEMA-440 guidelines (ATC, 2005), including updated empirical equations to estimate equivalent viscous damping and spectral reduction factor. Displacement Coefficient Method (DCM) was initially introduced in ATC-40 and provides a considerable simple empirical equation to calculate the seismic response of structures using different specific coefficients. Given that those first methods fail to take higher modes into consideration, Modal Pushover Analysis was proposed by Chopra and Goel (2001) and has been modified later on according to the seismic behaviour of higher modes (Chopra et al., 2004). On the other hand, with the development of adaptive pushover analysis algorithms, the methodologies developed on adaptive or fully adaptive scope enjoyed an increasing development. Different adaptive pushover based methods were recently proposed by different scholars, such as the Adaptive Capacity Spectrum Method (ACSM) (Casarotti and Pinho, 2007) or the Adaptive Modal Combination Procedure (AMCP) (Kalkan and Kunnath, 2006), together with its recently modified version (MAMCP) (Shakeri et al., 2013).

The response estimates obtained from the seven aforementioned nonlinear static procedures (CSM, DCM, N2, MMPA, ACSM, AMCP and MAMCP) were compared to nonlinear dynamic analyses, considered as baseline, at two levels. The first directly compared static and dynamic engineering demand parameter estimates to calibrate each NSP with respect to its different variants, whereas the second analysed the relative performance in terms of fragility curves. The overall procedure that has been followed to assess the performance of the different NSPs, when calculating fragility curves, can be summarized in the following steps.

1. Random generation of a population of 3D bridge models using Monte Carlo simulation;
2. Pushover analysis of each bridge and conversion to the equivalent SDOF system curve;
3. Estimate nonlinear target displacement for each bridge, for each of the selected Nonlinear Static Procedures, using a large set of ground motion records;
4. Identification of the global damage state based on the nonlinear response;
5. Representation of the cumulative percentage of bridges in each damage state versus the value of the intensity measure corresponding to each record;
6. Regression analysis to calculate the parameters defining the fragility functions.

RESULTS

The validation of each NSP was carried through direct analysis of the parameter Bridge Index (BI) defined according to Eq.3 as the median of the ratios between the response quantities, at different bridge locations, estimated through a specific NSP and the maximum response quantity estimated with nonlinear dynamic analysis. The response of the bridge for a given intensity level can be measured at different locations thus the parameter Bridge Index (BI) was chosen to directly compare the different methods, based on the response parameters at the performance point (PP).

$$BI = \frac{\Delta_{NSP}}{\Delta_{Dynamic}} \tag{3}$$

The second level of results regards fragility functions using nonlinear static procedures. Even though almost all available guidelines suggest the use of smoothed design spectra to calculate performance points for NSPs this study used the ground motion records selected from the PEER database, which are real, non-scaled records.

With respect to the size of the population of bridges, in order to guarantee representative results, a brief preliminary parametric study was carried out, testing statistical convergence. In this study, 10'000 RC bridges were generated to estimate the maximum displacement of the middle pier and the mean value of those 10'000 bridges was assumed as the exact solution. Afterwards, samples of different sizes were generated (from 5 to 300), and the corresponding mean values were used to calculate the relative error, define as the difference between the mean value derived from each sample and the exact solution divided by the latter. The results indicated that when the size of sample is larger than 170, the relative error remains below 4% thus, conservatively, the size of the samples to use as case study has been chosen as 200.

Bridge Index (BI) for NSP predictions

For the assessment and calibration of each NSP, nonlinear static and dynamic analyses were run on 200 bridges with ten of the fifty selected records (PGA ranging between 0.2g and 1g).

In terms of individual NSP calibration, using pushover analysis that makes use of 1st mode shape proportional or uniform load patterns yields similar results, within the employment of CSM. The observation of the N2 results led to similar conclusions to what has been found for CSM: the 1st mode proportional load pattern yields slightly more accurate predictions. The BIs obtained from DCM-based estimates show that, again, the 1st mode proportional load pattern leads to better predictions, which clearly states for the significant influence of the choice for the lateral load patterns of pushover analysis on the prediction of the bridge performance point. Fig.2 summarizes the median BIs obtained when employing each of the tested NSPs in its optimal version.

