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USING DIFFERENT UNCERTAINTY MODELS FOR SEISMIC ASSESSMENT OF RC BRIDGES

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Abstract. *The seismic assessment of structures depends on a large number of aleatory and epistemic uncertainties, which are majorly associated to the estimation of the structural demand and capacity, both usually featuring considerable dispersion levels, particularly when reinforced concrete structures are being assessed. When focusing on bridges, additional complexity may be introduced by the irregular behaviour in the transverse direction. Several procedures may be used for the assessment of the seismic safety of bridges, deterministic or probabilistic, and all rely on an accurate prediction of the demand, obtained via linear or nonlinear static or dynamic analysis. This work employs both static and dynamic analysis methods for demand estimation within a relatively straightforward framework to compute the failure probability of existing bridges. Different variables typically considered in a seismic assessment procedure (geometry, material properties, earthquake records, intensity level) are statistically characterized, catering for a global simulation process, where each iteration step is associated to an independent structural nonlinear static or dynamic analysis. Failure probability is then obtained through different uncertainty models, corresponding to the convolution of capacity and demand distributions or the probabilistic analysis of a safety indicator, defined as the difference between capacity and demand at each random simulation realisation. A case study of seven bridge configurations, with different (ir)regularity levels, is considered together with a relatively large set of real earthquake records. The simulation process is carried out using Latin Hypercube sampling, expected to considerably reduce the number of realizations with no reliability loss. Conclusions have allowed the identification of vulnerable configurations and shown the differences in considering different uncertainty models.*

1 INTRODUCTION

A typical seismic safety assessment procedure involves a set of relevant elements that need to be properly defined, until the seismic safety itself is assessed. Such ultimate step may, similarly to all the other safety problem components, feature different approaching scenarios, even though it will essentially consist in the comparison of the demand, estimated based on a properly selected seismic ground motion input, with the capacity of the structural elements to accommodate it, characterised according to the geometry, material properties, nonlinear behaviour models, among others. Even though the deterministic approaches are most likely the ones that practitioners are most familiarised with, given that it certainly goes along with the traditional design practice or structural safety verifications, the research trend of the past decades is mostly addressing the employment of probabilistic approaches, which tend to gain weight, as rationally more consistent, e.g. [1-3]. However, the employment of probabilistic procedures for seismic assessment by practitioners is far from being straightforward, which gives room to the proposal of relatively simple methodologies that do not require deep mathematical formulations but still provide accurate results. Even if simple, any probabilistic procedure requires a substantial amount of statistical information, in comparison with the deterministic ones, to characterise the uncertainty in the structural safety problem. Such information includes the probabilistic characterisation of seismic hazard, as well as the capacity and structural demand.

For what concerns the seismic demand prediction, even if obviously depending on the structural characteristics and on the seismic hazard itself, the main differences tend to be associated to the nature of the employed analysis approach, either static or dynamic, each comprising different concepts and procedures. It is commonly recognised that nonlinear dynamic analysis (NDA), which reproduces directly the real phenomenon, will produce more accurate results. Nevertheless, several drawbacks may be pointed out when considering dynamic analysis, particularly related to the employment of ground motion records, to the structural modelling (including the reproduction of damping phenomena, the post-elastic behaviour and corresponding energy dissipation, during loading and unloading periods), amongst others. In addition, attention must also be paid to computational onus, which is well known to be significant in dynamic analysis. The commonly employed alternative to NDA is nonlinear static analysis, based on pushover analysis, which typically has the goal of somewhat enveloping all, or at least a considerable part, of the possible dynamic analysis results at each intensity level. This kind of outcome is accomplished through the application of an increasing lateral force or displacement vector to the structure. Even if inherently limited with respect to its dynamic counterpart, nonlinear static analysis constitutes nonetheless a very valuable tool within the framework of performance-based seismic assessment of structures, since it readily provides important insight into the main characteristics of the response of the latter when subjected to horizontal excitation. Consequently, in the last decade or so considerable effort has been placed in the development of credible, yet relatively simple, pushover-based methodologies, an effort that lead to the introduction of the so-called Nonlinear Static Procedures (NSP) in a number of guideline documents, such as ATC-40 [4, 5] and FEMA-273 [6], or design codes, such as Eurocode 8 [7, 8]. The main drawback of the pushover-based approaches is noticeably the level of simplification, given that one assumes that the structural behaviour obtained from horizontal pseudo-static loading is somehow equivalent to its dynamic counterpart. Although these have focused mainly on the application to seismic assessment of buildings, rather than bridges, recent studies, listed in the subsequent Section, have been made to explicitly verify the application of NSPs to individual and populations of bridge structures. However, these studies have been directed mainly at the direct validation of the nonlinear static procedures in predicting the seismic demand and hence there is still a need for assessing the performance of such procedures within the scope of probabilistic safety assessment.

