APPLICATION OF DIRECT DISPLACEMENT-BASED DESIGN TO MULTI-SPAN SIMPLY SUPPORTED DECK BRIDGES WITH SEISMIC ISOLATION: A CASE STUDY

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ABSTRACT:
The Direct Displacement-Based Design (DDBD) is a modern method for the seismic design of structures. The fundamental goal of DDBD is to obtain a structure which respond according to a given target displacement profile, when subjected to earthquakes consistent with a given reference response spectrum. The DDBD method has been recently developed for different structural types, including: frame/wall buildings, masonry buildings, continuous bridges and structures with seismic Isolation Systems (IS's). In this paper, the DDBD procedure developed by the authors of this paper for bridges with seismic isolation is applied to a case study, represented by a 10-spans simply supported deck bridge of the Italian A16 Highway. The key aspect of the proposed design procedure is the target displacement profile of the bridge. It is specified by assigning a predetermined displacement pattern and a target displacement amplitude to the deck. The target displacement amplitude is assigned by the designer to comply with a given performance level, expressed through a limit value of the maximum displacements of IS's, piers and joints.
In the paper, the background and implementation of the proposed design procedure is presented first. This is followed by some design examples and validation studies through nonlinear time-history analyses.

KEYWORDS: Direct Displacement-Based Design, Seismic Isolation, Bridges, Time-History Analysis

1. INTRODUCTION

Displacement-Based Design (DBD) approaches have received great attention in recent years, as they seem to be the most appropriate ways for implementing concepts of Performance-Based Earthquake Engineering (Bertero and Bertero, 2002). One of such approaches is the so-called Direct Displacement-Based Design (DDBD) method, proposed by Priestley and co-workers (Priestley et al., 1993, 2007). The fundamental goal of DDBD is to obtain a structure which respond according to a given target displacement profile, when subjected to earthquakes consistent with a given reference response spectrum. The DDBD method has been recently implemented for different structural types, including buildings and bridges with seismic Isolation Systems (IS's) (Cardone et al., 2008).
The performance objectives of the DDBD procedure proposed by Cardone et al. (2008) for bridges with IS's can be summarized as follows: (i) the seismic response of superstructure (deck, joints, restrainers) and substructure (piers, abutments, foundations) must remain elastic under the design earthquake, (ii) the ultimate capacity of the IS, in terms of both strength and displacement, must not be exceeded under the design earthquake, (iii) adequate clearance must be provided to the joints, in order to accommodate the IS displacements, in both longitudinal and transverse direction, thus avoiding impacts between structural elements or damage to movement joints.
The proposed DDBD procedure has been specialised for different IS types (Skinner et al., 1993), including: (1) High Damping Rubber Bearings (HDRB), (2) Lead Rubber Bearings (LRB), (3) Friction Pendulum Bearings (FPB) and (4) combinations of Flat Sliding Bearings (FSB) with different auxiliary devices. As far as the bridge configuration is concerned, the proposed DDBD procedure can be applied to multi-span both continuous deck and simply supported deck bridges with piers of different heights.
The key aspect of the proposed design procedure is the target displacement profile of the deck. It is specified by assigning a suitable displacement pattern and a target displacement amplitude to the deck, separately in the

transverse (Fig. 1(a)) and longitudinal direction of the bridge. In the transverse direction, in particular, a rigid translation of the deck is imposed, by assuming an optimal distribution of pier-IS stiffnesses. In the longitudinal direction, instead, a target value of the deck-abutment joint displacement is assigned while the maximum relative displacements of the internal joints (for simply supported deck bridges) are only checked to satisfy given limit values.

In the next step of the procedure, the nonlinear MDOF model of the bridge is converted into an equivalent linear SDOF system (see Figure 1(b)), based on the standard equations of the DDBD method (Priestley et al., 2007), which provide the design displacement (Δ_d), effective mass (m_e) and equivalent damping (ξ_{eq}) of the equivalent linear SDOF system, based on the target bridge displacement profile. A Response Spectrum Analysis (RSA) is then performed (see Figure 1(c)) to determine the equivalent stiffness (K_{eq}) of the equivalent linear SDOF system and its design lateral force $F_d = K_{eq} \cdot \Delta_d$, corresponding to the design base shear of the real structure. Finally, the IS mechanical characteristics and pier reinforcement ratios consistent with the selected target displacement profile and corresponding design force levels are derived.

In the following paragraphs the basic modelling assumptions of the proposed procedure are presented first. This is followed by a brief description of the main steps of the proposed procedure. Finally, the design procedure is applied to a case study and the expected seismic response compared to the results of Nonlinear Time-History Analysis (NTHA).

