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ACCURACY OF SIMPLIFIED PROCEDURES FOR THE FRAGILITY ASSESSMENT OF BRIDGES

Pierre GEHL[1] and Dina D'AYALA[2]

Abstract: This paper explores the accuracy of simplified methods based on direct displacement based assessment (DDBA) for the derivation of fragility curves for typical highway bridges. Two variants of the DDBA procedure are implemented: Effective Modal Analysis (*EMA)*, setting a given target deformation level and computing the corresponding seismic level, and Direct Effective Modal Analysis *(dirEMA)*, computing directly the structural response from an input ground motion. These are applied to 16 bridge models, differing for the level of continuity of their deck and the type of pier-to-deck connections. The results obtained with this two simplified analysis approaches are used to derive component- and system-level fragility curves, which are then compared to fragility functions obtained from nonlinear dynamic analyses. While significant discrepancies are observed for some of the component fragility curves, it is found that good agreement can still be found at the system level, possibly due to the existence of minor failure modes that have a negligible impact on the global damage probability. Therefore these simplified methods could be helpful for the efficient derivation of fragility curves for a larger number of bridge typologies.

Introduction

The vulnerability of critical infrastructure to seismic risk has been a growing concern for stakeholders, especially due to the interconnected nature of these systems and their potential for the propagation of cascading events. More precisely, in the case of transportation systems such as road or railway systems, it has been observed that bridges play a critical role, both due to their relative vulnerability to seismic loading and the consequences they may induce in the case of a failure. In this context, significant effort has been dedicated to the seismic assessment of highway bridges: however, because of the numerous bridge typologies with varying number of spans that may compose a given highway network (e.g. including, viaducts, river crossings, road crossings, overpasses, etc.), an in-depth analysis of each bridge through resource-consuming time-history dynamic analyses may not constitute the most adequate approach.

Therefore there is a need for an assessment procedure that yields accurate estimates while being simple enough for a systematic application to a large portfolio of bridges. Parallel to the simplified nonlinear displacement-based static methods (e.g. ATC-40, FEMA-273 or N2 methods), Sadan et al. (2013) and Cardone (2014) have introduced a direct displacement-based assessment (DDBA) procedure, which is the follow-up of the displacement-based design approach by Priestley et al. (2007). This method is based on the evaluation of the secant modal properties of the bridge system (i.e. through the estimation of the secant stiffness) for a given target level of deformation of the bridge, referred to as Performance Displacement Profile (PDP). Cardone et al. (2011) have applied this approach to the derivation of fragility curves: however, only the mean fragility parameter α is quantified, while the standard deviation β is arbitrarily assumed, due to the use of a single design spectrum as the applied demand on the structure. Another concern lies within the accuracy of the DDBA to predict structural responses in the nonlinear range and for a variety of bridge systems, since comparisons have only been carried out for a limited number of bridge types (Sadan et al., 2013; Cardone, 2014).

As a result, the present study proposes to investigate the accuracy of the DDBA method for a set of bridge systems that represent distinct typologies, especially in terms of deck continuity and pier-to-deck connection. The DDBA is compared with the nonlinear results obtained from the OpenSees platform (McKenna et al., 2000): it is proposed to use the derived fragility curves

[1] Research Associate, EPICentre, University College London, London, p.gehl@ucl.ac.uk
[2] Professor, EPICentre, University College London, London, d.dayala@ucl.ac.uk

as a performance indicator of the DDBA, since this measure has direct consequences on the damage assessment of the infrastructure components in the case of a risk analysis. Moreover, a variant of the method by Sadan et al. (2013) is introduced, in order to enable the direct computation of the structural response based on an input ground-motion record.

The Direct Displacement-Based Assessment (DDBA) method

The DDBA is initiated by defining a target deformation level for a given bridge component (e.g. pier or bearing): an iteration procedure is then carried out to update the modal properties of the structural system corresponding to the target deformation, by using the secant stiffness formulation. Cardone et al. (2011) have shown that the obtained displacement pattern of the bridge could be used to derive a fragility curve, if the target deformation is considered as a damage threshold: the input design demand spectrum is scaled and over-damped in order to account for the energy dissipation by the hysteretic cycles in the non-linear range, finally providing the intensity measure that corresponds to the target deformation. However, the following points could still be improved within the DDBA procedure:

- The standard-deviation of the median value of the fragility curve (i.e. dispersion) is arbitrarily chosen because a normalized design spectrum is used as input, rather than a suite of natural records.
- In order to estimate the spectrum scaling factor that will reach the target deformation, an inverse capacity spectrum method is used, where the coordinates of the performance point are derived from an equivalent single-degree-of-freedom approximation.
- Finally, while the DDBA iteration process starts from a target deformation level in order to estimate the corresponding intensity measure, the possibility to directly compute the bridge response from an input demand spectrum should be investigated.

Therefore it is proposed to use a set of natural ground motion records as input to the DDBA procedure, instead of a design demand spectrum: this modification enables to obtain a distribution of performance points for different levels of seismic intensity, thus accounting for the record-to-record variability. This variant of the DDBA procedure is later referred to as *EMA* (Effective Modal Analysis), which main steps are defined in Figure 1 (left).