In light of the above, this paper presents two different methodologies for the seismic safety assessment of structures, applied to bridges, which foresee the computation of the failure probability, through a relatively straightforward process, which can be used in current practice for seismic design or assessment purposes, without significant onus. In order to accomplish so, as well as to duly incorporate the uncertainty associated to all the considered variables of the safety problem, both capacity and demand are statistically characterised by means of assumed and/or adjusted distributions. Such statistical definition is carried out by obtaining, when necessary, random samples making use of the Latin Hypercube sampling (LHS) technique. Roughly, the safety assessment of a single structural system could be carried out through the statistical characterisation of a safety indictor, given by multiple, aleatory deterministic differences between the capacity and the demand, or a failure probability obtained from the independent statistical characterisation of the variables that are part of the process. Therefore, essentially, the difference between the two proposed methodologies lies on the way of dealing with the uncertainty of the different variables, which yield distinct failure probabilities. The distinction is established between accounting for uncertainty locally or globally. Both approaches are tested for a case study of seven bridge configurations, featuring different deck lengths and regularity levels. A companion paper addresses the relative accuracy of different nonlinear analyses (static and dynamic) in estimating the seismic demand within probabilistic framework, such as the ones herein proposed. This study also addresses the performance of a commonly employed nonlinear static procedure, within such probabilistic framework, with respect to the nonlinear dynamic counterpart, including the use of different versions of the static analysis, corresponding to different types of pushover load distributions. The parametric comparison of the static and dynamic approaches is then carried out.

2 SAFETY ASSESSMENT

The safety assessment herein carried out for RC bridges consists essentially of the probabilistic *comparison* between the demand, properly characterized through the probability density function of the structural effects, with the structural capacity, statistically defined in terms of parameters such as the geometry of the sections, properties and models of nonlinear behaviour of materials, among others. The statistical models of demand and capacity are then combined to obtain the failure probability according to two different models – local and global uncertainty – described ahead.

2.1 Nonlinear Static Demand

Nonlinear Static Procedures (NSP) are nowadays extensively spread within the earthquake engineering community to rapidly estimate the nonlinear demand, mainly due to its simplicity and potentially easy application when assessing a large number of structures. Although these methods were initially thought having the application to buildings in mind, some very recent endeavours have dealt with the application to bridges of this kind of procedures (e.g. [9-13]). These methods are essentially based on the computation of the pushover curve, a representation of the nonlinear force-deformation behaviour of the structure and with the use of appropriate transformation relationships, the capacity curve of the single-degree-of-freedom (SDOF) system equivalent to the original multi-degree-of-freedom (MDOF) structure may be obtained. At this point the main differences between the proposed methods arise, such as the contemplation or not and choice for a reference node, the way of reducing the demand spectrum so as to account for the hysteretic energy dissipation, the consideration of higher modes of vibration, on the way to determine the displacement demand for a given ground motion.

