Direct Displacement-Based Assessment of bridges: a case study

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SUMMARY:
Two different procedures for the Direct Displacement-Based seismic Assessment (DDBA) of bridges have been recently developed by the authors of this paper. Herein, they are applied to a case study derived from a typical highway bridge of the Greek Egnatia Motorway. The numerical predictions of the two DDBA procedures are compared to the results of Nonlinear response Time-History Analysis (NTHA), carried out using a set of seven natural records, compatible with the Eurocode 8 response spectrum for soil type B, scaled to the PGA values provided by the DDBA procedures for different Damage States (DSs) of the structure.

Keywords: Bridges, Seismic assessment, Damage states, Effective modal analysis, Adaptive pushover analysis.

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 must be seismically assessed and, if needed, retrofitted. From this point of view, the development of seismic assessment procedures which give reliable results and, at the same time, are sufficiently simple to be applied to a large stock of bridges could be very useful.

Traditional seismic assessment is basically based on the comparison between estimated base shear capacity and base shear demand specified by a seismic code. The base shear demand is found by reducing the elastic base shear corresponding to the elastic stiffness of the structure, by a code-specified force-reduction or behavior factor. The problems with this approach are that no assessment is made of the actual collapse mechanism, inelastic deformed shape and ductility demand of the structure. In recognition of the limitations of force-based design methods, several researchers have started proposing displacement-based approaches for the seismic design and assessment of structures, with the aim of providing improved reliability in the engineering process, by more directly relating computed response and expected structural performance. One of the most attractive displacement-based design approaches is the Direct Displacement-Based Design (DDBD) method proposed by Priestley et al. (2007). The fundamental goal of DDBD is to obtain a structure which will respond according to a given target displacement profile, when subjected to earthquakes consistent with a given reference response spectrum. The DDBD method has been specialized to different structural types, including frame buildings (Pettinga and Priestley, 2005), wall buildings (Sullivan et al., 2005), continuous deck bridges (Kowalsky, 2002) and structures with seismic isolation (Cardone et al., 2008; Cardone et al., 2010).

The AT1_L2 project of the ReLUIS 2010-2013 research program is aimed at the development and verification of Direct Displacement-Based Assessment (DDBA) procedures for different structural types. The task 7 of the AT1_L2 research project, whose main investigator is one of the authors of this paper, deals with existing bridges. Two different procedures for the Direct Displacement-Based seismic Assessment (DDBA) of bridges have been recently developed by the authors of this paper.

Basically, the first procedure is a revised improved version of the so-called Inverse Adaptive Capacity Spectrum Method (IACSM), recently proposed by the same authors for the seismic assessment of simply supported deck bridges (Cardone et al., 2011). The second procedure is derived from the displacement-based design method proposed by Kowalsky (2002) for continuous deck RC bridges with monolithic pier-deck connections.

In this paper the proposed DDBA procedures are applied to a case study derived from a typical highway bridge of the Greek Egnatia Motorway. The numerical predictions of the DDBA procedures are compared to the results of Nonlinear response Time-History Analysis (NTHA), carried out using a set of seven natural records, compatible with the Eurocode 8 response spectrum for soil type B, scaled to the PGA values provided by the DDBA procedures for selected damage states of the structure

2. DIRECT DISPLACEMENT-BASED ASSESSMENT PROCEDURES

The main steps of the proposed DDBA procedures are the same and can be summarized as follows: (i) structural modeling, (ii) derivation of a number of critical displacement profiles associated to different Damage States (DSs) of the structure, (iii) evaluation of the corresponding PGA values, based on the comparison between the seismic capacity of an equivalent SDOF model of the structure and the seismic demand of the expected ground motions represented by overdamped elastic response spectra and (iv) estimation of the vulnerability and seismic risk of the bridge under a (semi-)probabilistic perspective.

The fundamental step of both procedures is the definition of the so-called critical displacement profile of the bridge, corresponding to the inelastic deformed shape of the bridge associated to the attainment of a selected DS in a critical element of the bridge (the structural element where the selected DS occurs first). The main difference between the two procedures is that in the first procedure (DDBA1) the critical displacement profile is derived from a Displacement Adaptive Pushover (DAP) analysis of the bridge (Antoniou and Pinho, 2004) while in the second procedure (DDBA2) the critical displacement profile is derived through an Iterative Eigenvalue Analysis (IEA) (Kowalsky, 2002). Both procedures rely on the principles of the DDBD method (Priestley et al., 2007) to convert the nonlinear MDOF model of the bridge into an equivalent linear SDOF system.