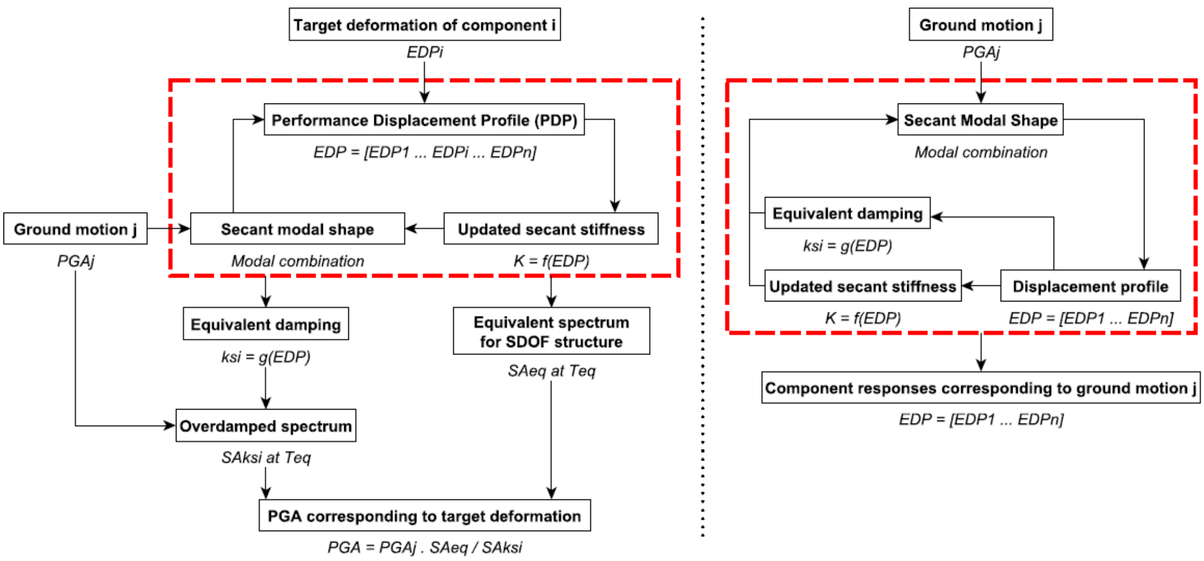

Figure 1. Main steps of the two simplified approaches (*EMA* on the left, *dirEMA* on the right).

It is observed that the main drawback of the *EMA* approach is the requirement to predefine a target deformation level (i.e. limit state) and to carry out the analysis for each component: for instance, in the case of a bridge system with *n* components and *m* damage states each, a total of *n* x *m EMA* analyses will have to be performed in order to obtain the fragility parameters for all components. Moreover, the input records have to be scaled in order to match the seismic level that corresponds to the target deformation: a lot of care is usually required in the scaling process, so that the selected ground motions keep consistent properties (e.g. frequency content) over a given range of seismic intensity. Finally, the *EMA* approach is based on the estimation of equivalent properties (e.g. spectral coordinates of the performance point) of the single degree-of-freedom (SDOF) structure: this approximation may also constitute a source of error.

Based on these observations, a more direct procedure is investigated (i.e. *dirEMA*, for direct Effective Modal Analysis, as show in Figure 1 right): as opposed to the *EMA* approach, unscaled ground motion records are used as inputs in the *dirEMA* procedure, while the structural response is directly quantified as a result. This is achieved by reorganizing the iteration cycle, which now includes the update of equivalent damping at each loop. Using this approach, all the component responses are obtained through only one run of the *dirEMA* algorithm and there is more flexibility in the choice of the input records, which can be either unscaled (e.g. 'cloud' analysis) or scaled (e.g. multi-stripe analysis or Incremental Dynamic Analysis).

The *EMA* and *dirEMA* methods have been implemented into an automated Matlab program (The MathWorks, 2013): only a limited amount of information is required to define the bridge system, i.e. mass matrix, connectivity between structural nodes, stiffness model for each component, set of acceleration time-history files, and the routine returns the structural response of the bridge components, if *dirEMA* is used, or the PGA level corresponding to the desired deformation level of a given component, if *EMA* is used.

Depending on the stiffness model that is used for each component (e.g. elastic-perfectly plastic, bilinear with strain hardening, trilinear…), a set of previously defined constitutive models are selected in order to update the secant stiffness and the equivalent damping ratio, based on the deformation or the ductility ratio of the component. For instance, secant stiffness is simply obtained by computing the ratio of the actual force that would be generated by the component in the nonlinear range (i.e. according to the stiffness model) over the actual deformation of the component. In the case of the damping ratio, relations providing equivalent damping as a function of ductility ratio can be found in Blandon and Priestley (2010).