There are many available procedures to carry out nonlinear static analysis, ranging from truly simplified methods, based on coefficients, to others, more complex, including nonlinear and modal effects. A first group of proposed NSPs corresponds to the earliest attempts to propose a static method that would quickly lead to accurate results and includes the pioneering Capacity Spectrum Method (CSM), introduced by Freeman [14] and implemented in ATC-40 guidelines, and N2 method [15] included in the recommended simplified procedures of the European Design Code [8]. Recently, an improved version of the CSM has been presented in FEMA-440 guidelines [5], mainly consisting of the update of the prescribed empirical relations to determine both the equivalent viscous damping and spectral reduction factor. Later, the Displacement Coefficient Method [6] has been proposed within the NEHRP Guidelines for the Seismic Rehabilitation of Buildings. An additional group of procedures includes more recent approaches to nonlinear static analysis, such as the Modal Pushover Analysis (MPA) [16, 17], the Adaptive Modal combination Procedure (AMCP), by Kalkan and Kunnath [18] and later adapted to bridges by Shakeri et al. [19], or the Adaptive Capacity Spectrum Method (ACSM), developed by Casarotti and Pinho [20] and optimized by Monteiro et al. [21] also for building frames. All of them come up with improvements, mainly the inclusion of higher modes contribution or, as in the ACSM, an alternative way of addressing the reference node issue.

Recent studies [11, 22] have addressed the validation of different NSPs in estimating the seismic response of single or populations of bridges and the conclusions tend to point out to N2 and ACSM or modified MPA as the most accurate nonlinear static procedures. Moreover, the increased accurateness when using improved methods (ACSM, MPA) did not seem particularly advantageous when compared to the employment of N2, numerically less onerous. Similar findings were observed for RC building frames in a study by Pinho et al. [23]. Based on such, the N2 method was selected herein to test the use of nonlinear static analysis in the probabilistic seismic safety assessment of bridges. In addition, adaptive pushover analysis, which does not require the definition of a reference node, was included in the comparison.

2.2 N2 Method

The N2 method was proposed by Fajfar and Fischinger [15], and features the use of pushover analysis of a MDOF model combined with the inelastic response spectrum analysis of its equivalent SDOF system. Its implementation in the bridges chapter of Eurocode 8 foresees the computation of at least two pushover curves; one derived with a loading vector shape factor that is uniform along the deck and another where the first mode proportional shape is used instead. Two different loading possibilities have been considered in the application of the N2 method: uniform load distribution and first mode load distribution. Because of the method’s reference node dependency, each of these modalities was carried out considering the reference node as both the deck’s centre of mass or the maximum modal displacement.

2.3 Latin Hypercube sampling

Safety assessment based on reliability calculations will involve repetitive simulation of several variables representing structural modelling parameters, which may become, for complex finite element models, quite time-consuming and demanding in terms of computational onus. In order to overcome such drawback, advanced simulation techniques have been proposed on the basis of relatively simple integrated modifications in the existing general procedures. The Latin Hypercube sampling (LHS) simulation scheme [24, 25] is one of such examples and can be seen as a particular case of the standard Monte Carlo (SMC) numerical simulation, requiring a smaller number of runs, tens to hundreds, to achieve an accurate random distribution. Such key features are accomplished by means of the stratification of the theoretical probability distribution function of input random variables. The method is not restricted to the estimation of statistical parameters of structural response but offers a quite broad usage domain that goes from sensitivity analysis to Bayesian updating, among other possibilities. The use of LHS as a valid sampling technique and is becoming widely employed in structural analysis of bridges [26-30].

2.4 Failure probability

The probabilistic methodologies considered herein are described in detail in a study by Monteiro et al. [31, 32] whereas a brief summary is provided next.