The proposed DDBA procedures provide the earthquake intensity levels (i.e. PGA values) corresponding to the attainment of the selected Damage States (DSs). The seismic vulnerability of the bridge is then described by means of a number of fragility curves, based on the PGA values associated to each DS. Finally, a seismic risk index of the bridge is evaluated as convolution integral of the product between fragility curves of the bridge and seismic hazard curve of the bridge site.

2.1. Bridge modelling

According to the Structural Component Modelling (SCM) approach (Priestley et al., 1996), the bridge can be schematized as one or more independent elastic beams, modelling the bridge deck(s), mutually connected by means of a series of nonlinear or equivalent linear elastic springs, modelling piers, abutments and bearing devices. The translational and rotational mass of the deck(s) is lumped in the centre 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. The equivalent linear elastic model, used within the IEA in the second DDBA procedure, is based on secant stiffness derived from the nonlinear skeleton curves of each element at the displacements of the k-th step of the iterative analysis.

The bilinear skeleton curves of piers are derived based on either approximate relationships (Priestley et al., 1996, Priestley et al., 1997, FHWA, 1996) or preliminary moment-curvature analysis of the critical section(s) of the pier. The shear strength of the pier as a function of the top pier displacement is taken into account and compared to the flexural resistance of the pier to determine the actual flexural/shear behavior of the pier.

The seismic behavior of seat-type abutments in the longitudinal direction of the bridge is captured with a compression-only non-dissipative elastic-perfectly-plastic skeleton curve with initial gap equal to the width of the deck-abutment joint. The mechanical properties of the abutments are derived from a combination of design recommendations (Caltrans 2006) and experimental test results on seat-type

abutments with piles.
Basically, five different types of bearing devices are found in highway bridges realised between the '60s and the '80s, namely: (i) steel hinges, (ii) dowel steel bars, (iii) sliding bearings, (iv) steel pendulum and roller bearings and (v) either bolted or unbolted neoprene pads. For brevity, in this paper the attention is focused on neoprene pads only, since they are used in the selected case study. The bilinear skeleton curve of neoprene pads is determined considering different failure mechanisms (Kostantinidis et al., 2008), including: (i) rubber shear failure, for bolted neoprene pads, (ii) sliding between neoprene and concrete surfaces and (iii) roll-over mechanisms, for unbolted neoprene pads. More details on the modelling of piers, abutments and bearing devices can be found in (Cardone et al., 2011).
For each structural element, a number of Damage States (DSi, i = 1,..4) are defined, based on the consequences in terms of damage that the attainment of each DS can produce. The DS of piers, abutments and neoprene pads are summarized in Table 2.1, where: θ_y and θ_u are the yield and ultimate rotation of the pier plastic hinge(s), respectively; θ_s is the chord rotation corresponding to premature shear failure; d_{gap}, $d_{y,ab}$ and $d_{u,ab}$ are the deck displacements corresponding to joint closure, attainment of the passive resistance and collapse of the abutment-backfill system, respectively; d_{fr} and d_{roll} are the relative displacements corresponding to the attainment of the friction resistance (concrete to rubber) and roll-over conditions, respectively, for unbounded neoprene pads; d_γ are the relative displacements corresponding to the attainment of given critical values of shear strain (γ) for bolted neoprene pads; d_{pad} and d_{uns} are the relative displacements corresponding to the pad dimension and deck unseating, respectively.

Table 2.1 Damage states for each structural element of the bridge.

ELEMENT (Failure Modes)		DS1 Slight Damage	DS2 Moderate Damage	DS3 Severe Damage	DS4 Collapse Prevention
PIERS	(Flexural)	θ_y	$\theta_y + 1/3\ (\theta_u - \theta_y)$	$\theta_y + 2/3\ (\theta_u - \theta_y)$	θ_u
	(Shear)	-	-	θ_s	$1.1\ \theta_s$
ABUTMENTS (Passive Resistance)		d_{gap}	$d_{y,ab}$	$d_{y,ab} + 2/3(d_{u,ab} - d_{y,ab})$	$d_{u,ab}$
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}
BOLTED NEOPRENE PADS (Shear Failure)		$d_{\gamma=150\%}$	$d_{\gamma=200\%}$	$d_{\gamma=300\%}$	d_{uns}