Identification of the main bridge typologies

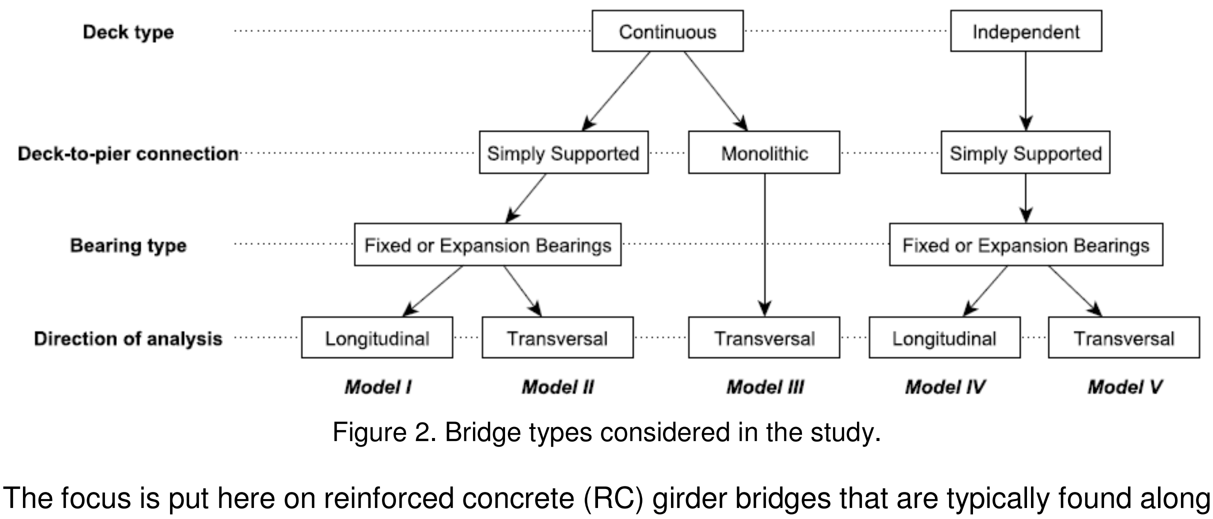

Figure 2. Bridge types considered in the study.

The focus is put here on reinforced concrete (RC) girder bridges that are typically found along road highways. A first distinction can be made regarding the deck, which can be either continuous or comprised of a set of independent spans. Then, the deck can be connected to the pier cap through various bearing devices (i.e. simply-supported deck) or it can be directly connected to the pier (i.e. monolithic connection), thus preventing relative movement for all degrees of freedom. Finally, depending on the bridge configuration, the large rigidity of the deck in the longitudinal direction and the possibility (or not) of relative displacement between the deck and the piers, seismic analysis of the bridge in the longitudinal direction may be

straightforward (i.e. only one degree-of-freedom), while the deformation of the deck in the transversal direction always allows relative displacements between the pier caps. As a result, five generic bridge layouts are proposed, as illustrated in Figure 2.
The mechanical properties and structural characteristics of the multi-span simply-supported concrete (MSSSC) girder bridge, as described by Nielson (2005), have been used to define the five generic models, with the same RC pier and deck properties: details of the structural properties are also provided in Gehl et al. (2014). Regarding the bearings, simplified stiffness models are proposed in order to facilitate the application of the *dirEMA* and *EMA* approaches: bilinear or trilinear models are used, in order to model the Coulomb friction law of the elastomeric bearings. In the case of fixed bearings, the stiffness model proposed by Cardone (2014) has been used, as shown in Figure 3.

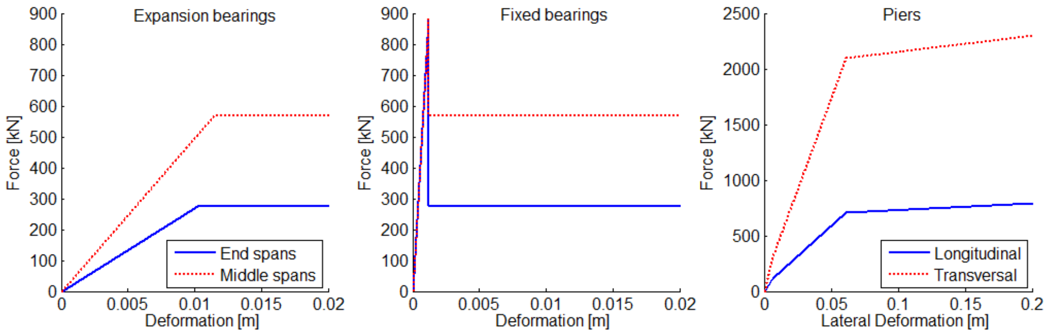

Figure 3. Stiffness models for expansion (left) and fixed (middle) bearings, and piers (right).

Variants of the five generic models are also developed, depending on the configuration of the expansion and fixed bearings (see Figure 4): this enables to sample a wide range of bridge layouts, from very rigid systems (i.e. only fixed bearings) to very flexible ones (i.e. only expansion bearings).

