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Displacement-based seismic assessment of reinforced concrete bridges

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ABSTRACT: The fundamental step of any Displacement-Based seismic Assessment (DBA) procedure is the definition of the Performance Displacement Profile (PDP) of the bridge, corresponding to the inelastic deformed shape of the bridge associated with the attainment of given damage states in some critical elements (piers, abutments, joints, bearing devices, etc.). The PDP definition is straightforward in the longitudinal direction, while many approaches can be followed to derive the PDP in the transverse direction. In this paper, a practice-oriented DBA procedure, based on a component modelling approach and Effective Modal Analysis (EMA), is presented. In the paper, the key aspects of the proposed procedure, including the definition of the deformation limits for a number of vulnerable elements and the basic steps of EMA for the definition of the PDP in the transverse direction, are presented first. Next, the proposed procedure is applied to a typical highway bridge of the Greek Egnatia Motorway.

1 INTRODUCTION

Recent earthquakes have repeatedly demonstrated the seismic vulnerability of existing bridges, due to their design based on gravity loads only or inadequate levels of lateral forces (Priestley et al. 1996). Bridges are of great importance after an earthquake, for allowing civil protection interventions and first aid organizations. As a consequence, they should be seismically assessed and, if needed, retrofitted. From this point of view, the development of a practice–oriented seismic assessment procedure, which is effective but also sufficiently simple to be applied to a large stock of bridges, could be very useful.

A few years ago, (Priestley et al. 2007) have proposed a displacement-based seismic assessment (DBA) approach for existing SDOF structures and multistory reinforced concrete buildings. As outlined in (Priestley et al. 2007), the main difficulties in the application of the DBA approach are: (i) the determination of which element first reaches a specified damage state or performance limit and (ii) what is the corresponding displacement profile throughout the structure. For that reason, they suggest the use of adaptive Push Over Analyses (POA) for the definition of the so-called limit-state displacement profile. As a matter of fact, however, pushover analysis requires specific modelling skills and implies significant computational efforts and time. In recognition of the aforesaid difficulties, herein, particular attention is paid to the development of suitable simplified approaches for the definition of the Performance Displacement Profile (PDP) of the bridge, to overcome the difficulties associated with POA.

In this context, a DBA procedure for Reinforced Concrete (RC) bridges has been recently proposed in (Cardone and Perrone 2013). The proposed DBA procedure can be applied to both continuous deck bridges and bridges with independent adjacent decks with internal joints. Moreover, the critical elements of the bridges are not limited to piers only but they include also piers, abutments, joints, bearing devices and shear keys. The fundamental step of the proposed DBA procedure is the definition of the bridge PDP, corresponding to the bridge inelastic deformed shape associated with the attainment of given Damage States (DSs) in some critical elements of the bridge. DSs are defined in terms of suitable displacement/deformation limits for piers, abutments, joints, bearing devices, and shear keys. The PDP associated with the selected DS is converted into the performance displacement of an equivalent elastic SDOF system, according to the principles of the Direct Displacement-Base Design (DDBD) method (Priestley et al. 2007). The return period and associated PGA value corresponding to the attainment of the selected DS is then evaluated following the DBA approach proposed in (Priestley et al. 2007). Based on the PGA values obtained, a number of fragility curves are derived to express the seismic vulnerability of the bridge under a semiprobabilistic point of view.

Finally, the seismic risk of the bridge is evaluated as convolution integral of the product between the fragility curves and the seismic hazard curve of the bridge site. In this paper, the key aspects of the proposed procedure are presented first. Next, the proposed DBA procedure is applied to a real case study represented by a typical highway bridge of the Greek Egnatia Motorway.

2 DISPLACEMENT BASED ASSESSMENT OF BRIDGES

The fundamental steps of the proposed DBA procedure can be summarized as follows: (i) definition of the displacement/deformation limits for the structural elements of the bridge, (ii) evaluation of the Performance Displacement Profile (PDP) associated to a given Damage State (DS) or Performance Level (PL) of the structure, (iii) conversion of the PDP into the performance displacement of an equivalent SDOF model and evaluation of the corresponding equivalent damping level, (v) estimation of the return period and PGA value associated with the performance displacement profile and, finally, (iv) evaluation of fragility curve and seismic risk index.