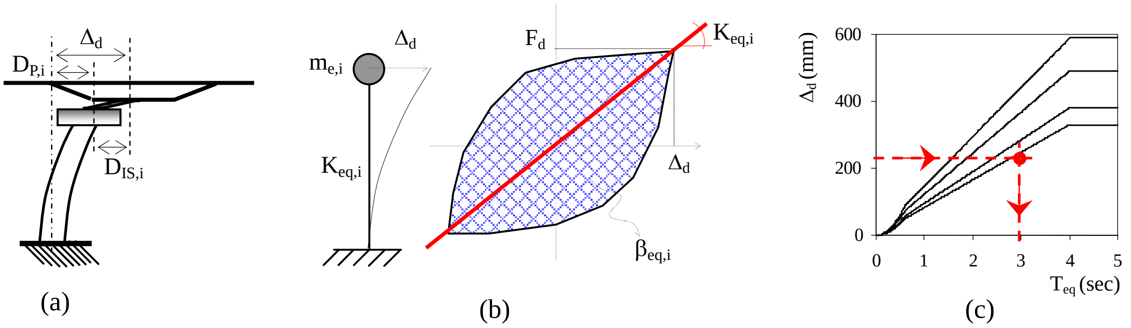


Figure 1: Schematic representation of the DDBD method for seismically isolated bridges.

2. BRIDGE MODELING AND SCHEMATIZATION

The bridge is preliminarily divided into a number of 2-DOF pier-IS-deck systems (see Fig. 2), characterized by two lumped masses, representing the tributary masses of deck ($M_{D,i}$) and pier ($M_{P,i}$), respectively, and two unidirectional elastic springs, modeling the lateral stiffness of the pier ($K_{P,i}$) and the effective stiffness of the IS at the design displacement ($K_{IS,i}$), respectively. Abutments, decks and foundations are considered as infinitely rigid and resistant.

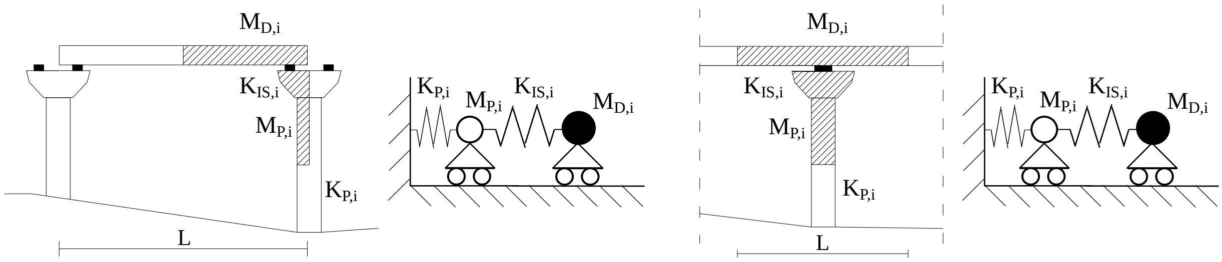
Each 2-DOF pier-IS-deck system is reduced to an equivalent 1-DOF system characterized by effective mass ($m_{e,i}$), equivalent stiffness ($K_{eq,i}$) and equivalent damping ($\beta_{eq,i}$) calculated through the well-known relationships (see Figs. 1 and 2):

$$m_{e,i} = \frac{(M_{P,i} \cdot D_{P,i} + M_{D,i} \cdot \Delta_d)^2}{M_{P,i} \cdot D_{P,i}^2 + M_{D,i} \cdot \Delta_d^2} \tag{2.1}$$

$$\frac{1}{K_{eq,i}} = \frac{1}{K_{P,i}} + \frac{1}{K_{IS,i}} \tag{2.2}$$

$$\beta_{eq,i} = \frac{D_{P,i} \cdot \xi_{P,i} + D_{IS,i} \cdot \xi_{IS,i}}{D_{P,i} + D_{IS,i}} \tag{2.3}$$

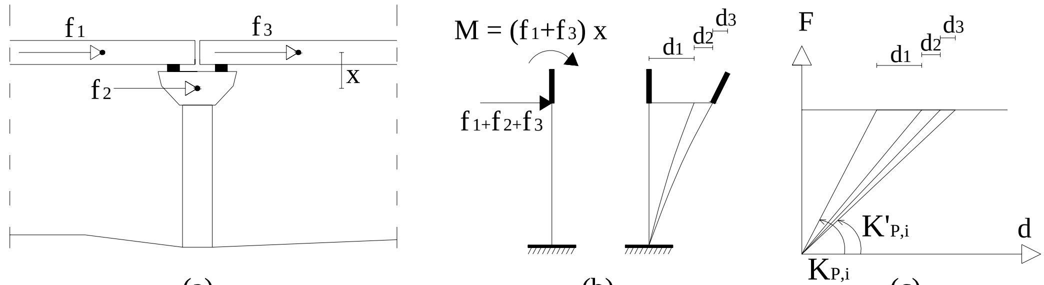
where $\xi_{P,i}$ and $\xi_{IS,i}$ are the viscous damping ratio of the i-th pier (typically 5%) and the effective damping ratio of the corresponding IS at its maximum displacement ($D_{IS,i}$), respectively.