2.2. Derivation of the target displacement profile

As said before, the main aspect that distinguishes the two proposed DDBA procedures is the approach followed in the definition of the critical displacement profile of the bridge associated to the attainment a selected damage state in a critical element of the bridge.
In the first procedure (DDBA1), a Displacement Adaptive Pushover (DAP) analysis is performed to define the critical displacement profile of the bridge associated to a selected damage state (see Figure 1). The DAP algorithm is implemented in the structural program SeismoStruct (SeismoSoft, 2006), which can be freely downloaded from the Internet. The DAP technique has been preferred to the other conventional (i.e. force-based) adaptive pushover techniques, to better estimate the inelastic deformed shape of the bridge. Compared to multiple-run pushover techniques (such as the Multimode Pushover Analysis (Chopra and Goel, 2001) and the Incremental Response Spectrum Pushover Analysis (Aydinoglu, 2003)), the single-run DAP technique results decidedly more suitable for DDBA applications.
The DAP analysis provides the capacity curve of the structure, separately in the longitudinal and transverse direction of the bridge. Each point of these capacity curves is univocally associated to a given inelastic deformed shape of the bridge, hence DS of its structural elements. Any inelastic deformed shape derived from DAP, therefore, can be considered as a potential critical displacement

profile of the bridge and used for the subsequent calculations.
In the second procedure (DDBA2) an Iterative Eigenvalue Analysis (IEA) is performed to derive the critical displacement profile of the bridge. In the first iteration of the IEA, the secant stiffness values of the structural elements are not known. The following approach is suggested. For what concerns the bearing devices, reference to their elastic/initial stiffness is made. For what concerns the piers, instead, a secant stiffness equal to 10 percent of the uncracked section stiffness can be assumed for piers expected to exceed their yield displacement, while reference to the effective elastic stiffness (typically of the order of 30-50 per cent of the uncracked section stiffness) is made for piers that are not expected to exceed their yield displacement. The deformed shape of the bridge (Δ_i) derived from modal analysis is scaled, based on the displacement corresponding to the attainment of a given DS in a (trial) critical element (pier, abutment, bearing) of the bridge. The element that first reaches or exceeds a given target displacement amplitude (see Table 2.1) is recognized as the critical element of the bridge and its displacement is the critical displacement u_{cr}. The displacements of the other elements ($\Delta_{eff,i}$) are obtained scaling the deformed shape in proportion to the ratio between the critical displacement (u_{cr}) and the corresponding value of the displacement pattern (Δ_{cr}). In the next step of the IEA reference is made to the secant stiffness of each element corresponding to the displacements $\Delta_{eff,i}$. The iterative analysis continues till there is no significant change in the critical displacement profile of the bridge ($\Delta_{eff,i}$) between two consecutive steps of analysis. The iterative procedure normally converges in 3-4 iterations.

2.3. Evaluation of the PGA values corresponding to selected Damage States of the bridge

The critical displacement profile of the nonlinear Multi Degree of Freedom (MDOF) model of the bridge, associated to a selected DS, is converted into the critical displacement ($S_{d,DS}$) of an equivalent elastic Single Degree of Freedom (SDOF) system, according to the principles of the DDBD method (Priestley et al., 2007). The corresponding spectral acceleration ($S_{a,DS}$) is then expressed as the ratio between the base shear ($V_{b,DS}$) and the effective mass ($M_{e,DS}$) of the bridge derived from the DDBD method.
The seismic demand associated to each DS is represented by a reference over-damped elastic response spectrum, whose seismic intensity (PGA_{DS}) 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 DS. To this end, the following routine is followed: (i) derive the actual displacement of each structural member, from the critical 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 formula by Grant et al. (2004) has been made to estimate the equivalent viscous damping of piers and bearing devices. The global equivalent viscous damping of the bridge ($\xi_{e,DS}$) is evaluated by weighting the contributions of each structural member as a function of the strain energy of each element at its maximum displacement.
Once the equivalent viscous damping of the entire bridge has been determined, the corresponding demand spectrum can be derived from the normalized 5%-damped reference response spectrum, using a proper damping reduction factor (Cardone et al., 2008). In this study, reference to the damping reduction factor adopted in the Eurocode 8 has been made.
Finally, the PGA value associated to the selected DS is determined as the ratio between the acceleration level ($S_{a,DS}$) corresponding to the selected DS and the normalized spectral acceleration ($S_{a1,DS}$) at the effective period of vibration ($T_{e,DS} = 2\pi(S_{d,DS}/S_{a,DS})^{1/2}$) and global equivalent viscous damping ($\xi_{e,DS}$) of the structure.