In the proposed practice-oriented procedure, bridge modeling is performed according to a Structural Component Modeling (SCM) approach (Priestley et al. 1996), in which the bridge is schematized as one or more independent elastic beams, modeling the bridge deck(s), mutually connected by means of a series of nonlinear springs, modeling piers, abutments and bearing devices. The bilinear skeleton curves of piers are derived based on either approximate relationships (Priestley et al. 1996, FHWA 1996) or preliminary moment-curvature analysis. Shear strength is also taken into account. The bilinear skeleton curve of bearing devices is determined considering different possible failure mechanisms (e.g. shear failure, sliding and roll-over mechanisms for neoprene pads) and assuming a frictional (concrete-to-concrete or rubber-to-concrete) post-failure behavior up to deck unseating. The skeleton curve of seat-type abutments, in the longitudinal direction, is defined considering a compression-only elastic-perfectly-plastic behavior, with initial gap equal to the width of the deck-abutment joint, and mechanical behavior governed by the backwall-backfill passive resistance interaction, in accordance with (Caltrans 2006) provisions. The skeleton curve of shear keys is derived considering both the sliding-shear and strut-and-tie collapse mechanisms.

The equivalent-linear-elastic models used within EMA are based on the secant stiffness, derived from the nonlinear skeleton curve of each element, at the displacement of the k-th step of the iterative analysis (see section 2.2). More details on bridge modeling can be found in (Cardone 2014).

The translational and rotational mass of the deck(s) is lumped in the center of mass of each span. If necessary, a tributary mass of the piers (1/3 of the pier height plus the cap beam) is taken into account.

2.1 Definition of deformations/displacement limits

The approach followed in this study was that of defining displacement/deformation limits for four seismic Performance Levels (PLi, i = 1,..4), differing in terms of damage severity for each bridge element. Herein, the attention is focused on piers, unbolted neoprene pads, seat-type abutments and shear keys, because they are found in the selected case study (see Section 3). More details on displacement/deformation limits of other bridge elements can be found in (Cardone 2014).

For piers, flexural damage is defined in terms of concrete compression or steel tension strain limits (see Tab 1), whichever occur first. For instance, PL1 is deemed attained when the first of the following events occurs: (i) the maximum compression strain of concrete attains a limit value of 0.004, (corresponding to concrete spalling), (ii) the maximum reinforcement tensile strain attains a limit value of 0.015. Similarly, PL3 is deemed attained when either (i) the maximum reinforcement tensile strain attains a limit value of $0.6\varepsilon_{su}$, ε_{su} being the steel ultimate strain, or (ii) the maximum compression strain of concrete reaches a limit value (corresponding to stirrups failure) of:

$$\varepsilon_{c,DS3} = 0.004 + 1.4\frac{\varepsilon_{su} f_{yh} \rho_v}{f'_{cc}} \leq 0.02 \qquad (1)$$

where f_{yh} is the transverse reinforcement yield strength, ρ_v is the volumetric ratio of transverse reinforcement and f'_{cc} is the compression strength of the confined concrete.

For neoprene pads, damage is defined by specific displacement limits as shown in Table 1, where d_{fr} and d_{roll} are the relative displacements corresponding to the attainment of the friction resistance (concrete-to-rubber) and roll-over mechanism (Konstantinidis et al. 2008), respectively; d_{pad} and d_{uns} are the relative displacements corresponding to the pad dimension and deck unseating, respectively. The displacement limits for abutments are defined as a function of d_{gap}, $d_{y,ab}$ and $d_{u,ab}$ which are the deck displacements corresponding to joint closure, attainment of the passive resistance and collapse of the abutment-backfill system, respectively. Also shown in Table 1 are the displacement limits for RC shear keys, in which $d_{gap,t}$ corresponds to gap closure and d_{usk} to shear key failure.

Table 1 Damage states for each structural element of the bridge.

