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DIRECT DISPLACEMENT-BASED SEISMIC ASSESSMENT OF MULTI-SPAN SIMPLY SUPPORTED DECK BRIDGES

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Abstract. *Traditional seismic assessment is based on a simple comparison of estimated base shear capacity and base shear demand specified by a seismic code. The required code base shear is found by reducing the elastic base shear force corresponding to the elastic stiffness of the structure, by a code-specified force-reduction or behaviour 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 and assessment methods, several researchers have started proposing displacement-based approaches for earthquake engineering evaluation and design, with the aim of providing improved reliability in the engineering process by more directly relating computed response and expected structural performance. The need for accurate seismic assessment methods is particularly important for bridges, due to the crucial role that some bridges (especially highway bridges) have after an earthquake, for allowing the civil protection interventions and first aid organizations. In Italy, many of these "strategic" bridges have been built without antiseismic criteria and, therefore, they need to be assessed against seismic risk. In this extent, the development of an assessment procedure which gives reliable results and, at the same time, is sufficiently simple to be applied on a large population of bridges in a short time is very useful. In this paper a Direct Displacement-Based Seismic Assessment (DDBA) procedure satisfying the aforesaid requirements is proposed. In the paper, the proposed DDBA procedure is applied to a number of bridge configurations derived from a 4-span reinforced concrete simply-supported deck bridge of the A16 Italian highway. Some analyses are also repeated following the traditional force-based seismic assessment approach. Finally, the predictions of the proposed DDBA procedure are compared to the results of nonlinear time history analyses.*

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 [1]. Many bridges are of great importance after an earthquake for guaranteeing civil protection interventions and first aid organizations. As a consequence, they have to be in service immediately after the earthquake. In Italy, as well as in other seismic countries, many bridges have been built before the '80s, without antiseismic criteria. Therefore, 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.

In recognition of the limitations of force-based design and assessment methods, several researchers have started proposing displacement-based approaches for earthquake engineering evaluation and design, with the aim of providing improved reliability in the engineering process by more directly relating computed response and expected structural performance. A rather complete literature review of the subject is reported in Calvi [2], where most displacement – based approaches proposed in the literature, are summarized, critically reviewed and compared, to favour code implementation and practical use of rational and reliable methods. One of the most developed displacement-based methods is the Direct Displacement-Based Design (DDBD) approach proposed by Priestley et al. [3]. A Model Code for the DDBD of structures has been recently published [4], as part of the 2005-2008 RELUIS project.

In this paper, a Direct Displacement-Based seismic Assessment (DDBA) procedure is presented and applied to a number of bridge configurations derived from a multi-span reinforced concrete bridge of the A16 Italian highway. The predictions of the proposed procedure are then compared to the results of nonlinear time-history analysis carried out using a set of seven accelerograms compatible with the Eurocode 8 [5] response spectrum. In addition, a comparison with the traditional force-based approach has been made for a number of selected case studies.

2 DIRECT DISPLACEMENT BASED ASSESSMENT PROCEDURE (DDBA)

The fundamentals of proposed DDBA procedure are derived from previous studies by Petrini et al [6]. In particular, the proposed procedure is based on an Iterative Eigenvalue Analysis (IEA) to obtain the target displacement profile of the bridge, corresponding to a selected Performance Level (i.e. target displacement) of the structure. The Multi Degree of Freedom (MDOF) model of the structure is then converted into an equivalent Single Degree of Freedom (SDOF) system, according to the principles of the DDBD [7], [8]. Hence, the Peak Ground Acceleration (PGA_{PL}) corresponding to the selected Performance Level (PL) is evaluated.

In this paper, the proposed DDBA procedure is specialised to multi-span reinforced concrete simply-supported deck bridges. In particular, the influence of the bearing devices placed between pier/abutment and decks is taken into account. At this stage of the study, the attention is focused on the transverse response of the bridge.

The proposed DDBA procedure can be summarized in three main steps:

(i) Structural modelling, through the acquisition of structural information;

(ii) Derivation of the bridge target displacement profile associated to a selected performance level of the structure, through an Iterative Eigenvalue Analysis (IEA);

(iii) Evaluation of the corresponding PGA value, 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 an overdamped elastic response spectrum.

2.1 Structural Modelling and Damage States

The bridge model has been purposely kept as simple as possible in order to reduce the complexity of the analysis. In accordance with the structural component modelling approach [1], the bridge structure has been divided in a number of independent rigid diaphragms, modelling the bridge decks, mutually connected by means of a series of nonlinear springs, modelling bearing devices and piers (see Fig. 1). Obviously, different modelling strategies can be pursued without influencing the core of the proposed DDBA procedure. Table 1 summarises the basic modelling assumptions adopted in this study for each bridge component, to describe their monotonic and cyclic behaviour, within modal and nonlinear dynamic analysis, respectively.

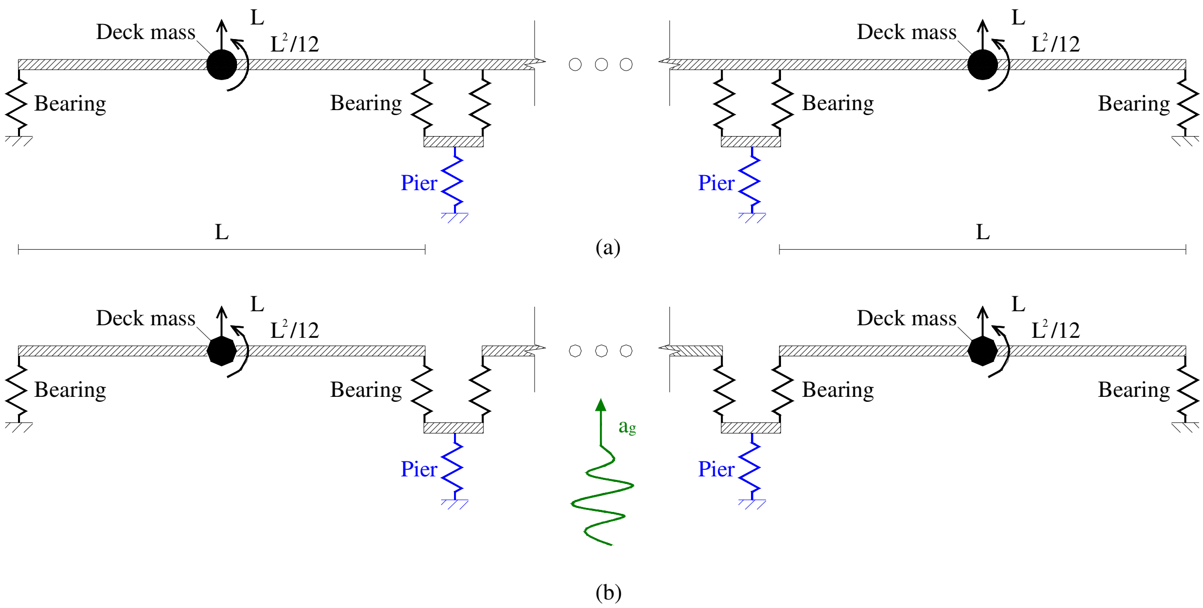


Figure 1: Schematization of multi-span simply supported (a) continuous and (b) isostatic deck bridges.

COMPONENT	MODELLING ASSUMPTIONS	DAMAGE STATES
Foundations/ Abutments	Infinitely rigid and resistant.	-
Deck	Diaphragm behaviour. Lumped translational and rotational masses.	-
Piers	Nonlinear force-displacement behaviour. Flexural behaviour based on moment-curvature analysis. Shear strength, lap-splice effects, buckling of bars and P-Δ effects considered.	Plastic hinge formation. Ultimate rotation capacity. Shear failure.
Bearing devices	Mechanical behaviour described by means of nonlinear force-displacement relationships.	Device failure. Post-failure sliding.

Table 1: Types of bridge components and basic modelling assumptions considered in the procedure.

The translational and rotational mass of each deck have been lumped as shown in Figure 1. Decks and foundations have been considered as infinitely rigid and resistant. Piers have been modelled with nonlinear springs characterized by a bilinear backbone curve. In this study, the lateral force-displacement relationships of the piers have been derived based on preliminary elasto-plastic pushover analyses of the piers, schematized as elastic beams with plastic hinges at the end(s). The plastic hinge behaviour has been derived from a moment-curvature analysis of the critical cross section(s) of the pier, considering the axial force due to gravity loads and the effects of concrete confinement and steel strain-hardening. In this study, reference to the models by Mander et al. [9] and Menegotto-Pinto [10] has been made for confined/cover concrete and steel, respectively. Lap-splices and buckling effects have been also considered in the moment-curvature analysis (see Fig. 2(a)). The moment-curvature relationship thus obtained has been properly bilinearized (see Fig. 2(a)) and then converted in the moment-rotation behaviour of the plastic hinge, whose length has been evaluated according to the formula proposed by Priestley et al. [1]. P-Δ effects due to gravity loads and premature shear failure have been considered in the pushover analysis (see Fig.2(b)). As far as the cyclic behaviour of the piers is concerned, reference to the Takeda degrading-stiffness-hysteretic model has been made [11] (see Fig. 2(c)).