2.4. Fragility curves and seismic risk

The PGA values thus obtained represent an estimate of the median threshold values of the PGA related to the selected DSs. They can be used to derive a number of fragility curves, which provide the probability of exceedance of the selected DSs, as a function of the seismic intensity of the expected ground motions. In the proposed procedures, the fragility curves are expressed by a lognormal cumulative probability function. According to previous studies (Dutta and Mander, 1998, Basöz and Mander, 1999, Kappos and Paraskeva, 2008, Paraskeva and Kappos, 2010), a value of the lognormal standard deviation (βc), which takes into account the uncertainties related to input ground motion, bridge response, material characteristics, etc., equal to 0.6 can be assumed for existing RC bridges. The final step of the proposed procedures is the evaluation of the seismic risk associated to the selected DS, with the use of hazard maps, which provide the PGA values at the bridge site having a given probability of exceedance (e.g. 10%) in a given interval of time (e.g. 50 years). In the proposed procedures, a seismic risk index is computed as convolution integral of the product between the seismic vulnerability of the bridge, expressed by the fragility curves, and the seismic hazard of the bridge site, expressed by the hazard curve at the bridge site. The risk index thus obtained provides the probability of exceedance of the selected DS, conditioned to the hazard of the bridge site.

3. APPLICATION OF THE PROPOSED DDBA PROCEDURES TO A CASE STUDY

The proposed DDBA procedures have been applied to a case study represented by a typical Greek bridge. A very common class of bridges in Greece is that of multi-span bridges with continuous deck, realised with precast prestressed beams connected through cast in-situ R/C slabs, simply supported through bearings on single shaft piers with either rectangular or circular hollow section (Moschonas et al., 2009) (see Figure 1).

The selected case study 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%. The width of the expansion joints between deck and abutment back-wall is taken equal to 100 mm. Concrete of class B35 has been assumed for the prestressed beams of the deck, while concrete of class B25 has been assumed for piers, abutments and foundations. Steel of class BSt 500/550 and BSt 1700/1900 has been assumed for rebars and prestressed cables, respectively. The connection between deck beams and piers is realised by four unbolted neoprene pads, whose geometric and mechanical characteristics are summarized in Table 3.1.

Two different bridge configurations have been considered in order to fully assess the accuracy of the proposed DDBA procedures, i.e.: (i) a continuous deck bridge and (ii) a bridge with four independent decks with internal joints of 120 mm width. The schematic layout of examined bridge configurations is shown in Figure 2.

Piers exhibit a flexural behaviour (no effects due to shear). The bilinear skeleton curves of the piers have been derived from elastic-plastic pushover analysis. The effective elastic stiffness is equal to 9929 kN/m for piers 1 and 3 and 1853 kN/m for pier 2. The elastomeric bearings placed on abutments and short piers feature a sliding failure mechanism with a friction coefficient concrete to rubber (μ_{fr}) of 40% while those placed on the slender pier a roll-over failure mechanism. Elastic shear stiffness and horizontal strength of the bearing devices are summarized in Table 3.1.

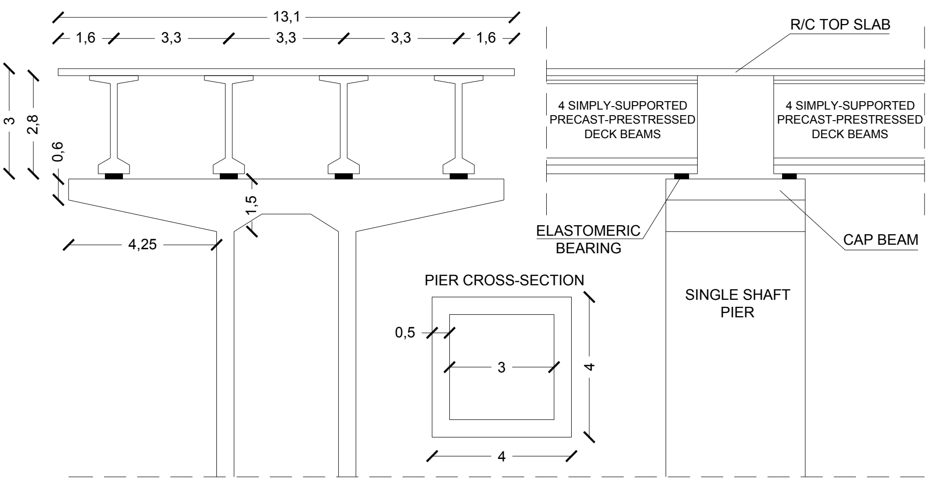

Figure 1. Structural scheme of the selected case study.