ELEMENT (Failure Modes)		PL1 Slight Damage	PL2 Moderate Damage	PL3 Severe Damage	PL4 Collapse Prevention
Piers (Flexure)		$\varepsilon_{c,DS1}$=0.004	$\varepsilon_{c,DS2}$= $\varepsilon_{c,DS1}$+1/3($\varepsilon_{c,DS3}$-$\varepsilon_{c,DS1}$)	$\varepsilon_{c,DS3}$	1.5 $\varepsilon_{c,DS3}$
		$\varepsilon_{s,DS1}$=0.015	$\varepsilon_{s,DS2}$= $\varepsilon_{s,DS1}$+1/3($\varepsilon_{s,DS3}$-$\varepsilon_{s,DS1}$)	0.6ε_{su} ≤ 0.05	0.9ε_{su} ≤ 0.08
Unbolted Neoprene Pads	(Sliding)	d_{fr}	d_{fr} +1/3(d_{pad} - d_{fr})	d_{pad}	d_{uns}
	(Roll-over)	d_{roll}	d_{roll} +1/3(d_{pad} – d_{roll})	d_{pad}	d_{uns}
Abutments		d_{gap}	$d_{y,ab}$	$d_{y,ab}$ + 2/3($d_{u,ab}$- $d_{y,ab}$)	$d_{u,ab}$
Shear Keys		-	-	d_{usk}	-

2.2 Evaluation of the performance displacement profile

The performance displacement profile is defined with an iterative eigenvalue analysis, referred to as Effective Modal Analysis (EMA). The EMA approach adopted in this study is inspired to the procedure proposed by (Kowalsky 2002) for continuous deck concrete bridges with monolithic pier-deck connections, with or without abutment restraints in the transverse direction. In this study, EMA is applied to continuous deck bridges with deformable pier-deck connections through bearings. EMA is an iterative procedure based on equivalent linear models of the bridge components. The EMA provides the effective modal shape of the bridge for each selected PL (see Fig. 1) and it can be easily implemented in electronic spreadsheet.

In the first step of the EMA, a trial value is assumed for the effective stiffness of each bridge element. Reference to the elastic stiffness can be made for elements that are supposed to remain elastic (e.g. superstructure and abutments). For piers and/or bearings a suitable effective stiffness (ranging from 10 to 50% of the elastic stiffness) can be assumed as initial trial value. The closer the estimates of the effective stiffness are to the actual values, the faster the procedure converges.

A modal analysis is then performed. The resultant first modal shape (ϕ_i) is recorded as displacement pattern of the bridge. Next, the displacement pattern is scaled, based on the displacement corresponding to the attainment of a given DS in a (trial) critical element of the bridge (pier, abutment, bearing, etc.), to get a tentative PDP of the bridge (see Fig. 1). The element that first reaches a given displacement limit in the current displacement pattern is recognized as the critical element of the bridge and its displacement is the critical displacement (D_{cr}). The displacements of the other elements d_i are obtained from ϕ_i in proportion to the ratio D_{cr}/ϕ_{cr}.

Higher modes effects shall be considered, through standard SRSS modal combination rule, when the first mode mass participation ratio results lower than 70–75% of the total mass (Kowalsky 2002). In the next step, the secant stiffness of the elements is updated, to reflect the current displacements (d_i) obtained from the first iteration. With the values of secant stiffness thus obtained, a new modal analysis is performed and a new PDP is derived. The iterative procedure continues till there is no significant change in the PDP between two consecutive steps of analysis. The iterative procedure normally converges in 3–5 iterations even for irregular bridges.