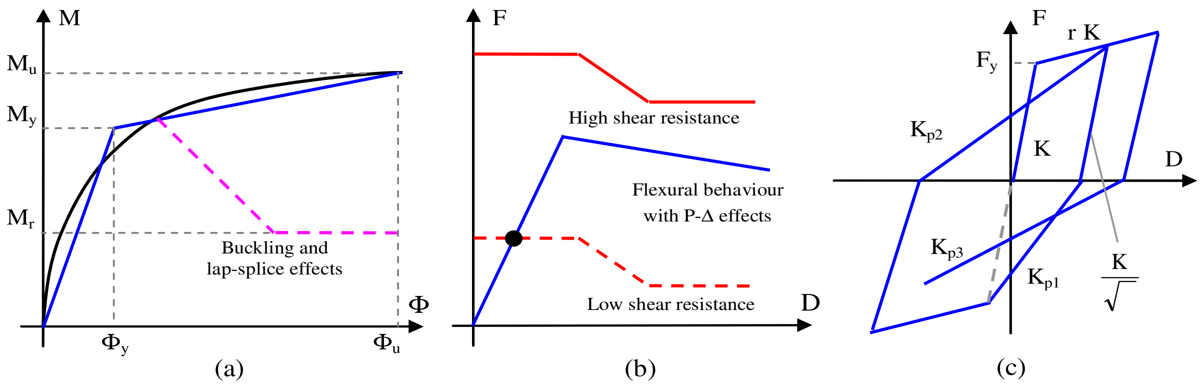
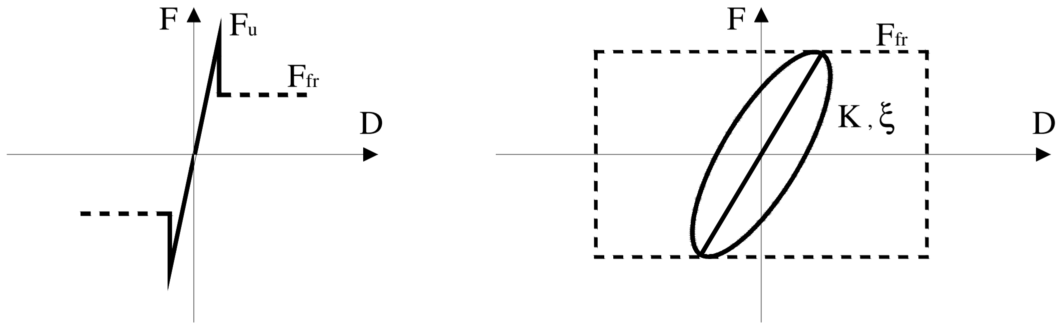


Figure 2: Pier modeling: (a) Moment-curvature analysis of the critical section, (b) force-displacement behaviour taking into account possible shear failure, (c) Takeda degrading-stiffness-hysteretic cyclic model.

The nonlinear behaviour of the bearing devices has been defined based on a comprehensive survey of the Italian highway bridge inventory. In particular, two different types of bearing devices have been considered, i.e.: (i) steel hinges and (ii) neoprene pads.

Steel hinges have been assumed to remain linear elastic up to failure (Fig. 3(a)), which is usually brittle, being due to the attainment of the shear strength (F_u) of the device. The shear stiffness has been estimated based on the geometric details available. During the analysis, the maximum shear force has been monitored. When the shear strength was prematurely exceeded, a post-failure frictional behaviour, corresponding to sliding between deck and pier cap, has been considered (Fig. 3(a)).

A linear visco-elastic behaviour has been considered for neoprene pads (Fig. 3(b)), whose shear stiffness (K) has been evaluated based on the dimensions (cross section area and thickness) of the pads and shear modulus (G) of neoprene. In this study, a shear modulus of 1 MPa and a viscous damping ratio (ξ) of 6% have been assumed for neoprene pads. The horizontal strength of the bearing system has been computed as the lowest between the shear resistance of the neoprene pads and the friction resistance between neoprene and concrete sliding surfaces. The shear resistance of neoprene pads has been associated to the attainment of a shear strain of 150%. The friction coefficient between neoprene and concrete has been taken equal to 70%.



(a) (b)

Figure 3: Force-displacement behaviour of (a) steel hinges, (b) neoprene pads.

A number of Damage States (DSs) have been defined for piers and bearing devices. The DSs have been grouped in three different Performance Levels (PLs) (see Tab. 2), based on the consequences in terms of damage that the attainment of each DS can produce. Obviously, this division is only formal and it is made only for the sake of clarity. The first Performance Level (PL1) includes post-earthquake Damage States in which only very limited structural damage has occurred (e.g. pier yielding, attainment of the horizontal strength in neoprene pads,….). The second Performance Level (PL2) includes post-earthquake Damage States in which significant damage to some structural elements has occurred but large margin against either partial or global collapse still remains (e.g. 50% of the ultimate ductility demand in piers, large post-failure displacements in neoprene pads,….). The third Performance Level (PL3) includes post-earthquake Damage States in which the structure continues to support gravity loads but retains no margin against collapse (e.g. collapse of steel hinges, pier collapse due to the attainment of ultimate ductility or shear strength, …).

PL1 (Slight Damage State)	PL2 (Moderate Damage State)	PL3 (Severe Damage State)
o Pier yielding o Neoprene pad failure	o 50% ultimate ductility of piers o Large post-failure displacements in neoprene pads	o Pier collapse o Steel hinge failure

Table 2: Performance Levels of the structure and Damage States of piers and bearing devices.

2.2 Iterative Eigenvalue Analysis (IEA)

An eigenvalue analysis is initially performed assuming the elastic stiffness of piers and bearings. The resultant mode shape in the transverse direction is recorded as the displacement pattern of the bridge. Next, in order to obtain the target displacement profile, the displacement pattern is scaled, based on the displacement corresponding to the attainment of a given DS in a (trial) critical element (pier or bearing) of the bridge. The element that first reaches a predefined target displacement amplitude in the current displacement pattern is recognized as the critical element of the bridge and its displacement is the critical displacement Δ_{cr}. The displacements of the other piers and bearings are obtained from Δ_{cr} in proportion to the mode shape (ϕ_i):

$$\Delta_i = \frac{\Delta_{cr}\phi_i}{\phi_{cr}} \tag{1}$$

where ϕ_i is the component of the mode shape vector related to the i-th pier or bearing, ϕ_{cr} is the component of the mode shape vector related to the critical pier, Δ_i is the displacement of the i-th structural element.

Next, the secant stiffness of each element corresponding to the displacements Δ_i are found:

$$K_i = \frac{F_i}{\Delta_i} \tag{2}$$

where F_i is the force value corresponding to the pier displacement Δ_i found on each force-displacement curve. New secant stiffness values are used in the next eigenvalue analysis and a new displacement shape is obtained. The iterative procedure continues till there is no significant change in the displacement shape. The iterative procedure normally converges in 3-5 iterations.

At this point, the effect of higher modes must be checked and if needed the bridge displacement shape must be revised. If the mass participation ratio at the end of the IEA process is lower than 65-70% of the total mass, higher mode effects shall be considered. To this end, reference has been made to the so-called Effective Mode Shape (EMS) method [12], in which the displacement of each element (piers and bearings) is calculated with the following equation:

$$\Delta_{i,j} = \phi_{i,j} \cdot PF_j \cdot S_{d,j} \tag{3}$$

where the index i represents the element number and the index j represents the mode number, $\Delta_{i,j}$ is the displacement of the i-th element for the j-th mode, $\phi_{i,j}$ is the modal displacement of i-th element for the j-th mode, PF_j is the participation factor of the j-th mode and $S_{d,j}$ is the spectral displacement for the j-th mode obtained by entering the 5% damped elastic displacement spectrum with the modal period of the j-th mode. The final displacement shape of the bridge, considering the higher mode effects, is calculated by any appropriate modal combinations (SRSS, CQC etc.) of these displacements.

2.3 Evaluation of the PGA associated to given Damage States of the structure

The equivalent SDOF displacement of the bridge (Δ_e) is derived, from the displacement profile obtained at the end of the IEA, through the following equation:

$$\Delta_e = \frac{\sum_j \left(m_j \cdot \Delta_j^2 + \mathrm{I}_j \cdot \delta_j^2 \right)}{\sum_j m_j \cdot \Delta_j} \tag{4}$$

where m_j and Δ_j are the translational mass (μL) and the horizontal displacement of the centre of mass of the j-th deck, respectively, I_j and δ_j are the rotational mass ($\mu L^3/12$) and the rotation around the vertical axis of the j-th deck, respectively. The force level associated to Δ_e is given by the global base shear (V_b) obtained at the end of the IEA.

The equivalent SDOF mass (m_e) is then given by:

$$m_e = \frac{\sum_j m_j \cdot \Delta_j}{\Delta_e} \tag{5}$$

The next step of the procedure is to determine the seismic demand associated to each DS, represented by a reference over-damped elastic response spectrum. This step requires the evaluation of the equivalent viscous damping of the bridge by the combination of the damping contributions of each structural member (piers and bearings). The equivalent damping of the bearing devices can be calculated through the well-known Jacobsen approach [13]:

$$\xi_{b,j} = \frac{E_{visc} + E_{hyst} + E_{fr}}{2\pi \cdot F_{b,j} \cdot \Delta_{b,j}} \tag{6}$$

in which E_{visc}, E_{hyst} and E_{fr} identify the energy dissipated by the device, through its viscous, hysteretic or frictional behaviour, in a cycle of amplitude $\Delta_{b,j}$, being $\Delta_{b,j}$ the displacement of the device and $F_{b,j}$ the corresponding force level.

As far as piers are concerned, reference has been made to the following relationship:

$$\xi_{p,k} = 0.05 + \frac{1}{\pi}\left(1 - \frac{(1-r)}{\sqrt{\ }} - r \quad \right) \tag{7}$$

which relates the equivalent hysteretic damping of the pier to its displacement ductility (μ) and post-yield hardening ratio (r). The aforesaid relationship has been derived by Kowalski et al. [14], by applying the Jacobsen's approach to the Takeda degrading-stiffness-hysteretic model.