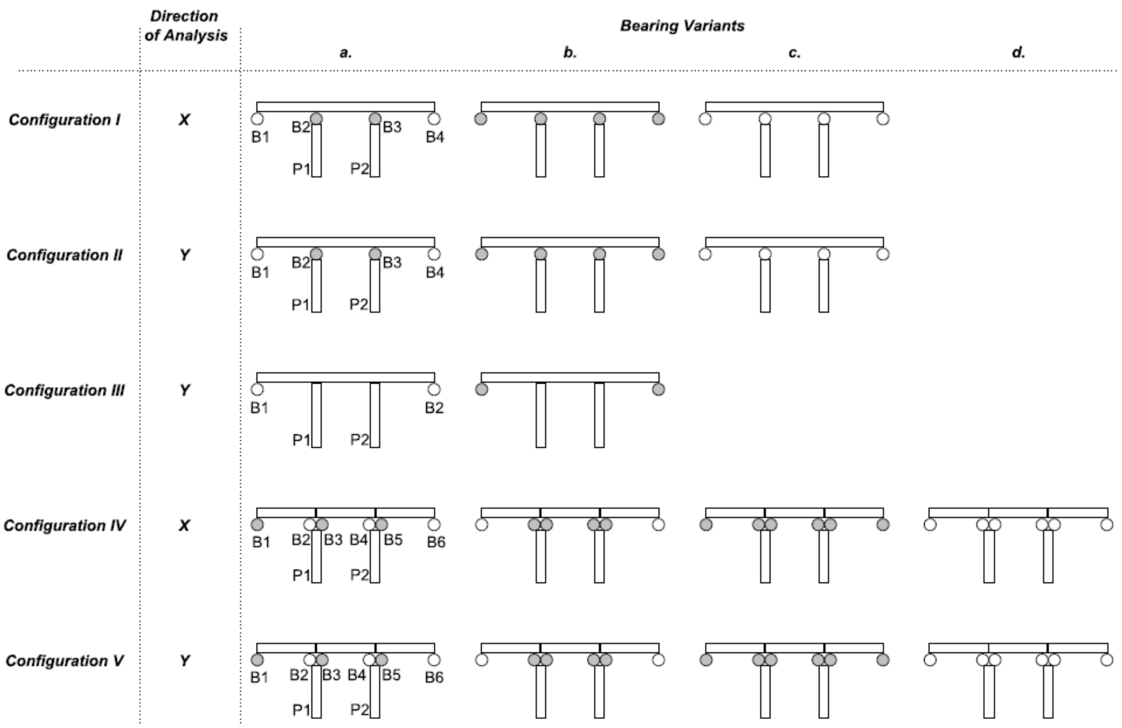

Figure 4. Layout of the bridge models considered in the study. White (respectively grey) circles represent expansion (resp. fixed) elastomeric bearings.

Finally, the bridges are modelled in OpenSees (McKenna et al., 2000) and in the *EMA* tool, following the reduction procedure described in Gehl et al. (2014): all components are modelled by zero-length connectors, whose behaviours are approximated by piece-wise linear curves that are based on their constitutive laws. Specific models have to be developed for each direction of analysis (i.e. longitudinal or transversal), due to the different stiffness of piers and decks that are observed in the two directions.

Derivation of component fragility curves

Each bridge model from Figure 4 is submitted to a set of 288 unscaled ground-motion time-histories, through OpenSees nonlinear dynamic analyses or *EMA* and *dirEMA* methods. The input records are obtained with a synthetic method (Pousse et al., 2008), for a given range of epicentral distance (R_{epi} between 10 and 100 km) and magnitude (M_w between 5.5 and 7.5). For each of the bridge components that are likely to sustain any damage (i.e. RC piers, fixed and expansion bearings), the seismic response is recorded and compared to the limit state thresholds proposed in Table 1: the deck is assumed to remain elastic for the applied level of seismic loading. Only slight and moderate damage states are considered in the present study, since most components have not reached further damage states under the selected range of ground motions (i.e. moderate seismicity): thus, due to the scarcity of data points for heavier damage states, stable estimates for the corresponding fragility parameters cannot be guaranteed.

Table 1. Median values for prescriptive limit states, as proposed by Nielson (2005)

Component	Slight	Moderate
Pier (longitudinal) – *Curvature ductility* / Lateral Deformation [mm]	*1.00* / 52	*1.58* / 70
Pier (transversal) - *Curvature ductility* / Lateral Deformation [mm]	*1.00* / 60	*1.58* / 74
Fixed elastomeric bearing – Deformation [mm]	30	100
Expansion elastomeric bearing – Deformation [mm]	30	100

Fragility curves are expressed as a cumulative lognormal distribution function with respect to PGA, while the distribution parameters (median α and standard deviation β) are derived through a Generalized Linear Model (GLM) regression, using a *probit* model as the link function. This approach considers the component responses (i.e. deformation) as binary variables (i.e. 1 if damage, 0 if not) and the fragility parameters are estimated so that they maximize the likelihood function. In the case of the *EMA* procedure, the results are given as a distribution of PGA for a given limit state threshold: therefore, the corresponding fragility curves are simply obtained by fitting a lognormal distribution function over the numerical distribution of PGA values.