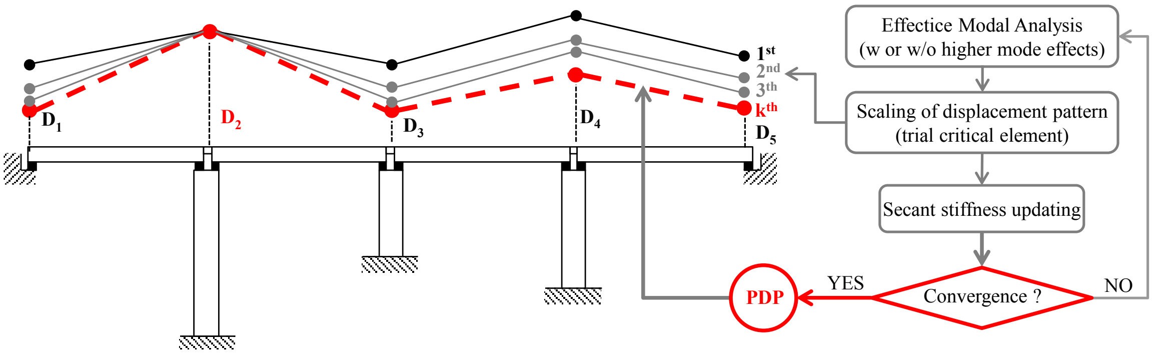

Figure 1: Definition of the performance displacement profile through the effective modal analysis.

2.3 **Equivalent SDOF system**

The PDP defined in the previous step is converted into the Performance Point ($S_{a,PL}$, $S_{d,PL}$) of an equivalent linear SDOF system, based on the fundamental equations of the DDBD method (Priestley et al. 2007), herein slightly adapted to bridge structures as follows:

$$S_{d,PL} = \frac{\sum_j \left(m_j \cdot D_{j,PL}^2 + I_{Rj} \cdot \theta_{j,PL}^2\right)}{\sum_j m_j \cdot D_{j,PL}} \tag{2}$$

$$S_{a,PL} = \frac{V_{b,PL}}{M_{e,PL} \cdot g} = \frac{V_{b,PL} S_{d,PL}}{\sum_j m_j \cdot D_{j,PL} \cdot g} \tag{3}$$

$$T_{e,PL} = 2\pi \cdot \sqrt{\frac{M_{e,PL}}{K_{e,PL}}} = 2\pi \cdot \sqrt{\frac{S_{d,PL}}{S_{a,PL}\, g}} \tag{4}$$

where m_j and $D_{j,PL}$ are the translational mass and horizontal displacement of the center of mass of the j-th deck, respectively, I_{Rj} and $\theta_{j,PL}$ are the rotational mass and rotation about the vertical axis of the j-th deck, respectively, $V_{b,PL}$ is the base shear of the bridge and $T_{e,PL}$, $M_{e,PL}$, $K_{e,PL}$ are the effective period mass and stiffness of the equivalent SDOF system, respectively.

The seismic demand associated to each PL is represented by an over-damped elastic response spectrum, whose seismic intensity is still unknown at this step of the analysis. This requires the evaluation of the equivalent viscous damping of the bridge associated to the selected PL. To this end, the following routine is followed: (i) derive the actual displacement of each structural member, from the performance displacement profile of the bridge, (ii) evaluate the equivalent viscous damping of each structural member, based on its displacement/ductility demand, (iii) combine the damping contributions of all the structural members to get the equivalent viscous damping of the entire bridge.

Reference to the damping relationship proposed by (Grant et al. 2004) has been made to estimate the equivalent viscous damping of piers and bearing devices:

$$\xi_{j,PL} = \mu_j^{\lambda} \xi_{el,j} + \xi_{hyst,j} = \gamma \xi_{el,j} + a\left(1 - \frac{1}{\mu_j^b}\right)\left(1 + \frac{1}{\left(T_{e,j} + c\right)^d}\right) \tag{5}$$

where λ, a, b, c and d are coefficients dependent on the shape of the hysteretic cycles of the j-th element and T_{ej} is effective period of the j-th element. The global equivalent viscous damping of the bridge ($\xi_{e,pl}$) is evaluated by weighting the contributions of each structural member as a function of the strain energy of each element at its maximum displacement (Kowalsky 2002):

$$\xi_{e,PL} = \frac{\sum_{j=1}^{n} \xi_{j,PL} \cdot F_{j,PL} \cdot d_{j,PL}}{\sum_{j=1}^{n} F_{j,PL} \cdot d_{j,PL}} \tag{6}$$

where $\xi_{j,PL}$, $F_{j,PL}$, and $d_{j,PL}$ are the damping ratio, force level and displacement amplitude, respectively, of the j-th element at the selected PL.