The global equivalent damping of the bridge ξ_e is calculated by weighting the damping contributions based on the energy dissipated by each structural element:

$$\xi_e = \frac{\sum_{j=1}^{nb} \xi_{b,j} F_{b,j} \Delta_{b,j} + \sum_{k=1}^{np} \xi_{p,k} F_{p,k} \Delta_{p,k}}{\sum_{j=1}^{nb} F_{b,j} \Delta_{b,j} + \sum_{k=1}^{np} F_{p,k} \Delta_{p,k}} \tag{8}$$

Once the equivalent damping of the bridge has been determined, the corresponding demand spectrum can be derived from the reference 5%-damping normalized response spectrum, using a proper damping reduction factor [15].

The final step of the procedure is to determine the PGA value associated to the selected DS.

From a graphical point of view, the PGA associated to a selected Performance Point (PP) can be evaluated through a translation of the over-damped Normalized Response Spectrum (NRS in Fig.4) to intercept the selected PP, whose coordinates are the equivalent SDOF displacement Δ_e and the spectral acceleration:

$$S_e = \frac{V_b}{m_e} \tag{9}$$

where V_b is the total base shear of the structure corresponding to the deformed shape resulting from IEA

From an analytical point of view, the PGA associated to a selected Performance Point (PP) can be determined as the ratio between the spectral acceleration S_e corresponding to the selected PP and the normalized spectral acceleration at the effective period of vibration (T_e) and global equivalent damping (ξ_e) of the structure:

$$PGA = \frac{S_e}{S_{e1}(T_e, \xi_e)} \tag{10}$$

being:

$$T_e = 2\pi \cdot \sqrt{\frac{m_e}{K_e}} \tag{11}$$

and K_e is the equivalent SDOF stiffness given by:

$$K_e = \frac{V_b}{\Delta_e} \tag{12}$$

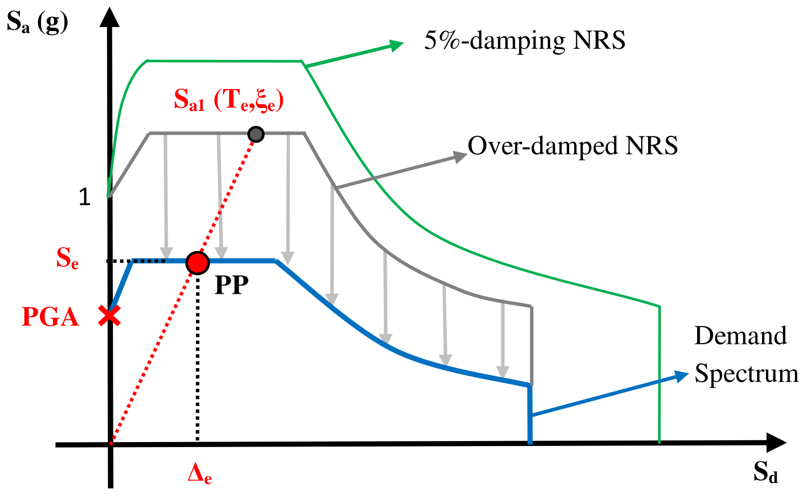


Figure 4: Evaluation of the PGA associated to a selected PP.

3 COMPARISON WITH NTHA RESULTS

3.1 Case Studies

The proposed DDBA procedure has been applied to a number of bridge configurations derived from a Reinforced Concrete (RC) bridge of the A16 Italian highway (Fiumarella viaduct). The Fiumarella viaduct is a 4-span simply-supported isostatic deck bridge of 135m total length, featuring two seat-type abutments and three identical frame-type piers characterised by five RC columns with 1.2m diameter circular cross section and 3.2m effective height. The longitudinal reinforcement ratio of each column is equal to 0.44%. The transverse reinforcement is realised by 10mm diameter hoops at 20mm spacing. The bridge presents steel bearings acting as a pendulum in the longitudinal direction and as a fixed steel hinge in the transversal direction. The yield strength of the reinforcing steel (type AQ50-60) is equal to 270 N/mm2. As far as concrete is concerned, a compression strength of 35 N/mm2 has been assumed, based on the design data available.

A set of nine bridge configurations have been derived from the real bridge configuration (F1 in Tab. 3). They are summarized in Table 3. The schematic layout of the ten bridge configurations is shown in Fig. 5. In the bridge configuration F2 neoprene pads have been considered in place of steel hinges. The stiffness of each line of neoprene pads has been assumed equal to 35000 KN/m, based on a comprehensive survey of similar Italian highway viaducts. The configurations from F3 to F10 have been derived from the configurations F1 and F2 by changing the pier layout (pier heights equal to H-2H-3H or H-3H-2H, with H = 3.20 m) and considering both isostatic and continuous decks.

BRIDGE CONFIGURATION	DECK	PIER LAYOUT (*)	BEARINGS
F1	Isostatic	H-H-H	Steel Hinges (SH)
F2	Isostatic	H-H-H	Neoprene (N)
F3	Isostatic	H-2H-3H	Steel Hinges (SH)
F4	Isostatic	H-2H-3H	Neoprene (N)
F5	Continuous	H-2H-3H	Steel Hinges (SH)
F6	Continuous	H-2H-3H	Neoprene (N)
F7	Isostatic	H-3H-2H	Steel Hinges (SH)
F8	Isostatic	H-3H-2H	Neoprene (N)
F9	Continuous	H-3H-2H	Steel Hinges (SH)
F10	Continuous	H-3H-2H	Neoprene (N)

* H=3.20 m

Table 3: Main characteristics of the analyzed bridges

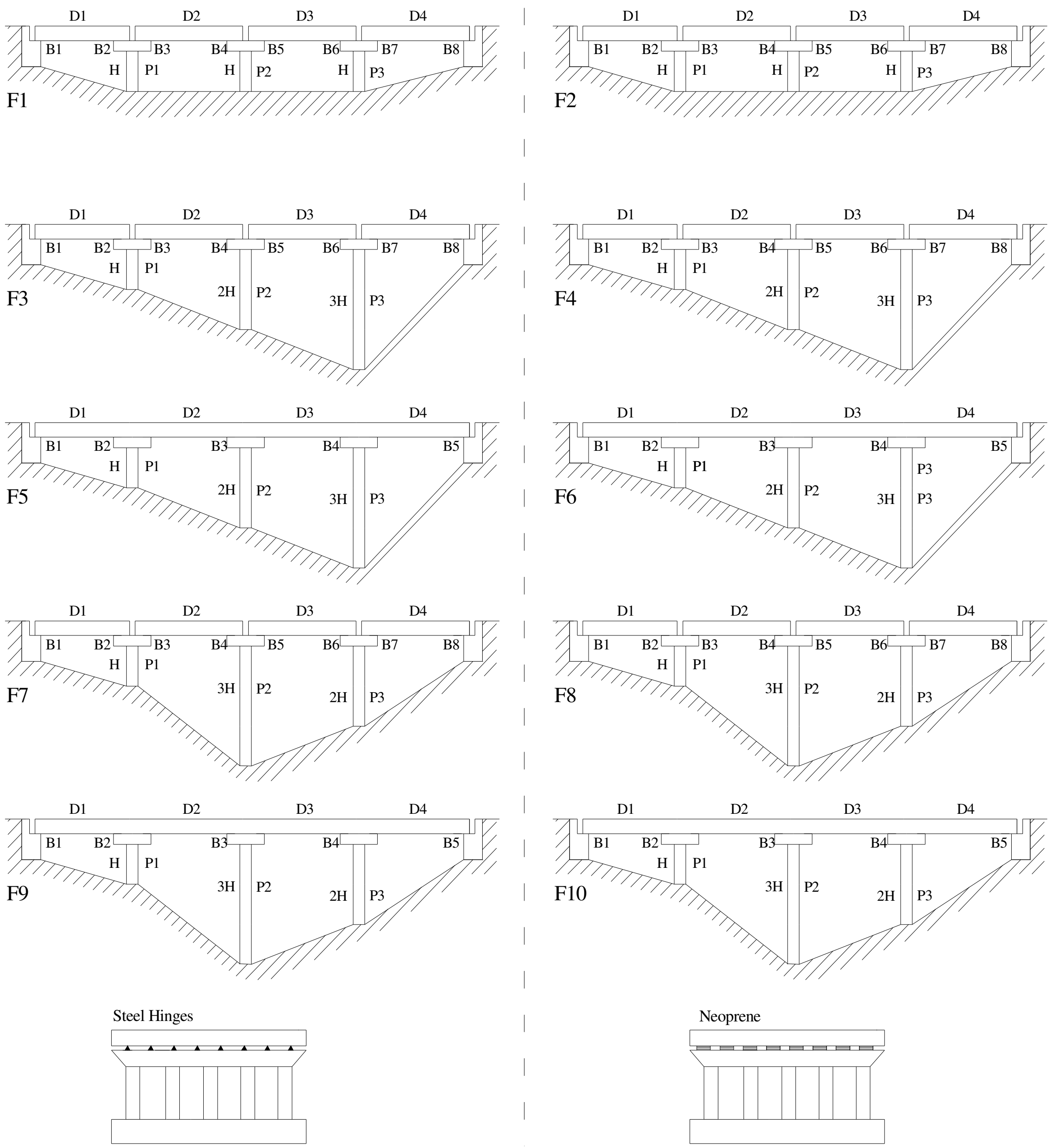


Figure 5: Schematic Layouts of the examined bridges.