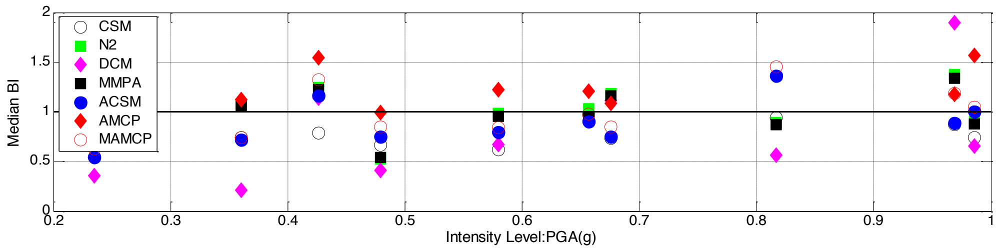

Figure 2. Median Bridge Index per intensity level for all seven methods

Generally, NSP-based response estimates of bridges get closer to the dynamic-based ones, thus more accurate, with increasing intensity level. In terms of relative performance of the different nonlinear static procedures, one can conclude that, in general, the "classic" procedures (CSM, DCM) tend to underestimate the response. In addition, it can be seen that DCM has a very variable behaviour within the considered intensity range, whereas CSM can be seen as the method with the steadiest behaviour, although it sometimes heavily underestimates the response displacement estimates. One of the possible reasons for the underestimating trend may be the fact that CSM uses equivalent viscous damping, which may easily be overestimated. As far as DCM is concerned, this NSP was initially proposed for buildings and some of the coefficients are that relevant for bridges thus the results derived from this procedure might be compromised. On the other hand, Fig.2 shows that the so-called innovative NSPs do bring in some improvement, given that BIs are definitely closer to unity. In spite of the fact that MAMCP considers the higher modes as elastic, ACSM yields somewhat similar results, which indicate that within the use of adaptive modal pushover analysis, the 1^{st} mode contribution is more important than the one from higher modes. MMPA takes higher modes contribution into consideration, which does seem to have led to better predictions, especially when compared to other methods that make use of 1^{st} mode based conventional pushover analysis. Similarly to CSM, but contrarily to N2 and MMPA, ACSM is a method that relies on the estimation of damping, and spectral reduction factor. In spite of making use of an adaptive pushover analysis, ACSM did not yield particularly more accurate predictions, exhibiting a general underestimating trend. As has been observed for other procedures, the accuracy of the ACSM predictions increases with the increasing of the intensity level. With respect to AMCP, considering both higher modes effects and post-yield stiffness degradation, it provides fairly good estimates for lower intensity levels whilst for higher intensity levels, considering the higher modes contribution through SRSS combination for the final response parameter prediction, AMCP tends to slightly overestimate nonlinear dynamic analysis. Finally, MAMCP provides slightly overestimating indexes for high levels of intensity.

Fig.3 plots the global Bridge Index, calculated across all the bridges and all the intensity levels, for every nonlinear static procedure applied in this study.

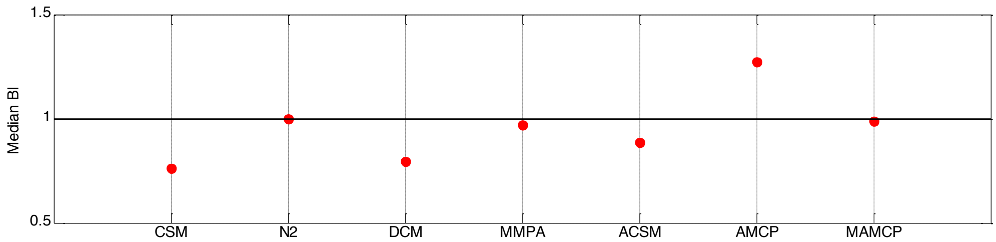

Figure 3. Global median Bridge Index for each NSP

MMPA, ACSM and, particularly, N2 and MAMCP stand as the best performing procedures, yielding global BI ratios practically equal to one, i.e., static-based estimates that globally match the dynamic ones. If such best performing procedures are considered, the range in which the majority of the estimates fall within becomes -/+10%.

Based on the results presented so far, even though adaptive pushover analysis considers the actual stiffness of the structure at each step, it still yielded underestimated predictions, which might be due to a further needed calibration of the equivalent viscous damping model. On the other hand, the N2 method, which makes use of a constant load pattern and inelastic spectrum, provided fairly good estimates. A similar scenario was found for the application of MMPA, which also make use of inelastic spectrum but takes higher modes into consideration. With respect to other characteristics of the procedures, such as computational and time demand, N2 would be preferable. It is worth mentioning that the proposed MAMCP also provides comparatively good estimates, coupled with considerable time saving by accounting for higher modes in elastic regime.