2.4.1. Local uncertainty model

This is considered the traditional approach, originally developed and applied in the work by Costa [33], which is focused on the definition of the probability density function of the seismic demand, which will be convolved with the capacity one. In order to do so, there is the need for a relationship between an intensity measure (herein considered as the peak ground acceleration) and a proper engineering demand parameter, measured from nonlinear static or dynamic analysis of the structure for that specific ground motion level. That relation is herein named vulnerability function and is essentially a mapping function, establishing the correlation between the seismic intensity, its probability and the actual structural demand, to be "compared" with the capacity to obtain the final collapse probability. For a predefined number of intensity levels, corresponding to different return periods and probabilities of occurrence, different real ground motion records or corresponding response spectra, scaled to match the corresponding PGA, are considered. Nonlinear static or dynamic analysis is carried out using each of the scaled accelerograms or spectra, yielding a sample of the seismic demand that accounts for record-to-record variability. For each ground motion intensity, and corresponding PGA, a statistical value, defined by the mean (or median) response measure, is computed and a polynomial function is adjusted to those points, to define the aforementioned vulnerability function, curve 3 in Figure 1 (*left*). Once the vulnerability function is defined, the statistical distribution of the seismic demand can be easily obtained through a numerical procedure based on equal areas under the probability density function, i.e. probability of occurrence, along with the changing of the variable from intensity measure to the selected engineering demand parameter. The entire procedure corresponding to the traditional approach is schematically depicted in Figure 1 (*left*). The seismic intensity is characterised by its probability density function, the curve 1. Analogously, for every intensity measure level for which curve 1 is defined, the seismic demand is obtained, in terms of a chosen parameter (for instance, ductility). The so-called vulnerability function, curve 3, is therefore defined, representing the structural demand with increasing intensity. The latter function is fundamental to 'intersect' with the seismic action probability density function and define the statistical distribution of the seismic demand, curve 4. In other words, by putting together the probability density associated to an intensity measure level with the corresponding structural demand, one can obtain the probability density of such demand to be verified.

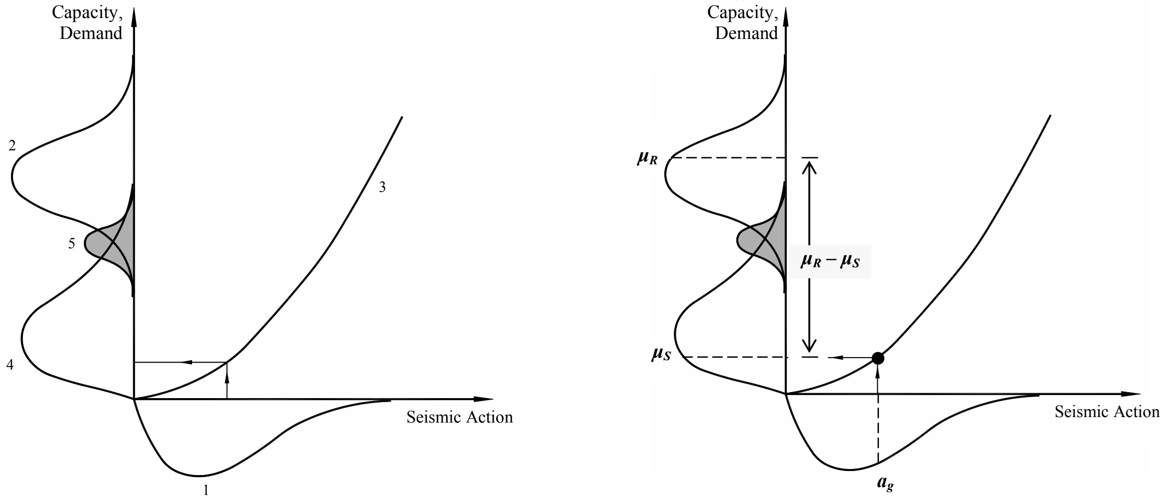

Figure 1: Failure probability computation with: (*left*) local uncertainty, (*right*) global uncertainty.

At the same time, the distribution of the capacity, curve 2, in terms of the engineering demand parameter used to define curves 3 and 4 (curvature ductility) needs to be defined. This step is typically carried out using random simulation procedures (herein Latin Hypercube sampling [24, 25, 34, 35] was adopted), based on the statistical distributions of the variables with a relevant role, usually cross section material properties. Finally, the convolution of capacity with the demand yields the intended failure probability.