3.2 Application of the DDBA procedure

Figure 6 shows the displacement profiles obtained at each step of the IEA for the bridge configurations with irregular pier layouts. As can be seen, the displacement profile considerably changes considering the inelastic behaviour of the bridge elements, since plastic deformations are mainly concentrated in a few elements, due to the irregular bridge configuration. The most significant changes, however, take place in the first 2-3 steps of the analysis. The comparison between the initial and final displacement profile emphasizes the differences between the traditional force-based approach, based on the elastic response of the structure reduced by a proper behaviour factor, and the proposed DDBA procedure, based on the use of an inelastic displacement profile scaled at a given target displacement amplitude.

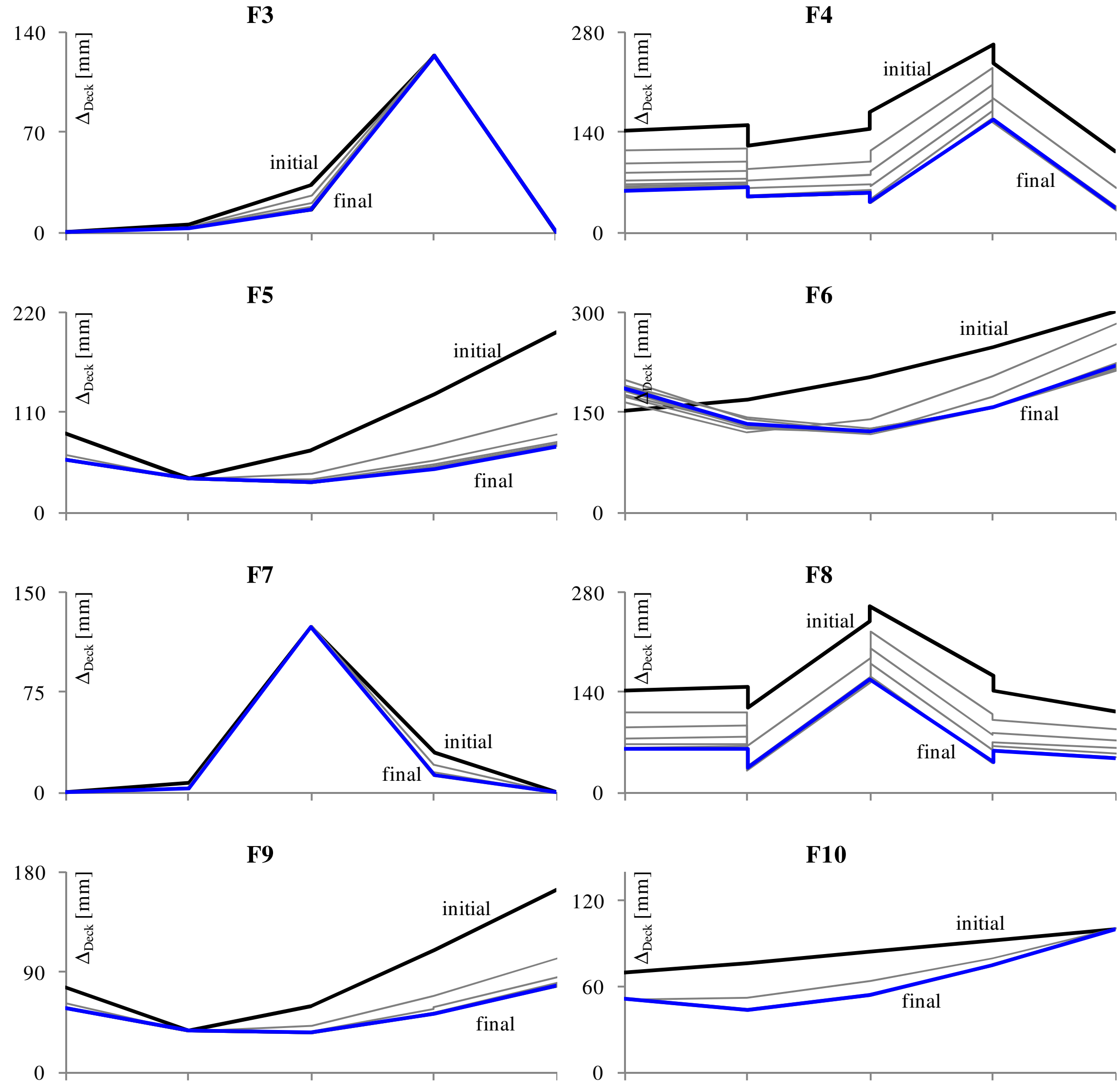


Figure 6: Displacements profiles of the bridge configurations with irregular pier layouts at each step of the IEA.

Table 4 summarizes the results obtained from the application of the DDBA procedure to the case studies described in the paragraph 3.1. The results are expressed in terms of PGA values associated to a number of PLs, corresponding either to pier collapse or failure of neoprene pads. The critical element of the bridge, where first the selected damage state is reached, is also specified in Tab. 4. The PGA values listed in Tab. 4 have been used to scale the accelerograms for nonlinear time-history analysis.

BRIDGE CONFIGURATION	DECK	PIER LAYOUT*	BEARINGS	PERFORMANCE LEVELS	PGA VALUES
F1*	Isostatic	H-H-H	SH	PL3: collapse of P2	0.565 g
F2	Isostatic	H-H-H	N	PL1: elastic limit of B4 PL2: Large displacement on B4	0.206 g 0.538 g
F3	Isostatic	H-2H-3H	SH	PL3: collapse of P3	0.447 g
F4	Isostatic	H-2H-3H	N	PL3: collapse of P3	0.442 g
F5	Continuous	H-2H-3H	SH	PL3: collapse of P1	0.491 g
F6	Continuous	H-2H-3H	N	PL3: collapse of P1	0.626 g
F7	Isostatic	H-3H-2H	SH	PL3: collapse of P2	0.459 g
F8	Isostatic	H-3H-2H	N	PL3: collapse of P2	0.415 g
F9	Continuous	H-3H-2H	SH	PL3: collapse of P1	0.472 g
F10	Continuous	H-3H-2H	N	PL2: Large displacement on B4	0.405 g

* H=3.20 m

Table 4: Performance levels, damage states and corresponding PGA values for the examined bridges.

3.3 Comparison with NTHA results

Comprehensive nonlinear response Time-History Analyses (NTHA) have been carried out to assess the accuracy of the proposed DDBA procedure. The bridge models for NTHA have been implemented in SAP2000_Nonlinear [16], adopting the same modelling assumptions made in the IEA within the proposed DDBA procedure. The NTHA have been performed using a set of 7 artificial accelerograms strictly compatible, on average, with the 5%-damped acceleration response spectrum provided by Eurocode 8 for soil type B [5]. The input ground motions have been scaled to the PGA values provided by DDBA for the calculated PLs (see Tab. 4). A total of 77 NTHA (10 bridges * 7 accelerograms * 1-2 damage states) have been carried out. The accuracy of the proposed procedure has been evaluated by comparing the bridge displacement profile expected based on DDBA with the envelope of the maximum bridge displacements (averaged on 7 accelerograms) obtained from NTHA.

A preliminary comparison between DDBA predictions and NTHA results has been made considering an elastic performance level of the structure (corresponding to the shear failure of neoprene pads at 0.206g for the bridge configuration F2 of Tab. 4), in order to select the best modal combination rule between SRSS and CQC. The comparison is shown in Figure 7. As can be seen, the CQC combination rule leads to errors less than 10% while the SRSS combination rule leads to errors up to 30%.

In Figure 8, DDBA predictions and NTHA results are compared for the PL3 of the bridge configuration F1 (regular pier layout and isostatic decks supported by steel hinges in the transverse direction), representing the real Fiumarella viaduct. Based on the DDBA outcomes, the critical pier, where collapse first takes place, is the pier P2 at 0.565g PGA. Piers P1 and P3, on the contrary, undergo negligible plastic deformations. The comparison between expected and 'actual' deformed shapes clearly points out the great accuracy of the DDBA in the prediction of the PGA values associated to severe damage states. Indeed, the percent errors in the evaluation of the 'actual' maximum deck displacements do not exceed 14% and, on average, they result of the order of 9%. The NTHA results also confirm that pier P2 is the critical element of the bridge with ductility demands that differ from those derived from DDBA less than 8%. It is worth to observe that in Figures 7-17 displacement profiles and bridge coordinates are reported in two different scales. From a graphical point of view, this determines a distortion of the deformed shape of the bridge that considerably amplifies the rotations of the decks.

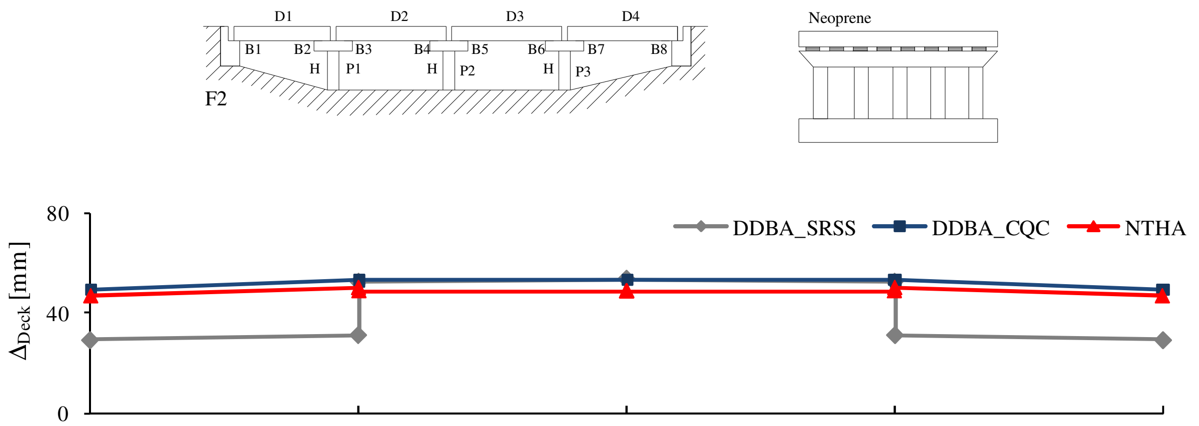


Figure 7: Comparison between NTHA results and DDBA predictions using the CQC and SRSS combination rule.