The accuracy of the *EMA*- and *dirEMA*-derived component fragility curves has then to be quantified with respect to the nonlinear OpenSees results: while the direct comparison of both fragility parameters α and β does not necessarily provide a straightforward visualisation of the "error rate", it is proposed instead to use the Kolmogorov-Smirnov (K-S) distance, which measures the largest absolute difference between two distribution functions (see Figure 5). This metric has previously been used by Gehl et al. (2015) for the comparison of fragility curves: it has the ability to directly express the maximal error in terms of probability of damage, which is a quantity that can be easily interpreted in practice.

A quick analysis of Figure 5 reveals the following points:

- Globally the performance of both *dirEMA* and *EMA* methods is not excellent and many fragility curves result in a K-S distance that is superior to 0.5: in other words, for some PGA values, the difference between the approximate probability of damage and the OpenSees one is exceeding 0.5, which represents a non-negligible bias.
- While there is no overall significant difference between the performance of *dirEMA* and *EMA* methods the *dirEMA* procedure seems to be slightly more accurate. This observation confirms that *dirEMA* could be used instead of *EMA*, since the former method is less time consuming than the latter while allowing the use of unscaled records.

- For simpler bridge models with lower numbers of in-series components (i.e. bearings), such as models *I* to *III*, the simplified approaches show reasonable accuracy. On the contrary, bridge models *IV* and *V* with simply-supported independent deck spans present the highest prediction errors, because components located towards the middle of the bridge are less restrained and they are more influenced by the response of adjacent components.
- Bridge models with only expansion bearings appear to lead to more accurate results, due to their simpler stiffness model (i.e. bilinear curve), as opposed to fixed bearings.