2.4.2. Global uncertainty model

An alternative approach to the local uncertainty based failure probability computation consists in considering the uncertainty associated to the different variables in a global manner, i.e. simultaneously rather than independently, within a global procedure carried out fully at each iteration step. It is recalled that in the procedure previously described the uncertainty has been taken into account independently, i.e. each relevant variable of the problem identified in Figure 1 (*left*) was defined by a statistical distribution, based on independent sets of random sampling. On the other hand, the approach herein presented uses the statistical distributions of the different variables differently, as illustrated in Figure 1 (*right*). First, the distributions are defined, characterising each input variable (e.g. material properties), the seismic hazard and the capacity. Once that initial step is completed, the assessment procedure is carried out completely, but individually, making use of those distributions. Using a global random simulation technique, the Latin Hypercube, each variable with known uncertainty, seismic intensity level, type of record and material properties, is randomly simulated; a nonlinear dynamic analysis is carried out on the bridge modelled with such parameters; the structural demand and capacity are determined; and the difference between those two figures, usually referring to ductility or displacement, is calculated. A statistical distribution is then adjusted to the variable given by such difference, which will enable the computation of the failure probability as the ordinate of the corresponding cumulative density function for the null abscissa. The difference (μ_R - μ_S), eventually non-positive, will represent the non-failure interval. The concept behind this procedure is that the entire process, up to the definition of such interval, can be repeated N times in order to fit a statistical distribution to that variable and the knowledge of such distribution will enable the calculation of the failure probability as the area under the corresponding probability density function, $f_{R\text{-}S}$, for (μ_R - μ_S) < 0. The global uncertainty model for the calculation of the failure probability has not been employed with nonlinear static analysis, for the sake of computational onus. However, given that it has yielded conservative results with respect to the local uncertainty model, it has been include in this comparison study for completeness.

3 CASE STUDY

The comparison of the use of different approaches for seismic demand estimate, within different safety assessment uncertainty models has been carried out using the a set of seven bridge configurations, corresponding to two bridge lengths (viaducts with four and eight 50 m spans), with regular, irregular and semi-regular layout of the piers' height and continuous deck-abutment connections supported on piles, with bilinear behaviour. The bridges are depicted, in Figure 2, where the label numbers 1, 2 and 3 stand for pier heights of 7, 14 and 21 metres, respectively.

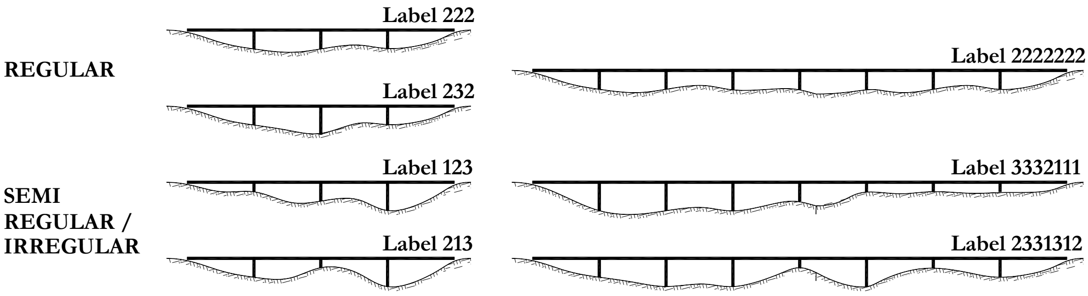

Figure 2: Case study bridge configurations.

The employed set of seismic excitations is defined by an ensemble of ten records selected from a suite of historical earthquakes [36] scaled to match the 10% exceeding probability in 50 years (475 years return period) uniform hazard spectrum for Los Angeles, which corresponds, in the current endeavour, to the intensity level 1.0. Figure 3 illustrates the pseudo-acceleration and displacement spectra for the selected earthquake records, with damping ratio of 5%, together with the median spectrum. The ground motions were obtained from California earthquakes with a magnitude range of 6-7.3 recorded on firm ground at distances of 13-30 km; their significant duration [37] ranges from 5 to 25 seconds, whilst the PGA (for intensity level 1.0) varies from 0.23 to 0.99g.