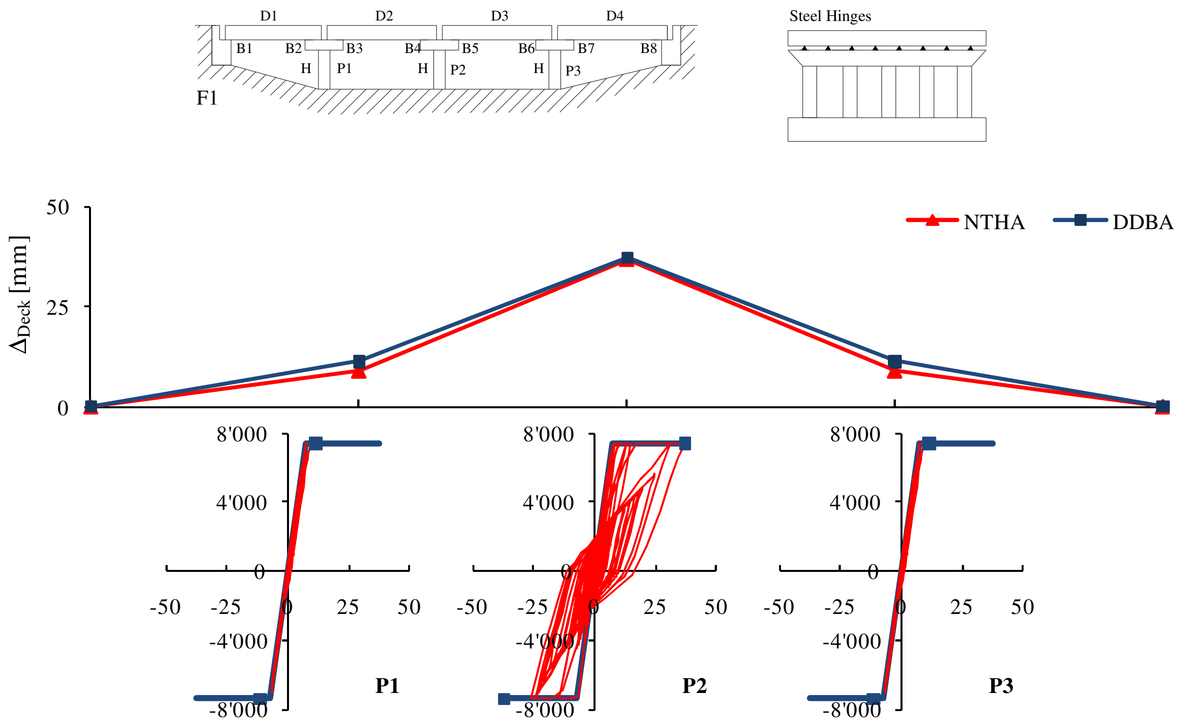


Figure 8: Comparison between DDBA predictions and NTHA results (average maximum values) in terms of deformed shape of the bridge and cyclic behaviour of some structural elements (F1 bridge configuration at PL3, PGA =0.565 g).

Similar observations can be made examining the seismic response of the bridge configuration F2 (see Fig.9), differing from the bridge configuration F1 for the pier-deck connections (neoprene pads instead of steel hinges). A moderate damage state (PL2), corresponding to a post-failure displacement of 100 mm in the bearing devices B4 and B5, occur at 0.538g. No plastic deformations are registered in the piers. The percent errors between DDBA predictions and NTHA results do not exceed 14% and, on average, result of the order of 9%.

The same level of accuracy in the prediction of the 'actual' maximum deformed shape of the bridge is found considering irregular pier layout and multi-span simply-supported continuous deck (Figs. 9-17). As a matter of fact, indeed, the percent errors between DDBA predictions and NTHA results never exceed 20% and, on average, result lower than 15%. Also the critical element of the bridge is always captured correctly.

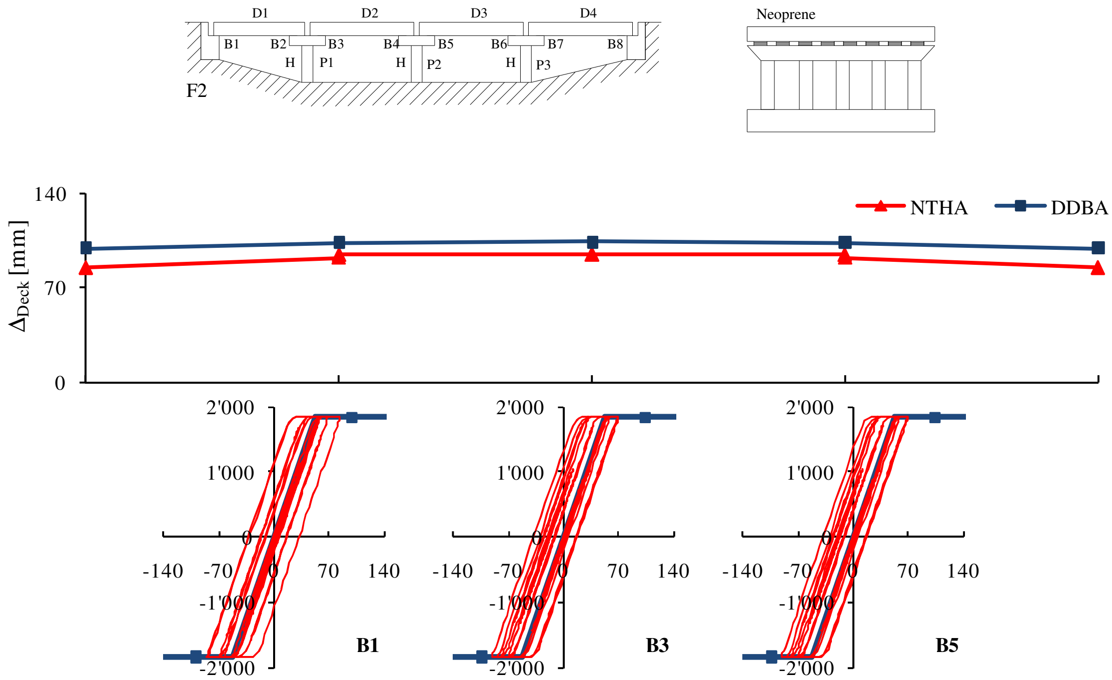


Figure 9: Comparison between DDBA predictions and NTHA results (average maximum values) in terms of deformed shape of the bridge and cyclic behaviour of some structural elements (F2 bridge configuration at PL2, PGA =0.538g).

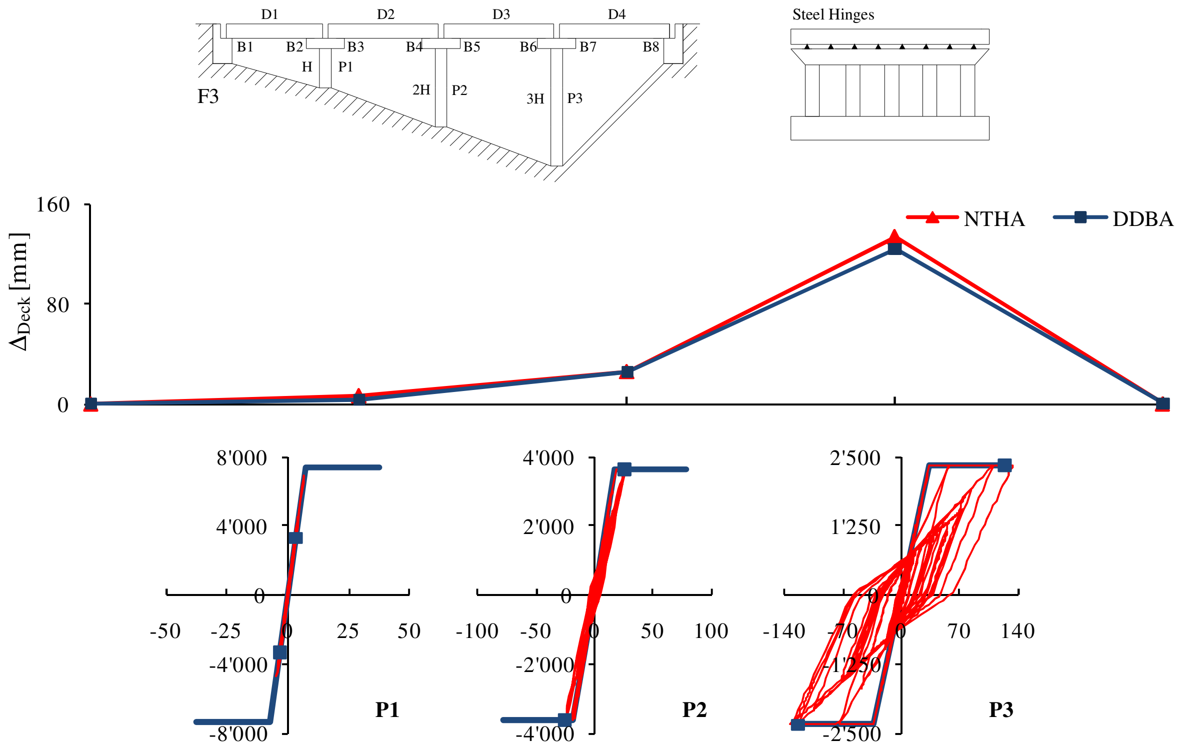


Figure 10: Comparison between DDBA predictions and NTHA results (average maximum values) in terms of deformed shape of the bridge and cyclic behaviour of some structural elements (F3 bridge configuration at PL3, PGA =0.447 g).