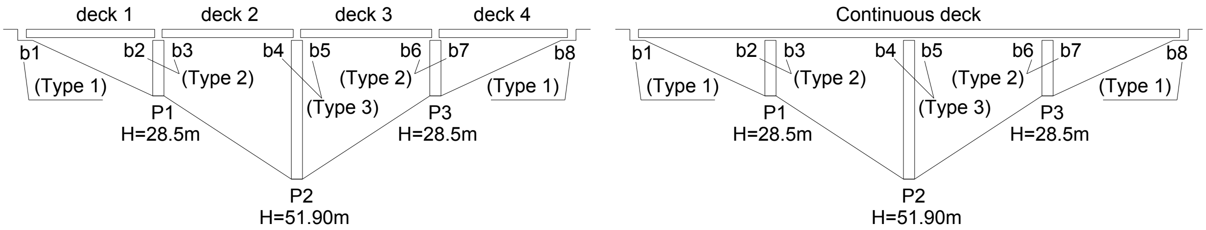

Figure 2. Schematic layout of the examined bridge configurations.

Table 3.1. Geometric and mechanical characteristics of the elastomeric bearings

Type	Shape	Dimensions (m)	Young's and Shear moduli E/G (kN/m^2)	Rubber thickness t_r (mm)	Horizontal stiffness K_h (kN/m)	Horizontal strength F_u (kN)
1	Circular	0.65	600000/900	90	3318	453
2	Rectangular	0.4 x 0.5	600000/900	77	2338	453
3	Rectangular	0.4 x 0.5	600000/900	110	1636	147

Table 3.2 summarizes the main results obtained from the application of the proposed procedures to the selected case study. The results are expressed in terms of PGA values associated to three different DSs, separately in the transverse and longitudinal direction of the bridge with both continuous deck and independent spans. The critical elements of the bridge, where first the selected DSs are reached, are also identified in Table 3.2.

In the longitudinal direction the seismic response of the bridge is mainly influenced by the behaviour of abutments and bearing devices. In the transverse direction, instead, the seismic response of the bridge is governed by the inelastic behaviour of the piers. The two procedures generally predict the same critical element, except for the DS2 and DS3 in the transverse direction of the bridge with independent spans. A good accordance between the PGA values predicted by the two procedures is observed, with maximum differences that do not exceed 15% and on average result of the order of 5%.

The fragility curves associated to the selected DSs are shown in Figure 3. As can be seen, for the continuous bridge an apparent weak direction cannot be identified. For the bridge with independent spans, on the contrary, the vulnerability seems to be greater in the longitudinal direction. In any case, the distance between the fragility curves in the two directions reduces with increasing the DS level, due to the activation of the abutment-backfill system and the major role played by the bearings in the longitudinal direction.

Table 3.2. Critical element and PGA values provided by the proposed DDBA procedures for the selected DSs.

INDEPENDENT SPANS		DS1		DS2		DS3	
		DDBA1	DDBA2	DDBA1	DDBA2	DDBA1	DDBA2
TRANSVERSE	Critical Element	P1/P3	P1/P3	P1/P3	B2/B7	P1/P3	B2/B7
	PGA_{DS} (g)	0.79	0.79	0.94	0.98	1.15	1.28
LONGITUDINAL	Critical Element	ABT	ABT	B3/B6	B3/B6	B3/B6	B3/B6
	PGA_{DS} (g)	0.32	0.31	0.68	0.70	1.09	0.93

CONTINUOUS		DS1		DS2		DS3	
		DDBA1	DDBA2	DDBA1	DDBA2	DDBA1	DDBA2
TRANSVERSE	Critical Element	B1/B8	B1/B8	B1/B8	B1/B8	P1	P1
	PGA_{DS} (g)	0.34	0.32	0.60	0.57	1.04	0.99
LONGITUDINAL	Critical Element	ABT	ABT	ABT	ABT	ABT	ABT
	PGA_{DS} (g)	0.19	0.18	0.65	0.71	1.00	1.18