Once the equivalent viscous damping of the bridge has been determined, a proper damping reduction factor, η_{PL}, is computed. In this study, reference to the damping reduction factor adopted in an old version of the Eurocode 8 (CEN 1998) has been made. The PGA value associated to the selected PL is determined with following procedure: (i) evaluate the bridge elastic displacement capacity as $S_{d,el,PL} = S_{d,,PL}/\eta_{PL}$ (ii) enter the elastic response spectra for the bridge site with the effective period, $T_{e,PL}$, and elastic displacement capacity, $S_{d,el,PL}$, to identify the associated seismic demand spectrum (see Fig. 2a), (iii) derive the associated return period, $T_{r,PL}$, (or mean annual frequency of occurrence, $MAFE_{PL}$), (iv) from the hazard curve, determine the corresponding seismic intensity level, PGA_{PL} (see Fig. 2b).

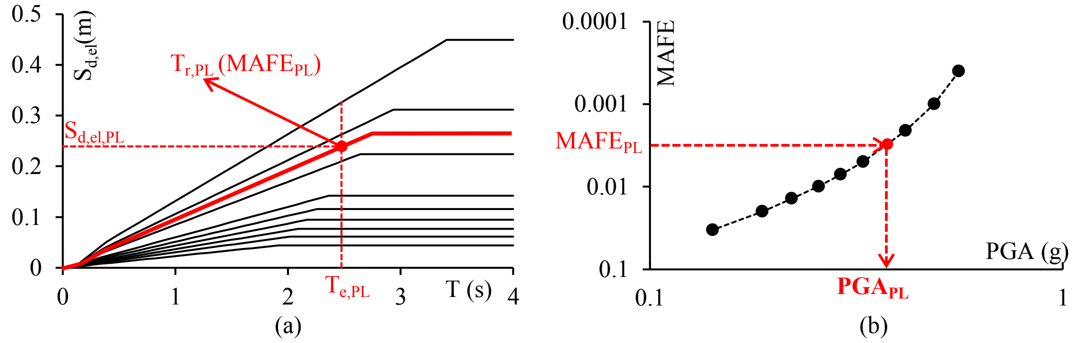

Figure 2: (a) Evaluation of the return period associated with the performance point of the structure, (b) Derivation of PGA_{PL} from the hazard curve of the bridge site.

2.4 Fragility curves and seismic risk index

The PGA_{PL} values obtained with the proposed DBA procedure can be deemed to represent an estimate of the median values (50% probability of exceedance) of the peak ground acceleration associated with the selected PL. They can be used to derive a number of fragility curves, which provide the probability of exceedance of the selected PL, as a function of the PGA of the expected ground motions. Fragility curves are typically expressed by a lognormal cumulative probability function:

$$P(D \geq PL | PGA) = \Phi\left[\frac{1}{\beta_{PL}} \ln\left(\frac{PGA}{PGA_{PL}}\right)\right] \tag{7}$$

in which $P(\bullet)$ is the probability of the Damage (D) being equal to or greater than the selected PL, for a generic seismic intensity (PGA), Φ is the standard lognormal cumulative probability function, PGA_{PL} is the median value of the seismic intensity provided by the DBA procedure and β_{PL} is the lognormal standard deviation which takes into account different sources of uncertainty. According to (ATC 2012), the fragility dispersion can be computed as follows:

$$\beta_{PL} = \sqrt{\beta_c^2 + \beta_{r,PL}^2 + \beta_{gm,PL}^2 + \beta_m^2} \tag{8}$$

where:

- β_c is the dispersion related to capacity variability, due to construction quality and material properties. According to (ATC 2012), β_c ranges from 0.1 (superior quality) to 0.4 (limited quality)
- $\beta_{r,PL}$ is the dispersion related to record-to-record variability. An estimation of $\beta_{r,PL}$ can be drawn from Table 5.27 of FEMA P-58 (ATC 2012);
- $\beta_{gn,PL}$ is the dispersion related to ground motion variability, which can be taken equal to $0.5(lnPGA_{84} - lnPGA_{16})$ where PGA_{84} and PGA_{16} are the 84-th and 16-th percentile values of the expected PGA for the bridge site, derived from the National Seismic Hazard map;
- $\beta_{m.}$ is the dispersion relevant to the quality and completeness of the analytical model. According to (ATC 2012), β_m can be assumed equal to 0.4 based on the modeling assumption made in the proposed procedure.