Fragility Curves

Response estimation for fragility curves calculations was based on statistic analysis and lognormal distribution has been assumed for each fragility curve as a reasonable choice. The procedure is described as follows:

1. Derive the linear regression function of the relationship between natural logarithm distribution of probability of exceedance and ln(PGA);
2. Calculate $\sigma = \dfrac{1}{m}$, $\mu = -b \cdot \sigma$ according to the linear function;
3. Calculate the lognormal distribution for each PGA and get the curve.

The global displacement ductility for each bridge has been assumed as the highest among the different piers and the same set of 50 ground motion records was used for calculating fragility functions. The fragility functions, calculated for the different limit states using the predictions of the different NSPs and the nonlinear dynamic analyses are presented in Fig.4. Again, each NSP was applied in its optimized fashion, according to the results described previously.

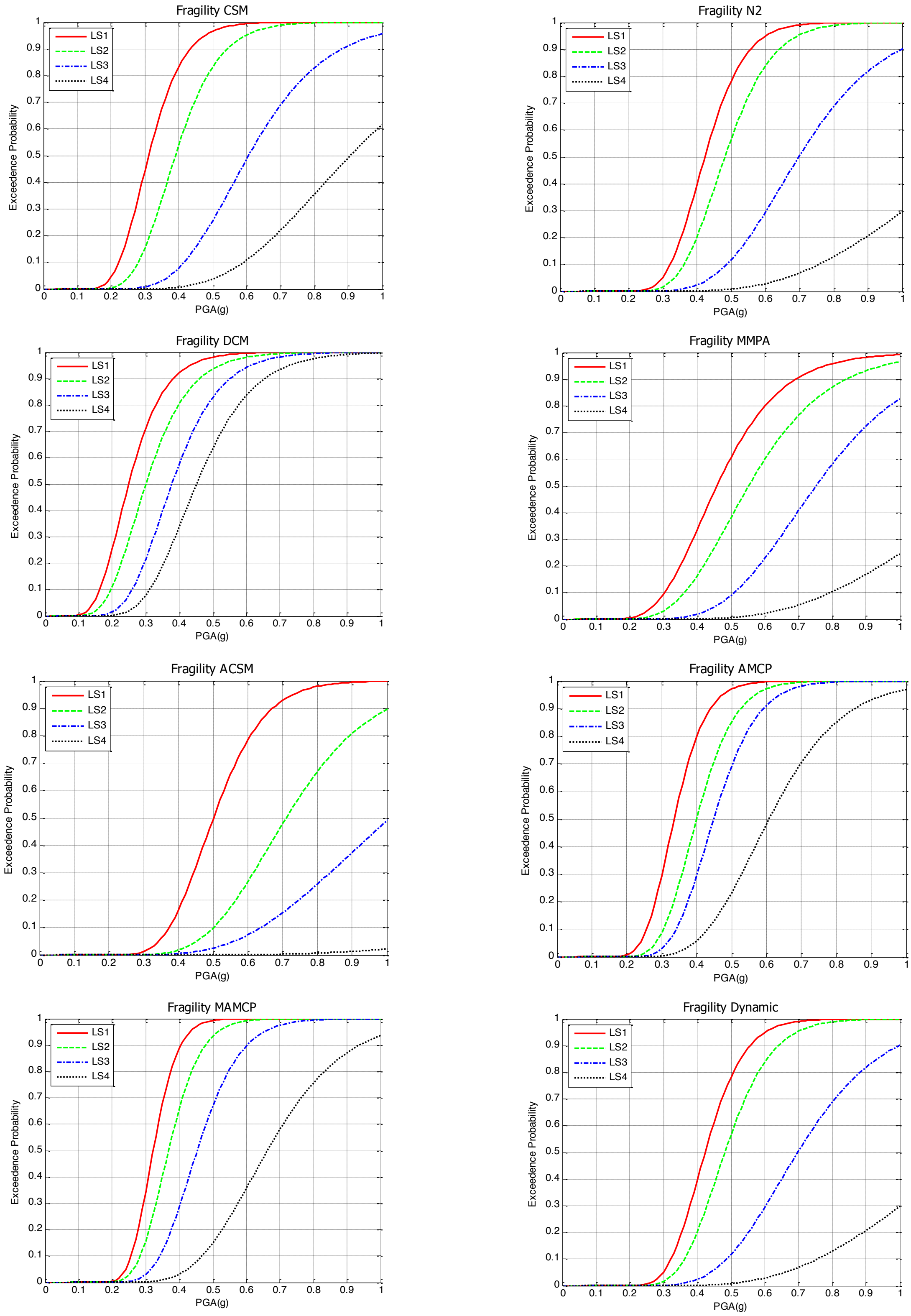

Figure 4. Fragility curves based on nonlinear static and dynamic analysis

The comparison of static- and dynamic-based fragility curves, for each of the limit states, is illustrated in Fig.5.