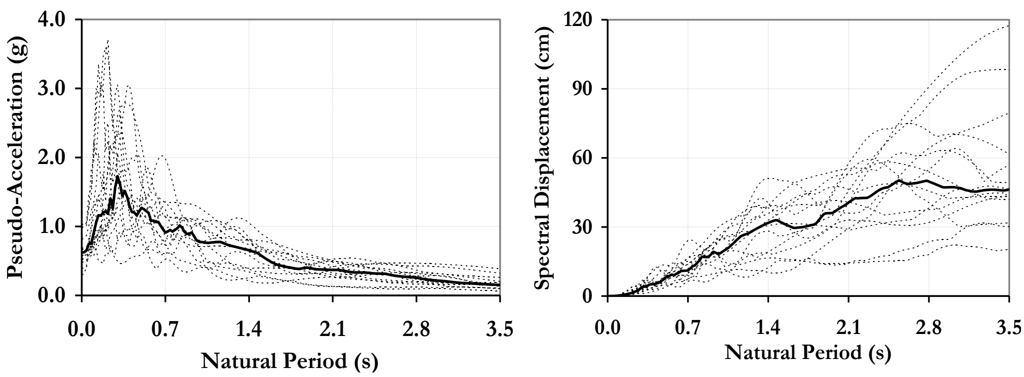

Figure 3: Elastic pseudo-acceleration and displacement spectra.

4 SAFETY ASSESSMENT – COMPARATIVE STUDY

Two different procedures for the failure probability computation have thus been considered (traditional numerical safety assessment with local uncertainty and numerical safety assessment based on global uncertainty simulation with Latin Hypercube sampling) together with nonlinear static or dynamic analysis for the estimate of the seismic demand. The suitability of the nonlinear static predictions has been scrutinised by direct comparison with the failure probability obtained with nonlinear dynamic analysis (NDA).

4.1 Nonlinear dynamic based failure probability – local uncertainty model

The collapse probability for the several bridge configurations, obtained with nonlinear dynamic analysis and applying the local uncertainty model is summarised in Figure 4. For the sake of simplicity and interpretation of the results in the same plot, only the failure probability of the critical pier of each bridge is presented. The results have been sorted by increasing failure probability with drift limitation. The capacity has been defined as drift-limited or not, which can result in important changes on the available ductility and consequent safety evaluation. When drift limitation was considered, a threshold of 5% drift was preliminarily assumed for the piers, after which it is considered, particularly for the 21m piers, that the top displacement is too high to ensure the stability of the deck.

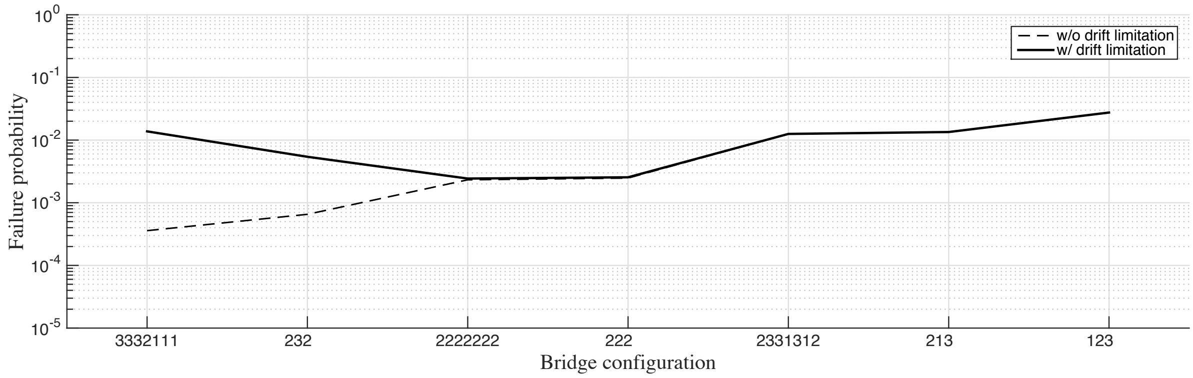

Figure 4: Failure probability per bridge configuration – local uncertainty approach.

Globally, there is agreement between considering the top deformation of the piers limitation or not, given that the trend of results is similar. The exception occurs when a type 3 pier becomes the most vulnerable, such as in configurations 232 and 3332111. Moreover, the distinction between regular/semi-regular and irregular configurations is immediate from Figure 4, the latter being more vulnerable of about one order of magnitude, all in the right side of the plot. The record-to-record variability of the seismic input plays a very important part and the failure probability, traditionally computed using mean ductility demand, will indeed vary considerably. The magnitude of the effect caused by such variability will depend on the bridge configuration or the pier characteristics.