Finally, a seismic risk index is computed as convolution integral (see Fig. 4c) of the product between the seismic vulnerability of the bridge (V), expressed by the fragility curves (see Fig. 4b), and the seismic hazard of the bridge site (P), expressed by the hazard curve (see Fig. 3c). The seismic risk index thus obtained provides the probability of exceedance of the selected PL, conditioned to the seismic hazard of the bridge site. It can be used by the network manager for screening and prioritization operations of a large bridge inventory, in view of possible retrofit measures.

3 APPLICATION TO A CASE STUDY

The 2nd Kavala ravine bridge (hereafter called Kavala bridge for simplicity) of the Greece Egnatia Motorway (see Fig. 3a, b) has been selected. It consists of four 45m long spans supported by three single shaft piers characterized by a square hollow section with 4m x 4m dimensions and 0.5m thickness. The central pier has an effective height of 51.9m while the other two piers have an effective height of 28.5m. The longitudinal reinforcement ratio of each pier is of the order of 1%. Concrete of class B35 is assumed for deck pre-stressed beams, while concrete of class B25 for piers, abutments and foundations. Rebars are made of BSt 500/550 steel. The connection between deck beams and pier is realized by four unbolted neoprene pads. Each pier features on the top a stopper mechanism (see Fig. 3b) realized to provide transverse restraint to the deck during moderate-to-strong earthquakes. The weight of the deck on abutments is sustained by unbolted laminated elastomeric bearings.

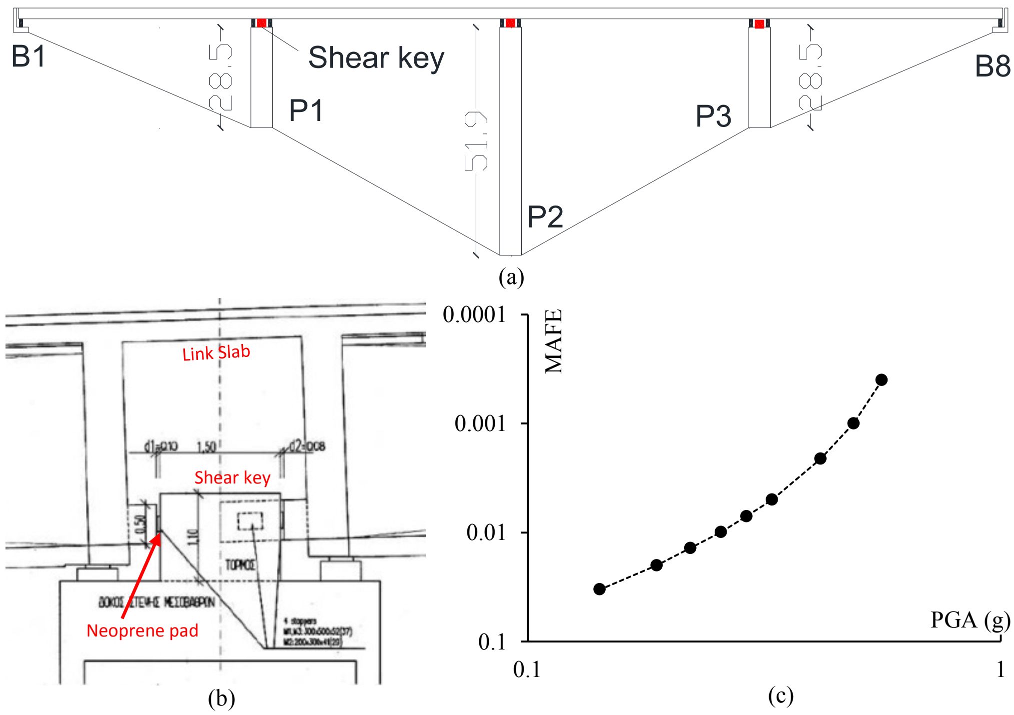

Figure 3: The 2nd Kavala bridge of the Egnatia Motorway: (a) Schematic bridge layout, (b) Details of link slabs and shear keys (c) Hazard curve of the bridge site.