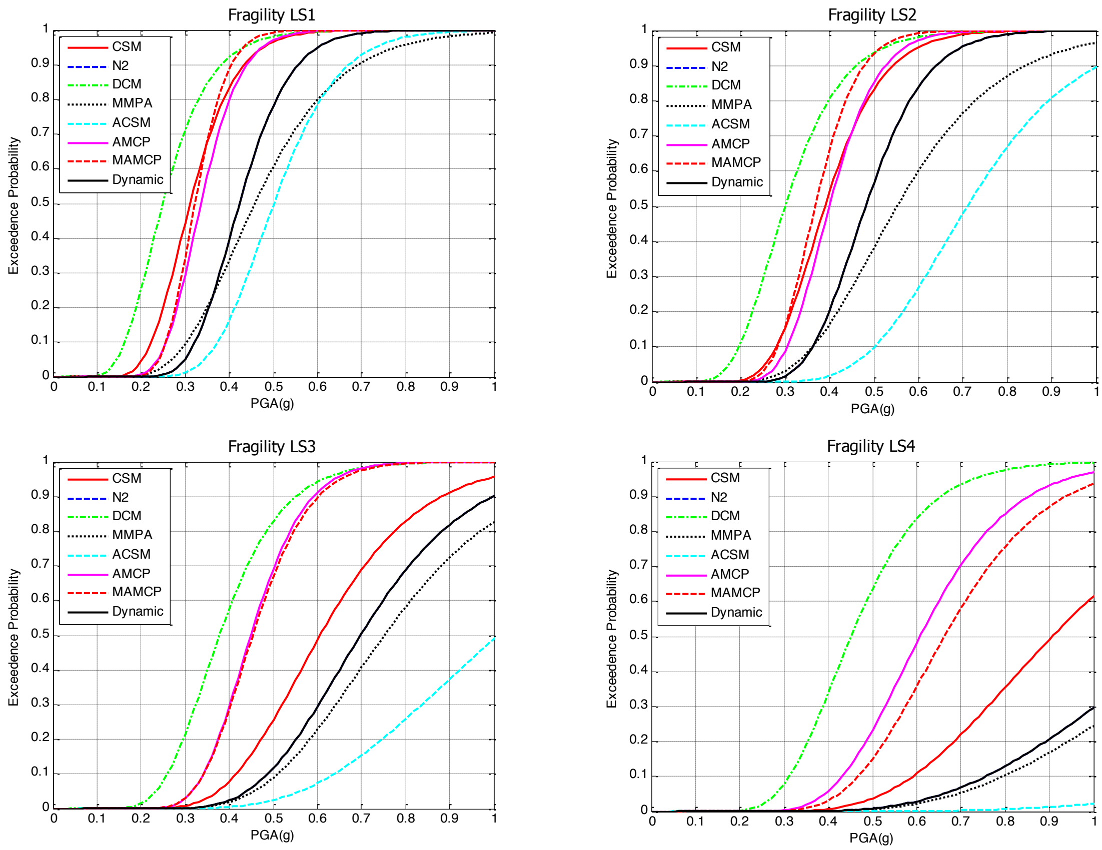

Figure 5. General results of each method for each Limit states

The dispersion among the estimates of the seven tested NSPs clearly increases with the intensity level (Limit State). Among the selected nonlinear static procedures, N2 provides the closest to dynamic analysis estimations, featuring also the lower computational onus, followed by MMPA. Generally, the methods requiring the definition of an equivalent viscous damping model, such as ACSM, AMCP and MAMCP, yield fragility curves farther from dynamic estimates. On the contrary, the methods making use of inelastic spectra, i.e. N2 and MMPA, did get closer to dynamic analysis. Adaptive pushover based methods, such as ACSM, did not necessarily. A possible reason could be related to the equivalent viscous damping model used in the procedure, which may be still leading to underestimation of the displacements when using ground motion records with low peak ground accelerations, such as the ones applied in this study.