4.2 Nonlinear dynamic based failure probability – global uncertainty model

The collapse probability obtained with nonlinear dynamic analysis and applying the global uncertainty model is summarised in Figure 5. Similarly to the safety assessment procedure using a local uncertainty model, which considers the variability of each component separately, there is a fair match of the collapse probability across the different configurations, whether drift limitation takes place or not.

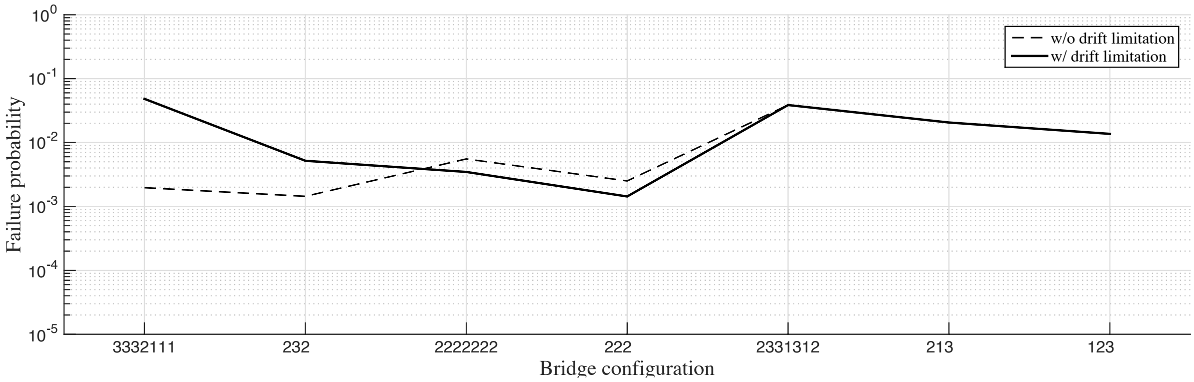

Figure 5: Failure probability according to bridge configuration – global uncertainty approach.

Major differences occur for the configurations where the critical pier is a type 3 one, 21 meters high, the one that is most affected by the capacity reduction. Again the distinction between regular and irregular bridges is very noticeable (the former are safer) together with the trend for the short configurations to be also less vulnerable.

4.3 Nonlinear static based failure probability

The failure probability of the different bridge piers, obtained with the employment of static and dynamic vulnerability functions (local uncertainty) are compared and illustrated in Figure 6. Again, for the sake of simplicity, the probability of failure is assumed as the highest among the different piers. Conventional and adaptive (*adapt*) pushover analysis have been used. According to the recommendations on the application of the N2 method, two loading distributions, uniform and 1st mode proportional (*mod* and *uni*), have also been tested. Additionally, two reference nodes have been considered: centre of mass of the deck and maximum modal displacement (*centr* and *max*), leading to a total of four versions using conventional pushover and one corresponding to the adaptive pushover, which skips the need for a reference node or load pattern.

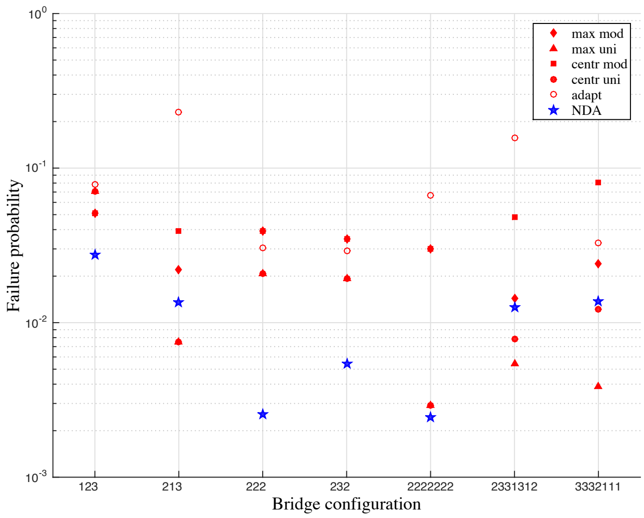

Figure 6: Critical pier failure probability, per bridge configuration.