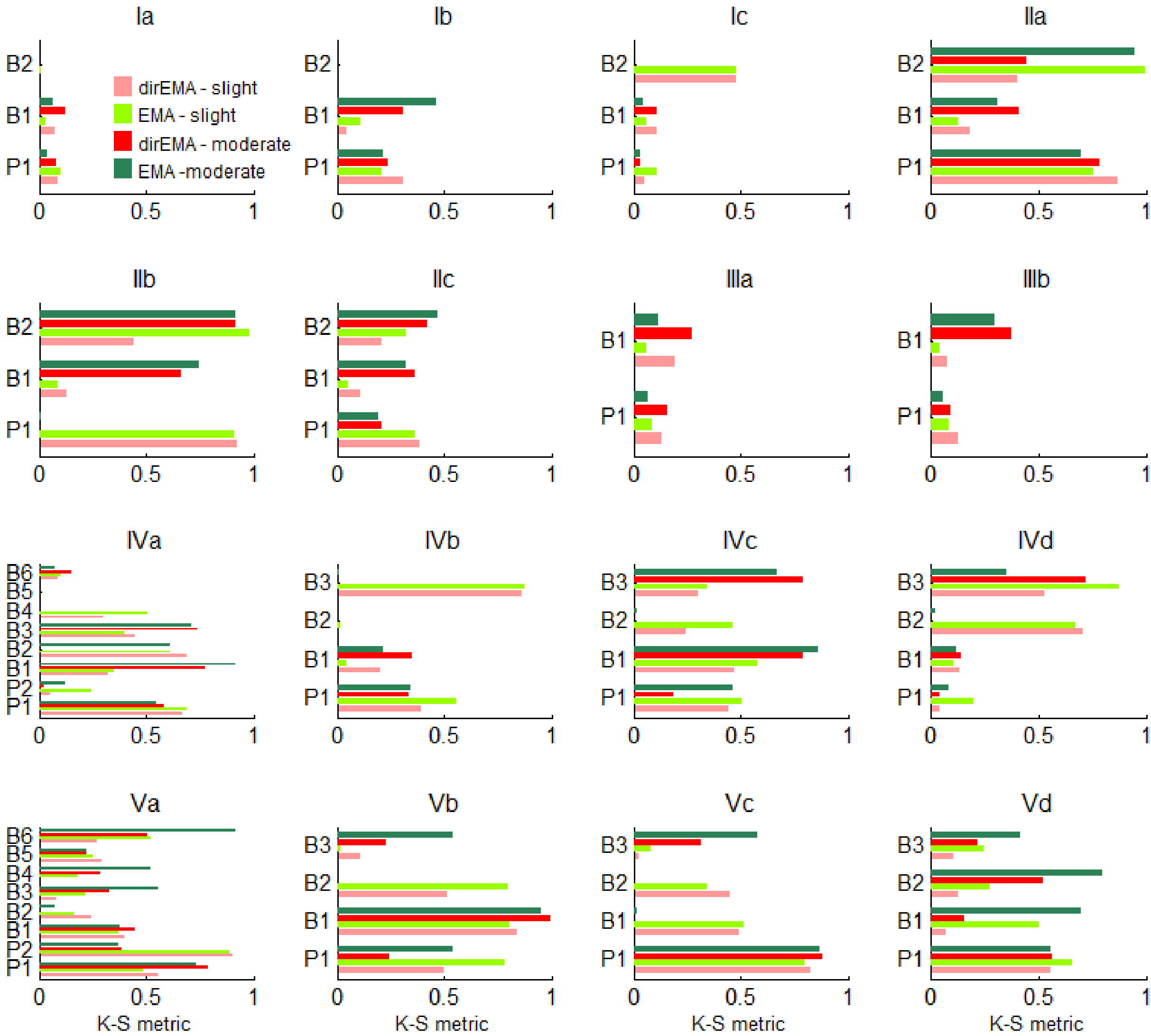

Figure 5. Kolmogorov-Smirnov distance between the OpenSees component fragility curves and the *dirEMA* (in red) and the *EMA* (in green) results, for the 16 bridge models. Except for models *IVa* and *Va*, all bridges are symmetric and only half of the components are represented.

Derivation of system fragility curves

Starting from the component fragility curves developed in the previous section, fragility curves at the system level are derived by using the system reliability method developed by Song and Kang (2009): it is assumed that the component damage states are consistent with each other, so that the system damage state (e.g. slight) occurs as soon as one of the components is in the same damage state (i.e. in-series system).

The statistical dependence between the component responses, which is due to their common dependence on the seismic parameter PGA, has to be taken into account when assembling the component fragility curves. This is achieved by introducing common source random variables that can be approximated by a class of Dunnett-Sobel variables (Dunnett and Sobel, 1955). Hence, the probability of the bridge system reaching or exceeding a given damage state DS can be expressed as:

$$P(DS|PGA) = 1 - \int_{-\infty}^{+\infty} \prod_{i=1}^{n} \left[1 - \phi\left(\frac{\log PGA - \log \alpha_i + \beta_i r_i x}{\beta_i \sqrt{1 - r_i^2}} \right) \right] \varphi(x) dx \qquad (1)$$

Where α_i and β_i are the fragility parameters for component i, φ is the standard normal probabilistic distribution function, Φ is the standard normal cumulative distribution function, and r_i is the approximated correlation factor for component i. Equation 1 can be numerically integrated and the system fragility curves for the 16 models are represented in Figure 6.