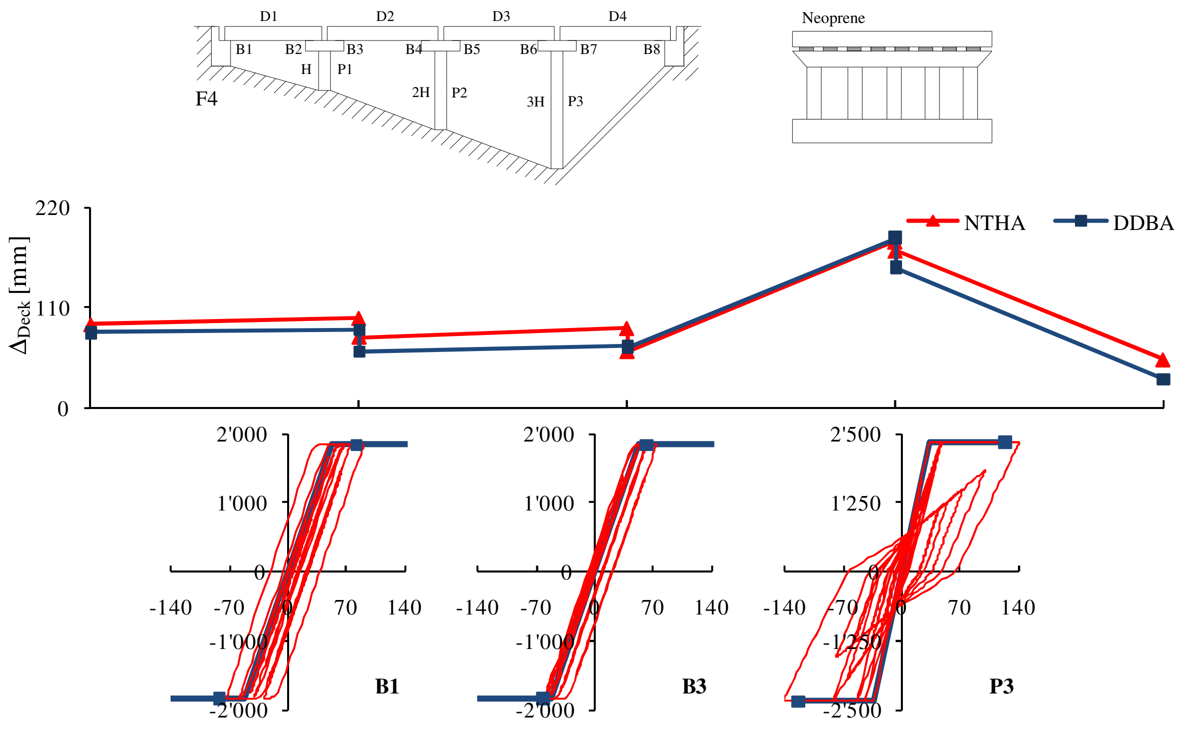


Figure 11: Comparison between DDBA predictions and NTHA results (average maximum values) in terms of deformed shape of the bridge and cyclic behaviour of some structural elements (F4 bridge configuration at PL3, PGA =0.442 g).

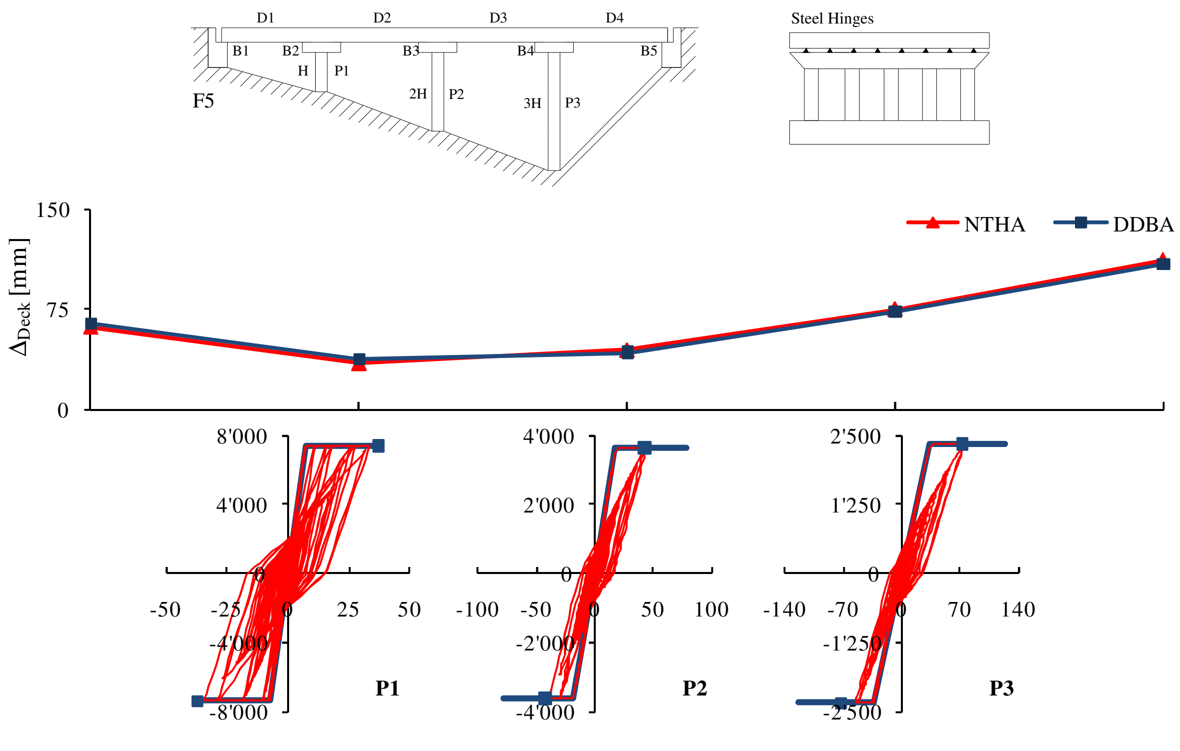


Figure 12: Comparison between DDBA predictions and NTHA results (average maximum values) in terms of deformed shape of the bridge and cyclic behaviour of some structural elements (F5 bridge configuration at PL3, PGA =0.491 g).

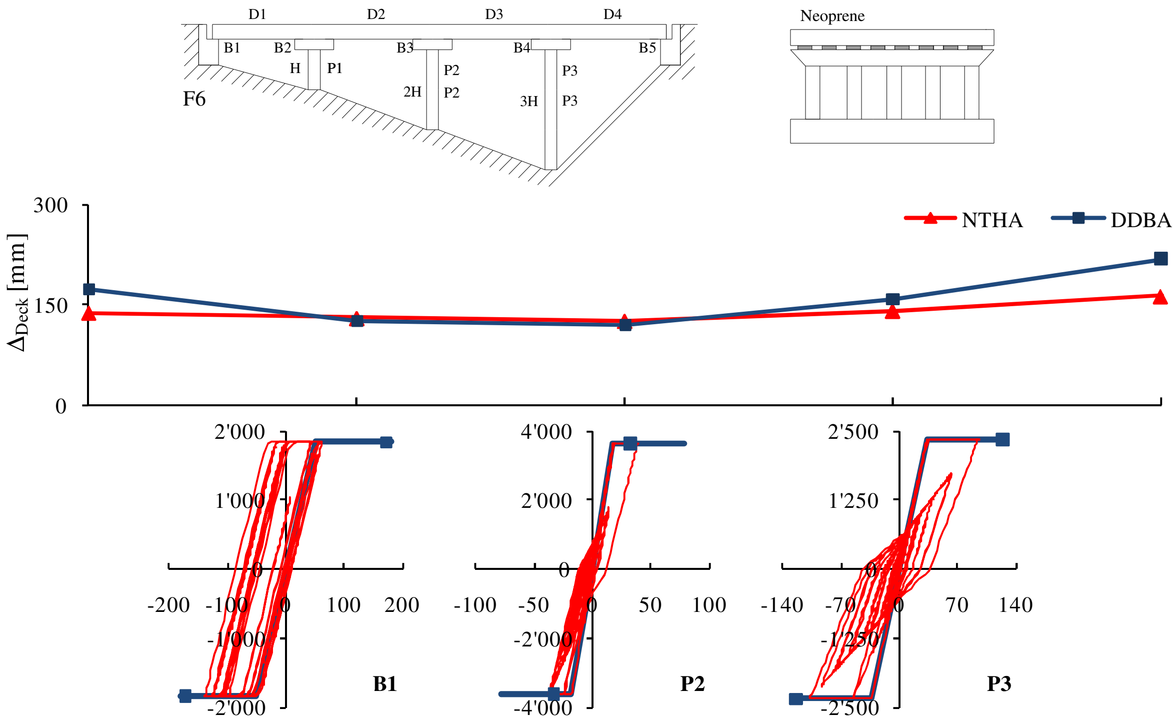


Figure 13: Comparison between DDBA predictions and NTHA results (average maximum values) in terms of deformed shape of the bridge and cyclic behaviour of some structural elements (F6 bridge configuration at PL3, PGA =0.626 g).