Given that many records feature median to low peak ground acceleration it is believed that higher modes did not play a predominant role, since the structures did not go far beyond the elastic range thus AMCP and MAMCP provided similar results. Adding this to the higher computational demand required to carry out adaptive pushover analysis for each relevant mode, such procedures can become rather elaborated while still not providing more accurate estimates of fragility curves. The authors would thus not recommended AMCP or MAMCP for the prediction of seismic performance of bridges, by means of fragility curves. CSM and DCM did not succeed as well to yield accurate estimates, when compared to nonlinear dynamic analysis, in tandem with what had been verified before, in terms of Bridge Index results.

CLOSING REMARKS

A number of past studies have addressed the ability of nonlinear static procedures, individually or comparatively to alternative counterparts, to assess the structural performance of single bridges. Much less attention has been paid, on the other hand, to such evaluation within a given region context, comprising a population of bridges, assessing the performance of the different approaches in terms of fragility functions. In this study, seven different nonlinear static procedures were applied to investigate the most suitable analytical approach(es) for deriving fragility functions of bridges for use in loss assessment studies. In order to fulfil such goal, a sufficiently large number (200) of 3D bridge models, using a representative bridge configuration of three piers and four spans of variable height and length, were randomly generated using a fibre model based structural analysis software (OpenSees). The generation of different bridges was based on the variation of a number of parameters, which included concrete strength, steel strength, pier heights, column diameter and bay lengths. For all those considered parameters a statistical distribution was defined. Seven nonlinear static procedures, which include Capacity Spectrum Method (CSM), Displacement Coefficient Method (DCM), N2, Modified Modal Pushover Analysis (MMPA), Adaptive Capacity Spectrum Method (ACSM), Adaptive Modal Combination Procedure (AMCP) and Modified Adaptive Modal Combination Procedure (MAMCP), were employed for estimating response displacement for a number of ground motion records. Nonlinear dynamic analyses were carried out and used to measure the accuracy of the nonlinear static procedures, using a set of 50 non-scaled real ground motion records, selected from PEER database, which presented similar fault mechanisms and potential seismic intensity to the Italy territory characteristics. Damage states were defined for different levels in terms of displacement ductility and the corresponding probability of exceedance was determined for each bridge. Lognormal distribution functions were used to characterize the fragility functions for each method.

The comparison of the response estimates derived from NSPs and nonlinear dynamic analysis was carried out in terms of Bridge Index, a ratio that directly divides the response displacement from each NSP by the maximum response displacement from nonlinear dynamic analysis, and in terms of corresponding fragility functions, using different NSP estimates. The main conclusions from the results of this study can be summarized in the following points:

- Generally, Nonlinear Static Procedures were able to provide reasonable predictions in terms of response displacements, when compared to the nonlinear dynamic estimates. Notable exceptions include the Capacity Spectrum Method (CSM) and Displacement Coefficient Method (DCM), which yielded poor estimates, mostly due to the fact that they constitute pioneering approaches (CSM) or feature a strong empirical basis (DCM) which can be hard to apply in bridge analysis. The Adaptive Modal Combination Procedure (AMCP) has considerably overestimated the dynamic results, for high ground motion intensity levels, whereas for low intensity levels it provided similar results to Modified Adaptive Modal Combination Procedure (MAMCP). The Adaptive Capacity Spectrum Method (ACSM) provided fairly good estimates for large peak ground acceleration records, while, for low intensity ground motion records, it tends to underestimate the response displacement. N2, followed by Modified Modal Pushover Analysis (MMPA), provided quite accurate estimates, with respect to nonlinear dynamic results, for both high and low intensity levels;
- Globally, nonlinear static procedures provided better estimates with increasing ground motion intensity. The improved methods, which make use of adaptive pushover analysis and take higher modes contribution into account did not perform significantly better than the traditional ones. Indeed, if a single procedure should be selected, by taking computational onus and time demand in consideration, N2 would rank first among the seven NSPs;
- When comparing the results of fragility curves derived from different nonlinear static approaches, one could confirm the conclusions from the individual NSP validation, i.e., N2 provides the closest results to the nonlinear dynamic ones, followed by MMPA. CSM and DCM, initially proposed for building structures, failed to provide good estimates of fragility functions of bridges. As the set of ground motion records considered in fragility analysis did not send the structures highly into the nonlinear range, AMCP and MAMCP yielded similar results due to the elastic behaviour of higher modes. ACSM, making use of equivalent viscous damping, turned out to underestimate the results when applied with low intensity ground motion records. Moreover, there is still significant dispersion among the different methodologies, which, apart from the conceptual differences, might be related to the definition of limit states or the lognormal distribution chosen.

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