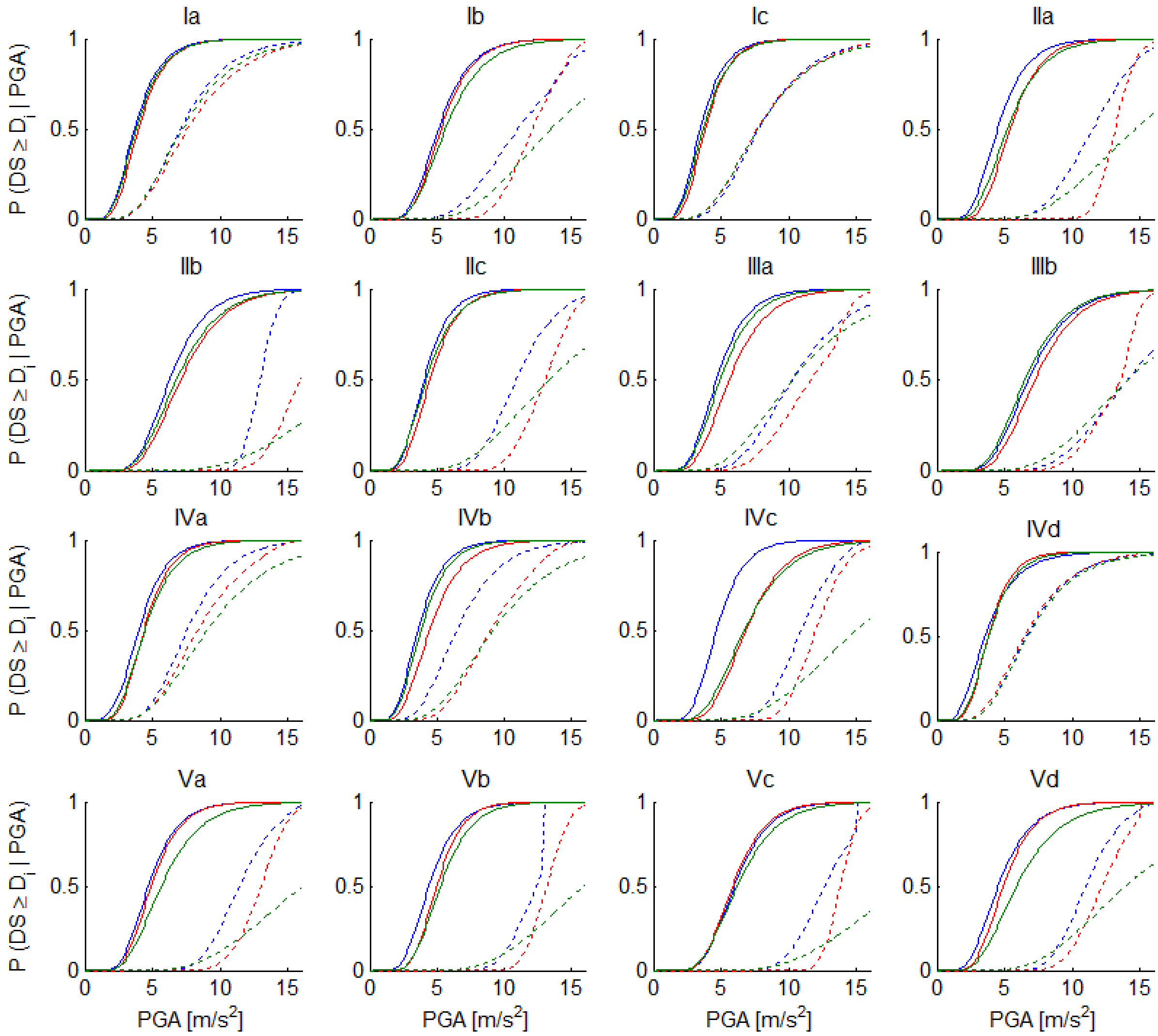

Figure 6. Fragility curves of the bridge system for slight (solid lines) and moderate (dashed lines) damage states. Curves derived from OpenSees, *dirEMA* and *EMA* are represented in blue, red and green respectively).

A first observation can be made on the satisfying performance of the *dirEMA* and *EMA* methods when considering the system fragility curves: it appears that the significant errors observed for the component fragility curves does not propagate to the same extent at the system level. This is explained by the various failure modes (i.e. individual component events) that may lead to a given system damage state: if some of the component fragility curves are inaccurately predicted but do not cause global system's failures, their impact on the probability of damage at system level is dramatically reduced.

The simplified models perform especially well for the slight damage state,which can be expected as it relies on elastic behaviour, while some biases start to appear at the moderate state: in general, both *dirEMA* and *EMA* approaches tend to provide slightly non-conservative results. The performance of both simplified methods is roughly equivalent, even though the *dirEMA* approach seems to give slightly better predictions for the moderate damage state. Finally, the models containing a high proportion of fixed bearings (e.g. *Ib*, *IIb*, *IVb*, *IVc*) generate the highest prediction errors, as opposed to expansion bearings that are associated with a simpler stiffness model (see Figure 3).

Discussion and sensitivity analysis

In order to provide a systematic comparison of the different approaches, 5% - 95% confidence intervals are computed for the system fragility curves: thanks to a bootstrap procedure, the estimated confidence intervals enable to account for the uncertainties due to the fragility curve derivation technique (i.e. GLM regression) as well as the record-to-record variability. For each of the 100 bootstrap samples, the component fragility curves are derived and assembled at the system level: the resulting confidence intervals are then obtained from the 5% and 95% percentiles of the different damage probabilities that are found at each intensity level. If the confidence intervals of two fragility curves are constantly overlapping over the considered range of seismic intensity, then there is a non-negligible possibility that both fragility models are statistically equivalent. An example is shown in Figure 7, where the confidence intervals of *OpenSees* and *dirEMA* are compared: it can be seen that the confidence intervals are overlapping for damage state D1, while this is not always the case for damage state D2.

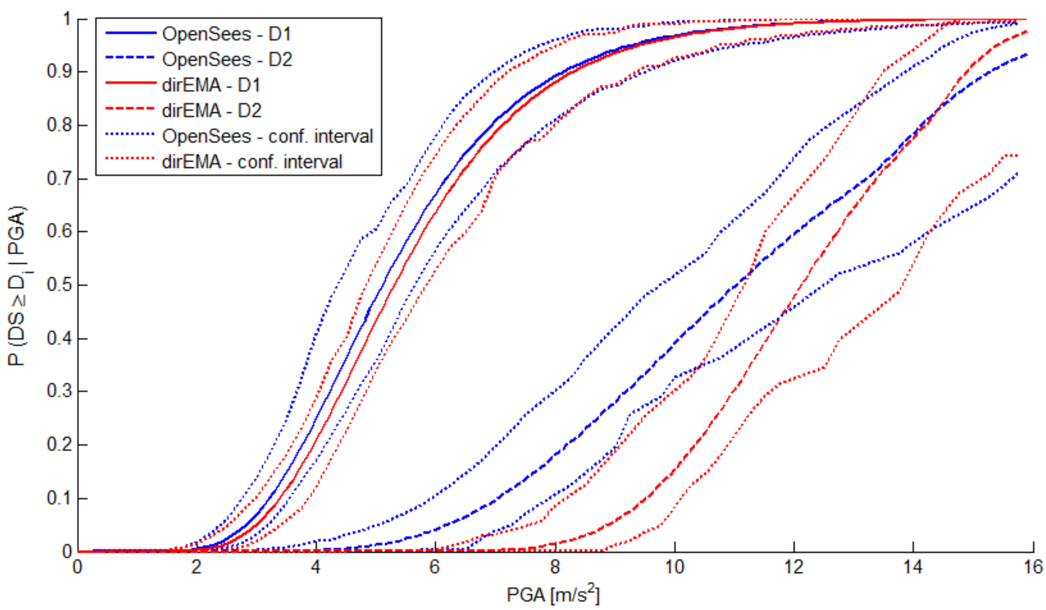

Figure 7.Example of overlapping confidence intervals for the system fragility curves of bridge model *Ib*