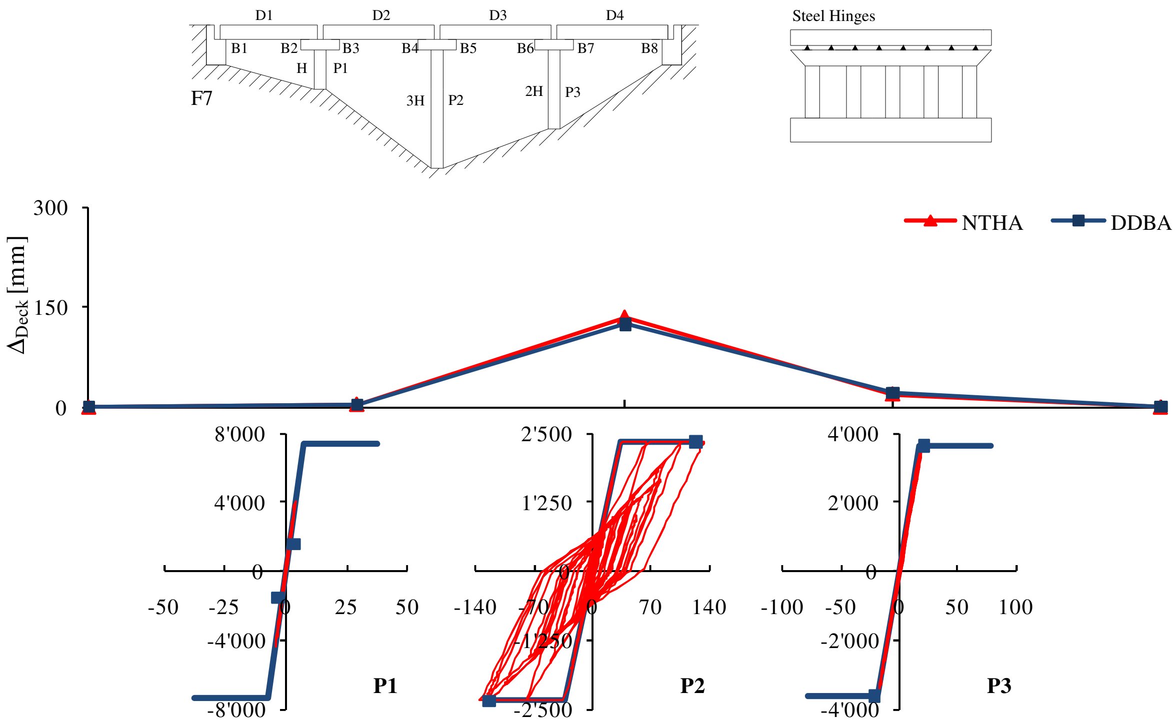


Figure 14: Comparison between DDBA predictions and NTHA results (average maximum values) in terms of deformed shape of the bridge and cyclic behaviour of some structural elements (F7 bridge configuration at PL3, PGA =0.459 g).

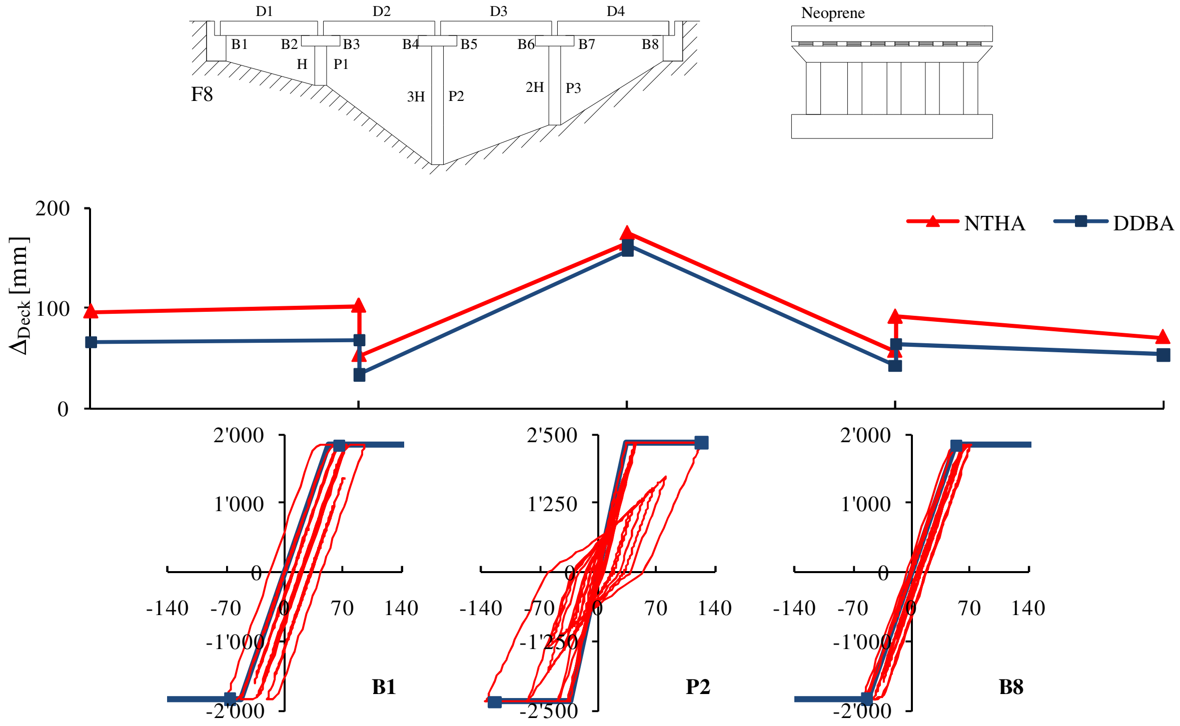


Figure 15: Comparison between DDBA predictions and NTHA results (average maximum values) in terms of deformed shape of the bridge and cyclic behaviour of some structural elements (F8 bridge configuration at PL3, PGA =0.415 g).

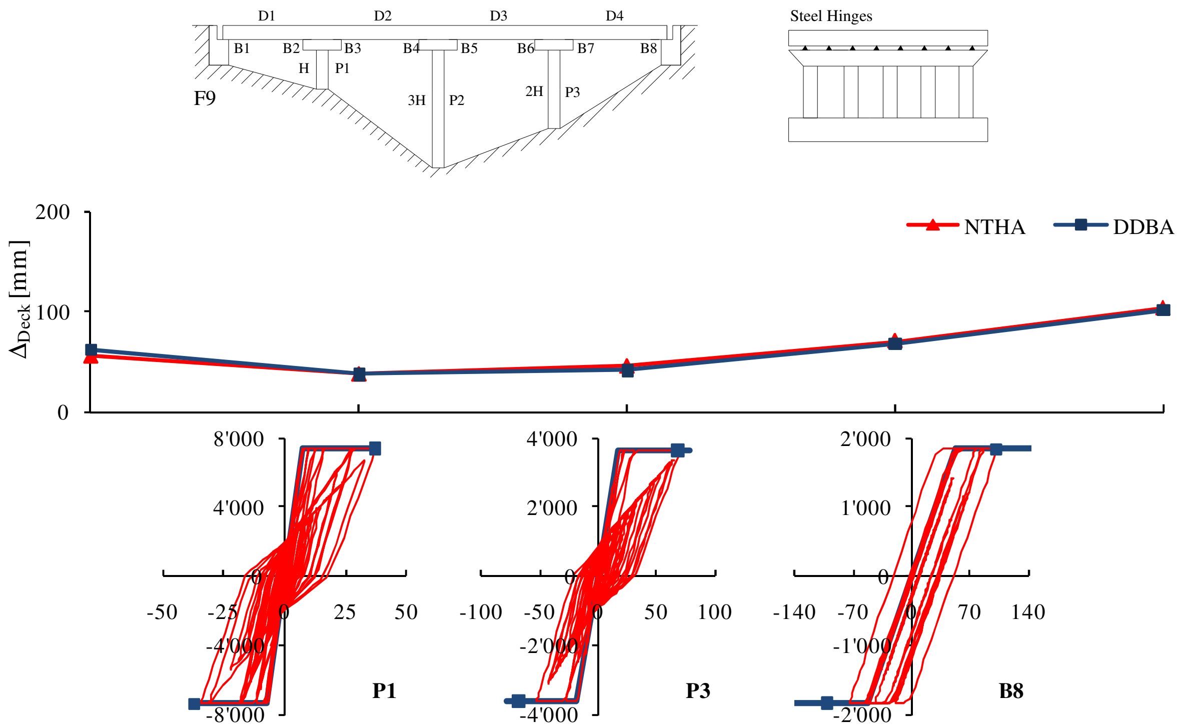


Figure 16: Comparison between DDBA predictions and NTHA results (average maximum values) in terms of deformed shape of the bridge and cyclic behaviour of some structural elements (F9 bridge configuration at PL3, PGA =0.472 g).