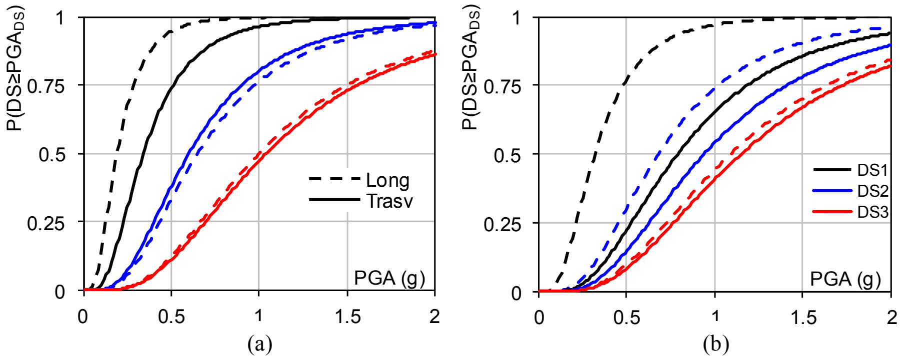

Figure 3. Fragility curves of the selected bridge with (a) continuous deck and (b) independent spans.

4. COMPARISON WITH NONLINEAR TIME HISTORY ANALYSIS RESULTS

The accuracy of the proposed DDBA procedures has been verified trough a series Non-linear response Time-History Analyses (NTHA). An accurate three-dimensional numerical model of the bridges has been implemented using the MIDAS-Civil software package. The deck(s) have been modelled using linear frame and shell elements. In particular, rectangular shell elements with drilling DOFs have been used for the RC top slab. The mass of the deck(s) has been lumped in the nodes of the shell elements. The longitudinal and transverse beams of the deck(s) have been modelled with linear frame elements, assuming a proper offset of the centroid with respect to the centre of the slab, to reproduce the effective stiffness and actual dimensions of the deck cross section of the bridge.
Piers have been modelled with the nonlinear force-based fiber elements with distributed plasticity. The pier cross section has been divided in 392 fibers of concrete and 144 fibers of steel, one for each longitudinal rebar. A proper number of Gauss-Lobatto integration points have been selected for the pier element as a function of the pier height. The stress-strain behaviour of concrete has been modelled with the law proposed by Mander (1988) for unconfined concrete. Reference to the model by Giuffré-Menegotto-Pinto (1973) has been made for steel rebars. The mass of the pier has been lumped in the nodes of the fiber elements.
The behaviour of the abutments in the longitudinal direction has been captured with a series of compression-only nonlinear links, characterized by an elastic-plastic skeleton curve with initial gap equal to the width of the joint and a non-dissipative cinematic cyclic behaviour. Possible effects due to closure of internal joints have been taken into account in the analysis of the bridge with independent decks, by means of a series of compression-only link elements with initial gap equal to the width of the joint and a stiff behaviour after closure.
Elastomeric bearings have been modelled with multidirectional nonlinear link elements. An elastic-perfectly-plastic cyclic behaviour (K_h = GA/t_r ; F_u = μ_{fr} W) has been assumed for neoprene pads characterized by a sliding failure mechanism, while an equivalent linear elastic cyclic behaviour (K^*_h = $K_h(1\text{-}d_{roll}/d_{pad})$), prior to slipping ($F_u$ = μ_{fr} W), has been assumed for neoprene pads characterized by a roll-over failure mechanism.

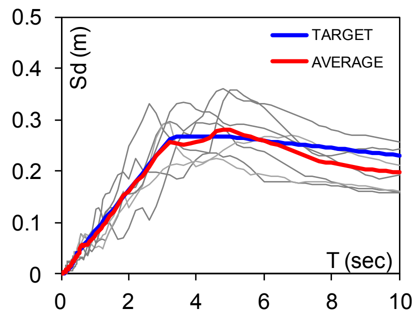

Figure 4. 5%-damped displacement response spectra of the accelerograms used in the NTHA.