It has been observed that the *EMA* fragility curves are usually associated with much narrower confidence intervals than the *OpenSees* or *dirEMA* ones: this is due to the fact that *EMA* directly provides a distribution of intensity values for each damage threshold, whereas an optimization process has to be carried out over the cloud of data points in the other methods. The issue of record-to-record variability might also have less influence on the final fragility estimations in the case of the *EMA* results, due to the scaling of the same records for each damage state.

Finally, based on the comparison of the confidence intervals, conclusions can be drawn on whether the differences between the system fragility curves are of any statistical significance, as summarized in Table 2.

Table 2. Combinations of bridge models and analytical methods for which the differences with the OpenSees results are not statistically significant.

	DS	**Ia**	**Ib**	**Ic**	**IIa**	**IIb**	**IIc**	**IIIa**	**IIIb**	**IVa**	**IVb**	**IVc**	**IVd**	**Va**	**Vb**	**Vc**	**Vd**
EMA	D1	*X*	X	X		X	X	X	X	X	X		X		X	X	
	D2	*X*		X				X	X				X				
dirEMA	D1	X	X	X	X	X	X	X	X	X	X		X	X	X	X	X
	D2	X		X		X		X	X	X		X	X				X

It can therefore be concluded that the *dirEMA* approach is more accurate than the *EMA* one (i.e. *EMA* is correct in 50% of the cases, and *dirEMA* in 75% of the cases) and that it can be applied to a majority of bridge models, especially the ones with fewer components (i.e. models *I*, *II* and *III*). A drop in the accuracy between the slight and moderate damage states can especially be observed for the *EMA* method. However, it should be stressed that the two simplified methods are conceptually different in the way they make use of the input ground motions: the *dirEMA* procedure directly uses the dataset of records (i.e. same approach as the OpenSees analyses), while the *EMA* approach requires the scaling of each record for each limit states. This difference in the treatment of the record-to-record variability may also explain why the fragility curves developed through *dirEMA* tend to be closer to the OpenSees results.

Conclusion

This paper has resulted in a critical appraisal of the DDBA method, which has been developed for the simplified seismic analysis of bridge systems. It has been shown that the use of natural ground motions as input to the procedure of Cardone et al. (2011) is feasible in practice and that it leads to the quantification of the standard deviation β, instead of requiring the selection of an arbitrary value. A variant – *dirEMA* – of the DDBA method has also been proposed, where the structural response is directly estimated from an unscaled ground motion, through an iteration loop. A rigorous comparison of the simplified approaches with nonlinear dynamic analysis, for a variety of bridge models with different types of deck and pier-to-deck connections, has led to the following conclusions:

- The performance of the *dirEMA* procedure is globally equivalent to the *EMA* approach over the different bridge models, which enables to validate the use of *dirEMA* as a viable alternative to the more cumbersome *EMA* method.
- The use of the system reliability method for the assembly of the component fragility curves results in much smaller error rates at the system levels, since some bridge components do not participate to the predominant failure mode.
- Thanks to the analysis of their respective confidence intervals, the system fragility curves do not present statistically significant differences for a majority of the models: the simplified approaches perform better when a reduced number of components are assembled in series, as opposed to models *IV* and *V*. Both *EMA* and *dirEMA* procedures tend to be less accurate when mainly fixed bearings are present: this observation is linked to the complexity of the stiffness and damping models that describe each component, which may be the source of initial errors in the estimation of the component's response. Therefore special care should be devoted to the definition of the component behaviour, especially regarding the selection of a relevant equivalent damping relation for each component type.
- The accuracy of the simplified methods decreases when moderate damage states are considered (i.e. D2 here): once a given level of ductility ratio is exceeded for a component, the estimation of the deformation in the nonlinear range becomes more and more subject to modelling errors (e.g. equivalent damping, number of hysteretic cycles, etc.). However, moderate damage is usually difficult to predict as the system is in hybrid condition. Once significant damage is reached, it can be expected that

differences may be lower, as they depend less on hysteretic behaviour and more on the failure of one dominant component and the way collapse is defined.

Finally, the above results have only been derived with single bridge models, where the mechanical properties are deterministically set. A probabilistic sampling of these properties could be helpful to further investigate the validity domain of both *dirEMA* and *EMA* (e.g. relative stiffness between the piers and the deck). By way of a closing remark, the DDBA methods are helpful to quickly model a bridge system and analyse its seismic fragility, especially when performing risk analyses over an extended geographic area: however, their use should be limited for moderate seismic intensity levels, since their accuracy for heavier damage states (e.g. collapse) could not be properly quantified yet.

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