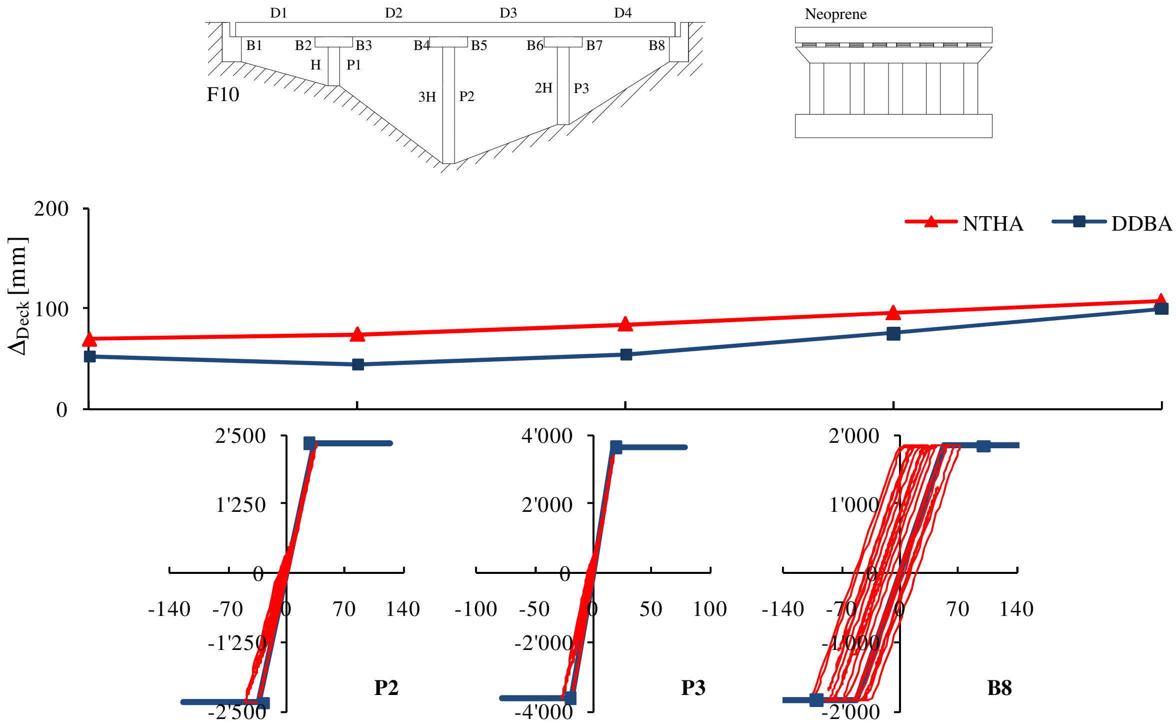


Figure 17: Comparison between DDBA predictions and NTHA results (average maximum values) in terms of deformed shape of the bridge and cyclic behaviour of some structural elements (F10 bridge configuration at PL2, PGA =0.405 g).

For more clarity, in Table 5 the average errors (ERR_{av}) between DDBA and NTHA results, computed based on the deformed shape of the entire bridge, and the error relevant to the critical element of the bridge (ERR_{cr}) are summarised for the ten bridge configurations considered.

BRIDGE CONFIGURATION	DECK	PIER LAYOUT (*)	BEARINGS	PL	PGA	ERR_{av}	ERR_{cr}
F1	Isostatic	H-H-H	SH	PL3-P2	0.565 g	5.01%	1.44%
F2	Isostatic	H-H-H	N	PL1-B4 PL2-B4	0.206 g 0.538 g	5.79% 8.64%	9.86% 9.69%
F3	Isostatic	H-2H-3H	SH	PL3-P3	0.447 g	0.72%	7.19%
F4	Isostatic	H-2H-3H	N	PL3-P1	0.442 g	15.71%	12.81%
F5	Continuous	H-2H-3H	SH	PL3-P1	0.491 g	1.97%	8.02%
F6	Continuous	H-2H-3H	N	PL3-P3	0.626 g	11.97%	15.59%
F7	Isostatic	H-3H-2H	SH	PL3-P2	0.459 g	1.21%	8.05%
F8	Isostatic	H-3H-2H	N	PL3-P2	0.415 g	21.38%	8.16%
F9	Continuous	H-3H-2H	SH	PL3-P1	0.472g	2.32%	0.61%
F10	Continuous	H-3H-2H	N	PL2-B8	0.405g	27.06%	6.81%

* H=3.20 m

Table 5: Comparison between DDBA and NTHA results: average error and error relevant to the critical element.

In Figure 18 the results derived from DDBA are compared to those abtained applying the traditional Force-Based seismic Assessment (FBA) approach, consisting in a Response Spectrum analysis, considering three different behaviour factors, equal to 1 (elastic behaviour), 1.5 and 3.5, respectively. The comparison is made for two different bridge configurations (F4 and F6 respectively) in terms of base shear of each pier and abutment. As can be seen, the accuracy of the FBA approach is very sensible to the definition of an appropriate behaviour factor and, in the investigated cases, is rather low. Assuming q = 1.5, for instance, the FBA approach captures with good accuracy the maximum base shear of the piers P1 and P2 while it considerably overestimate (by 90%) the maximum base shear of the pier P3. On the contrary, assuming q = 3.5 the FBA approach captures with good accuracy the maximum base shear of the pier P3 while it largely underestimate (by 40%) the maximum base shear of the piers P1 and P2.

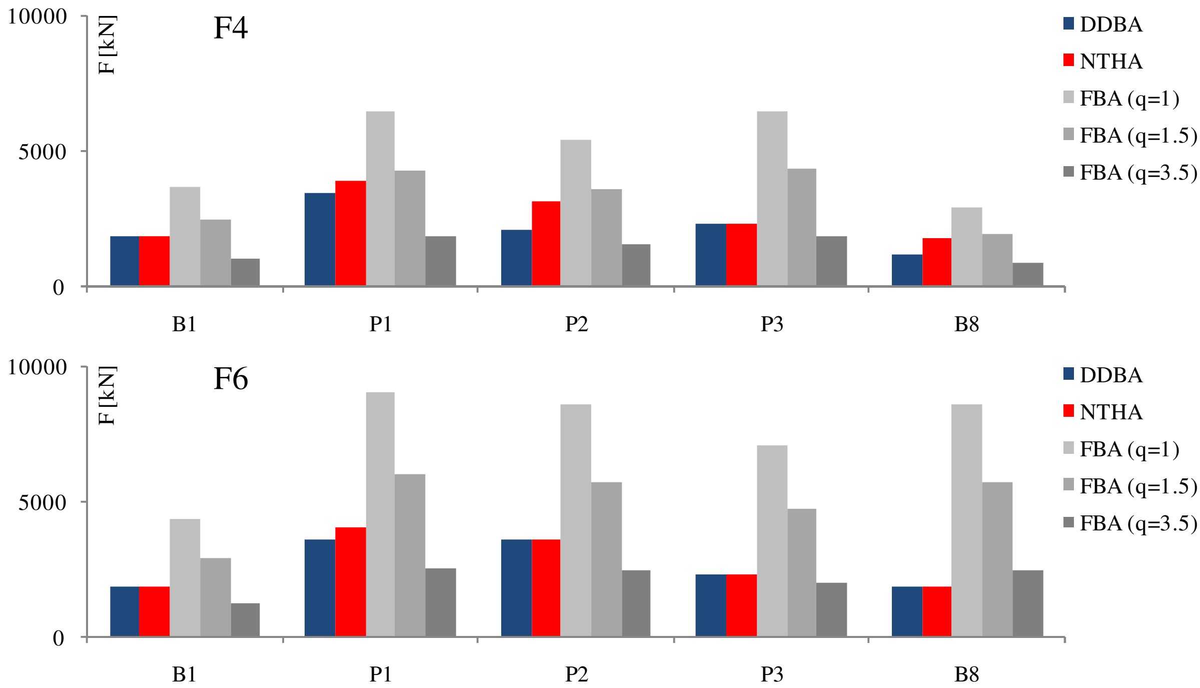


Figure 18: Comparison between DDBA, FBA and NTHA results (average maximum values) in terms of base shear of piers and abutments.

4 CONCLUSIONS

A Direct Displacement-Based Assessment (DDBA) methodology for the seismic evaluation of multi-span simply-supported deck bridges has been proposed. The proposed DDBA procedure provides the PGA values associated to given Damage States (DS) of the critical elements of the bridge (piers, bearing devices,....). The target displacement profile of the bridge corresponding to the selected DS is determined through an Iterative Eigenvalue Analysis (IEA).

In the paper, the proposed DDBA procedure has been applied to a set of ten multi-span simply-supported deck bridges, differing in pier layout (regular or irregular), bearing type (neoprene pads or steel hinges) and deck type (isostatic or continuous). The predictions of the DDBA procedure have been compared to the results of Nonlinear response Time-History Analyses (NTHA), carried out using a set of seven accelerograms, compatible with the EC8-soilB response spectrum, scaled to the PGA values provided by DDBA procedure for selected DSs. The comparison between DDBA predictions and NTHA results confirms the good accuracy of the proposed procedure in predicting the PGA values associated to slight-to-severe Damage States, regardless pier layout, bearing type and deck type. In all the examples of application considered, indeed, the DDBA correctly identify the critical element of the bridge, where first a given DS is reached. The displacement profile of the bridge predicted by the DDBA (including joint displacements, top pier displacements, bearing device displacements and deck rotations) is in good agreement with the maximum deformed shape of the bridge found with NTHA, with errors that do not exceed 15 %. Meanwhile, the accuracy of the traditional force-based seismic assessment approach is very sensible to the definition of an appropriate behaviour factor and, in the investigated cases, rather low.

Although the proposed methodology appears 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, in order to fully verify the proposed procedure. Future research shall focus also on the influence of different modelling assumptions and modelling approaches.

Finally, considering that the proposed DDBA procedure for multi-span reinforced concrete bridges is very simple and it can be applied to bridges with different typologies with small revisions, the use of this procedure is recommended when the time and conditions are limited to perform more detailed nonlinear analyses.

5 ACKNOWLEGEMENTS

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