Reference to the Rayleigh mass and stiffness proportional damping has been made to account for the effects of the inherent damping of the elastic structure. The NTHA have been performed using a set of seven natural records compatible, on average, with the 475-years return period 5%-damped displacement response spectrum provided by the EC8 for soil type B (see Fig. 4). The input ground motions have been scaled to the PGA values provided by the DDBA procedures for the selected DSs.
The accuracy of the proposed procedure has been evaluated by comparing the expected bridge displacement profiles with the envelope of the maximum bridge displacements (average over 7 accelerograms) obtained from NTHA. The Coefficient of Variation (CV) associated to the NTHA results, expressed as the ratio between the standard deviation (σ) and the absolute average value (μ) of the NTHA displacements, points out a significant dispersion in the NTHA results, due to the variability of the characteristics of the natural records employed in the numerical analysis. Average values of the CV increase while increasing the seismic intensity, ranging from 22% to 32% for the bridge with independent spans and from 20% to 40% for the continuous bridge.
In Figures 5 and 6, DDBA predictions and NTHA results are compared. The comparison is made in terms of deck displacements for the DS2 of the bridge with (a) independent decks and (b) continuous deck, separately in the transverse (Figure 5) and longitudinal direction (Figure 6). As can be seen, the critical displacement profiles provided by the DDBA procedures fall within the 2-sided 99% interval of confidence ($\mu \pm 2.58\sigma/\sqrt{n}$) considered for the NTHA results.
The comparison in the transverse direction points out the accuracy of the proposed DDBA procedures in the prediction of the PGA values associated to the selected DS. Indeed, the percent errors in the evaluation of the NTHA average maximum displacements of the deck(s) do not exceed 18% for the bridge with independent spans and 13% for the continuous deck bridge and, on average, they result of the order of 13% and 11%, respectively. In the longitudinal direction, the DDBA1 turns out to be more accurate than the DDBA2 procedure, with the percent errors that not exceed 8% for the bridge with independent decks and 13% for the continuous deck bridge.
To better measure the accuracy of the proposed DDBA procedures in capturing the ‘exact’ maximum deformed shape of the bridge and the ‘exact’ maximum pier displacement profile, two indices have been computed for each DS. The first index is referred to as Bridge Index (BI). It is defined as:

$$BI = median_{j=1,\ldots,2N_d}\left(\Delta_j = \frac{D_{j,DDBA}}{D_{j,NTHA}}\right) \tag{4.1}$$

where $D_{j,DDBA}$ is the displacement of the j-th deck end provided by the DDBA procedure, $D_{j,NTHA}$ is the corresponding maximum displacement (average on 7 accelerograms) derived from NTHA and N_d is the number of decks.
The second index is referred to as Pier Index (PI). It is defined as:

$$PI = average_{j=1,\ldots,N_p}\left(\mu_j = \frac{\mu_{j,DDBA}}{\mu_{j,NTHA}}\right) \tag{4.2}$$

where $\mu_{j,DDBA}$ is the ductility demand of the j-th pier according to the DDBA procedure, $\mu_{j,NTHA}$ is the corresponding ductility demand (average on 7 accelerograms) derived from NTHA and N_p is the number of piers.
In addition, a third index, referred to as Critical Element Index (CEI), is computed to evaluate the accuracy of the DDBA procedure in capturing the ‘actual’ seismic demand to the critical element of the bridge. The CEI index is defined as:

$$CEI = \left(\frac{\delta_{cr,DDBA}}{\delta_{cr,NTHA}}\right) \tag{4.3}$$

where $\delta_{cr,DDBA}$ is the displacement of the critical element of the bridge predicted by the DDBA procedure and $\delta_{cr,NTHA}$ the corresponding maximum displacement (average on 7 accelerograms) derived from NTHA.

In Table 4.1 the values of BI, PI and CEI relevant to the DDBA1 procedure are reported. It is worth noting that the ideal target value of BI, PI and CEI is always 1. As can be seen, the values of BI range between 0.85 and 1.12, the values of PI between 0.90 and 1.13 and the values of CEI between 0.85 and 1.13. This clearly proves the good accuracy of the proposed procedure for the selected case study.