The analytical model of the bridge has been implemented in SAP2000_Nonlinear, following the basic modeling assumptions discussed in section 2. The flexibility of the deck has been taken into account, considering the effective flexural stiffness of the transverse cross section of the deck. The piers of Kavala bridge exhibit a ductile flexural behavior with ultimate displacement ductility of the order of 2.85. Considering the presence of shear keys, a monolithic pier-deck connection has been assumed in the application of the proposed DBA procedure. The elastomeric bearings placed on abutments exhibit a sliding failure mechanism with a concrete-to-rubber friction coefficient of the order of 40%, while those placed on the piers remain elastic, mainly due to the role of shear keys.

Following the EMA approach, a series of iterative modal analyses have been performed. For PL1 the definition of the PDP is straightforward (no iterations are needed) since all the bridge elements are still elastic. For PL2 and PL3, instead, only a few iterations were needed to get the PDP of the bridge.

Figure 4a shows the PDPs of the bridge in the transverse direction derived following the EMA approach. As can be seen, the inelastic deformed shape of the bridge changes passing from PL1 to PL2 and PL3. In particular, at PL1 the sliding resistance of the bearings placed on abutments is attained. At PL2, yielding of piers P1 and P3 occurs. At PL3, inelastic deformations tend to concentrate in piers P1 and P3 (with ductility demand of 1.9) while pier P2 remains elastic. Table 2 summarizes the effective properties of the equivalent linear SDOF system and the PGA values corresponding to the attainment of each PL. It should be noted that reference to a hazard curve compatible with the seismic hazard of the bridge site (see Fig. 3c) has been made in this study.

Table 2 Main results of the proposed DDBA procedure following the EMA approach.

PL	V_{Base} (kN)	$S_{d,PL}$ (m)	$S_{a,PL}$ (g)	K_e (kN/m)	M_e (t)	T_e (s)	ξ_e (%)	η	$S_{d,el}$ (m)	PGA_{PL} (g)
PL1	7641	0.233	0.192	32786	4062	2.21	5.41	0.97	0.240	0.502
PL2	9929	0.384	0.246	25833	4112	2.51	8.63	0.81	0.474	0.806
PL3	10547	0.694	0.260	15201	4128	3.27	15.17	0.64	1.087	1.294

Figure 4b shows the fragility curves associated with the selected PLs. In particular, following the recommendation of FEMA P-58 (ATC 2012), values of dispersion β_{PL} equal to 0.54, 0.58 and 0.60 have been obtained for PL1, PL2 and PL3, respectively.

Finally, Figure 4c shows the seismic risk curves obtained multiplying the hazard curve (Fig. 3c) by the relevant fragility curve (Fig. 4b). From a graphical point of view, the seismic risk index corresponds to the area underneath each risk curve of Figure 4c. Values of the seismic risk index (corresponding to the annual probability of exceedance a given PL) of 0.035% (PL1), 0.01% (PL2) and 0.002% (PL3), have been thus obtained for the bridge under scrutiny.