The observation of the plot indicates that regular configurations feature lower collapse probabilities, particularly through nonlinear dynamic analysis, even though associated to larger variability in terms of different pushover approaches. Regarding the type of pushover to use, if a simple static analysis is intended, modal load pattern does not introduce significant advantage when compared to the uniform one, which, in addition, is easier to apply. Adaptive pushover, generally overpredicting, becomes relevant for highly irregular configurations, even if not necessarily for the critical piers. No important differences are found concerning the length of the configuration: short bridges are slightly less safe than the long ones together with a little less dispersion within different versions.

4.4 Overall comparison of approaches

Finally, Figure 7 illustrates the comparison of the static failure probabilities with the dynamic ones, obtained with local and global uncertainty models. For the sake of simplicity, and based on the conclusions drawn from Figure 6, only static failure probabilities based on conventional pushover, with uniform load pattern and central reference node, and adaptive pushover versions are presented.

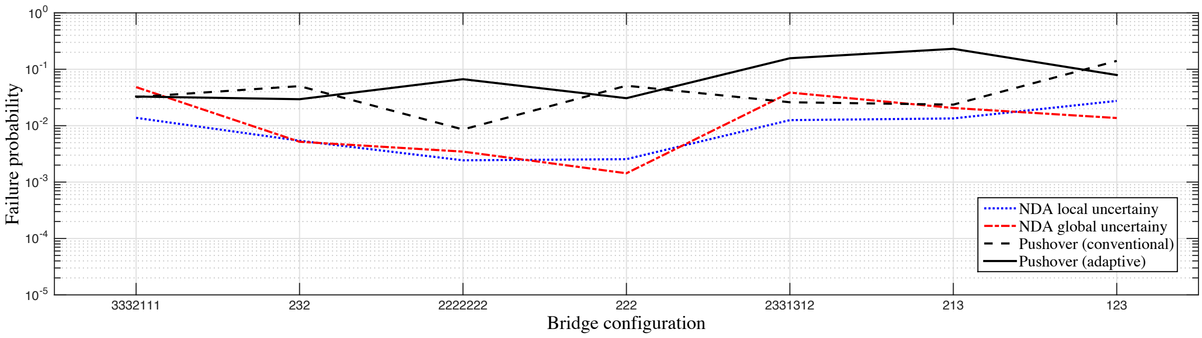

Figure 7: Failure probability, according to bridge configuration, for static and dynamic analysis based versions.

The general observation is that pushover-based estimation of seismic demand leads, in most of the cases, to higher failure probability, especially when using adaptive analysis, which is frequently associated to a failure probability of one order of magnitude above. When compared to the global uncertainty dynamic failure probabilities, which were observed to play on the safe side, the static based approaches still play an enveloping role. These results reinforce the important supporting role played by nonlinear static analysis in the use of probabilistic methods, when compared to dynamic counterpart.

5 CONCLUSIONS

The analysis of different approaches to carry out seismic safety assessment of reinforce concrete bridges constituted the main objective of this manuscript. Alternative procedures have been proposed, corresponding to uncertainty models that account for the variability of the different variables: material properties, type of record and intensity level. An innovative statistical sampling method has been used to characterise capacity as well as to compute the failure probability within a global simulation procedure, using nonlinear dynamic analysis as seismic demand prediction technique.

The parametric study concerning the aforementioned components of the safety assessment procedure, the observation of the performance of the alternative methodologies and the final comparison of results validated both of the uncertainty models in a range of seven bridges and ten real accelerograms, with good matching throughout the different configurations. The employed simulation method, Latin Hypercube, proved itself equally effective in simplifying the procedures by requiring a smaller number of realisations without accuracy loss.

In terms of nonlinear demand prediction, the observation of the results has indicated a tendency for the static analysis approaches to be conservative, even when applied with conventional uniform loading shape. The adaptive pushover is more suitable for the prediction of the behaviour of irregular configurations whereas conventional pushover, with uniform load pattern, is enough to get rather acceptable estimates for regular configurations. Within methodologies such as the probabilistic ones, in which the seismic demand needs to be estimated through a large number of nonlinear analyses, this feature constitutes an important asset, at least for a preliminary analysis.

The final results also confirmed that the seismic hazard and record-to-record variability are critical in for the evaluation of the structural vulnerability, as their consideration as a probabilistic variable within the global uncertainty model led to higher probabilities of failure.

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