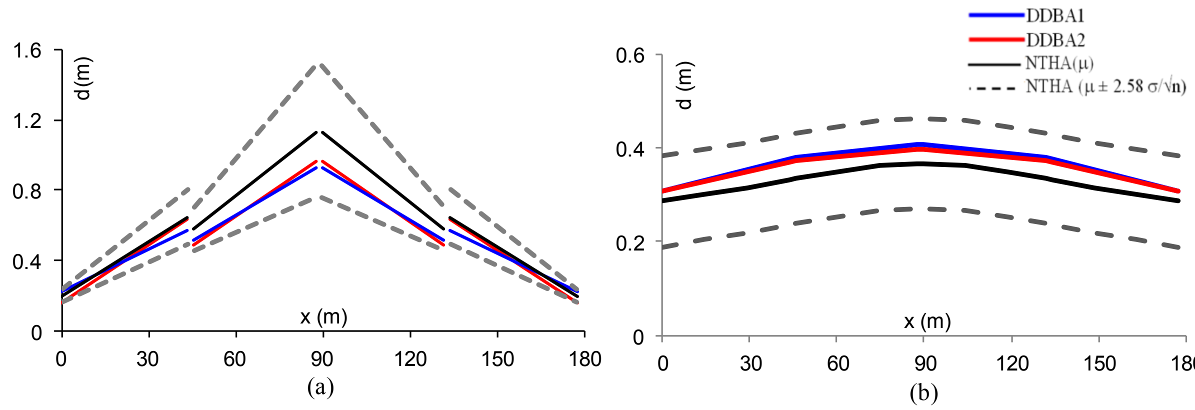

Figure 5. Comparison between DDBA predictions and NTHA results (μ ± 2.58 σ/√n) in terms of maximum deck displacements associated to the DS2 in the transverse direction of the bridge with (a) independents decks and (b) continuous deck.

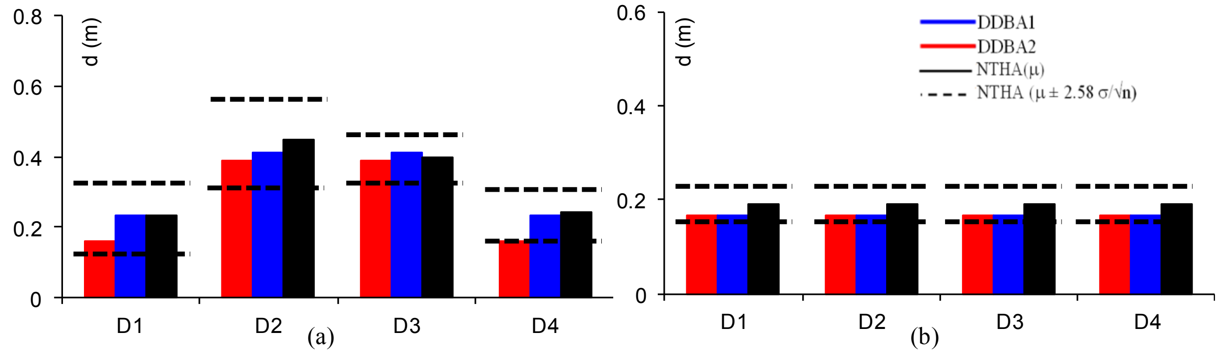

Figure 6. Comparison between DDBA predictions and NTHA results (μ ± 2.58 σ/√n) in terms of maximum deck displacements associated to the DS2 in the longitudinal direction of the bridge with (a) independents decks and (b) continuous deck.

Table 4.1. Bridge Index (BI), Pier Index (PI) and Critical Element Index (CEI) for the DDBA1 procedure.

	INDEPENDENT DECKS						CONTINUOUS DECK					
	TRANSVERSE			LONGITUDINAL			TRANSVERSE			LONGITUDINAL		
	DS1	DS2	DS3	DS1	DS2	DS3	DS1	DS2	DS3	DS1	DS2	DS3
BI	0.85	0.89	0.88	0.99	0.99	0.99	1.02	1.12	1.09	0.93	0.87	0.92
PI	-	0.92	0.90	-	-	-	-	-	1.13	-	-	-
CEI	0.85	0.92	0.90	0.91	0.98	0.91	0.88	1.08	1.13	0.93	0.87	0.92

5. CONCLUSION

Two procedures for the Direct Displacement-Based seismic Assessment (DDBA) of bridges have been recently developed by the authors of this paper. In this paper, they are applied to a case study represented by a typical Greek bridge. The predictions of the DDBA procedures have been compared to the results of Nonlinear response Time-History Analyses (NTHA), carried out on a refined numerical model of the bridge implemented in MIDAS-Civil. The comparison between DDBA predictions and NTHA results confirms the good accuracy of the proposed procedures in predicting the PGA values associated to slight-to-severe Damage States of piers, bearing devices and abutments.

Although the proposed procedures appear very promising, there are a number of aspects that require further investigation. Works are still in progress and additional numerical studies are being to be carried out, considering different bridge configurations, pier layouts and bearing types.

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