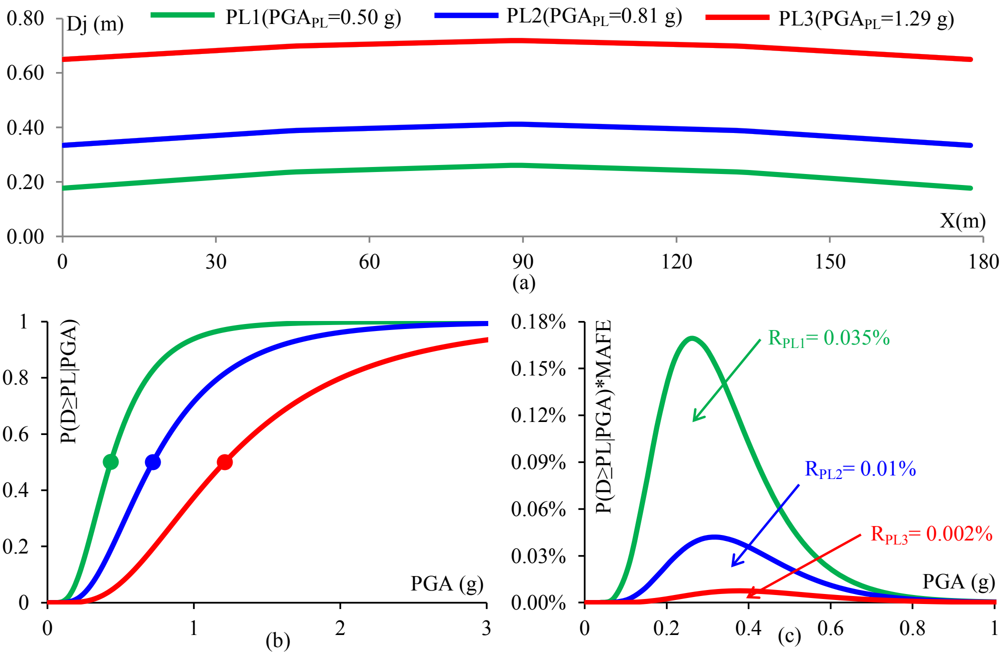

Figure 4: (a)Performance displacement profiles of the bridge in transversal direction, derived following the EMA approach, (b) Fragility curves for the selected PLs; (c) Seismic risk curves for the selected PLs.

4 CONCLUSIONS

A practice-oriented Displacement-Based procedure for the seismic Assessment (DBA) of RC bridges has been presented. Basically, the proposed DBA procedure relies on an Effective Modal Analysis (EMA) to derive the Performance Displacement Profile (PDP) of the bridge, corresponding to given damage states of the critical bridge components. The proposed procedure is able to consider different damage mechanisms and it can be applied to both continuous deck bridges and bridges with independent adjacent decks. Comparisons between DBA predictions and results of accurate analyses (not shown in the paper), show that the EMA approach is a valid alternative to non-linear static procedures for the definition of the PDP, as it allows to take into account higher-modes effects and deck deformability. Unfortunately, at the moment, EMA is not implemented in any structural program. Further work is needed to automatize the proposed DBA procedure.

Although the proposed procedure appears reasonably accurate and very promising for the seismic assessment of large bridge inventories, there are a number of aspects that require further investigation.

An aspect that requires more research is the modeling strategy of abutment and its hysteretic behavior. In particular, more studies are required for computing the equivalent viscous damping of abutments especially in the longitudinal direction, where the abutment response may play an important role in the bridge vulnerability, especially when the deck gap is small, being designed considering thermal expansion only. Equivalent viscous damping of hysteretic response with gap should be included in future research.

Further studies are also needed to include P-delta effects in the proposed procedure, especially for bridges supported on sliding bearings.

The simplified method for the derivation of fragility curves proposed in this study is based on the recommendations reported in FEMA P-58 (ATC 2012). These recommendations are not fully adequate for the seismic assessment of bridges. Modeling uncertainty and record-to-record variability, in particular, require more research, e.g.: in the DBA framework, dispersion may be related to the effective period and expected failure mechanism (pier plastic hinges, pier shear failure, abutment failure, bearing damage, deck unseating, etc.).

Finally, further studies are needed to improve the accuracy of the proposed DBA procedure for skewed and curved bridges, as well as for bridges with independent adjacent decks. In the latter case the assumption of independent decks may be not appropriate when strong deck pounding is expected. At the same time, the assumption of continuous deck in presence of link slabs (like in the case study presented in this paper) may be not accurate and damages related to link slab may be included.

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