(a) (b)

Figure 2 2-DOF systems for multi-span (a) simply supported and (b) continuous deck bridges.

The equivalent 1-DOF systems thus derived are assembled using the DDBD approach, exploiting the rigid-body constraints imposed by the deck(s). For continuous deck bridges, in particular, the DDBD method is applied to the bridge as a whole. For simply supported deck bridges, instead, the DDBD method is applied to independent stand-alone spans, considered as completely separated from the adjacent spans at the joints. In both the cases, the seismic responses of the bridge in the longitudinal and transverse direction are examined separately.
Actually, a series of modeling adjustments have been made, in order to take into account a number of aspects which should be neglected with the aforesaid modeling assumptions. The most important aspect refers to the piers of multi-spans simply supported deck bridges, which simultaneously undergo the seismic effects of two adjacent spans (Fig. 3(a)). Additional aspects neglected with the aforesaid modeling assumptions are the contributions to the horizontal top displacement of the piers due to the real geometry of the bridge. The simple model previously described, indeed, does not consider the real point of application of the inertia forces of the decks (Fig. 3(a)), neither the rigid rotation of the pier-cap of bents responding according to a cantilever scheme (Fig. 3(b)). As a consequence, an auxiliary reduced pier stiffness ($K'_{P,i}$ in Fig. 3(c)) has been defined, in order to capture the overall horizontal top displacement of the pier ($d_1+d_2+d_3$ in Fig. 3(b)-(c)).



(a) (b) (c)

Figure 3 (a) Bridge configuration, (b) deformed shape of the pier and single contributions to its top displacement, (c) auxiliary reduced pier stiffness.

The auxiliary reduced stiffness of the piers is derived from their real lateral stiffness ($K_{P,i}$ in Fig. 3(c)) through an iterative process, carried out within the step-by-step DDBD procedure described in the next paragraph, involving the displacements and stiffnesses of pier and IS, respectively.
More detailed information on the schematic force-displacement behaviour and basic mechanical properties assumed for piers and IS's can be found in (Cardone et al., 2008).

3. DIRECT DISPLACEMENT-BASED DESIGN OF BRIDGES WITH SEISMIC ISOLATION

The design philosophy of the proposed procedure is based on the general requirement that the full serviceability of the bridge should be maintained after the design earthquake, so that there should be no need to reduce traffic over the bridge nor to carry out any repairs. In accordance with the modelling assumptions of the proposed

procedure (see paragraph 2.), the bridge can be deemed to satisfy the aforesaid general requirement if the piers remain in their elastic range, the IS's do not exceed (with a proper safety factor) their ultimate displacement capacity and the joints present adequate space to accommodate the IS displacements in both longitudinal and transverse direction.
The fundamental parameter of the proposed procedure is the design displacement of the IS ($D_{IS,i}$). It is assigned by the designer based on preliminary considerations, taking into account the IS type selected. The pier displacements ($D_{P,i}$) and the relative displacements of the joints ($d_{J,i}$) are iteratively checked during the step-by-step design procedure, in order to verify that they actually result lower than the corresponding yield displacements ($D_{y,i}$) and available clearances (d_{gap}), respectively.
A preliminary selection of the IS type (and associated mechanical parameters), design IS displacement and (for new bridges) optimal pier reinforcement ratio is strongly recommended. This can be done using graphical tools similar to those reported in Fig. 4(a) and 4(b), for existing and new bridges, respectively. The diagrams of Fig. 4 show a number of high-damping elastic spectra in the so-called ADRS (Acceleration-Displacement-Response-Spectra) format. Basically, each IS type is characterised by a different damping level (Cardone et al, 2008). As a consequence, each IS type can be associated to a different group of response spectra. The dashed lines passing through the origin of the axis correspond to two limit values of the effective period of vibration of the bridge with seismic isolation, equal to $3T_{fb}$ (being T_{fb} the fundamental period of vibration of the bridge w/o seismic isolation) and 4 sec, respectively. The interceptions of such radial lines with the response spectra at 10% and 50% damping, define a preliminary range of possible IS design displacements. On the left hand side of each ADRS diagram of Fig. 4, the schematic displacement vs. acceleration relationship of the most critical (lowest shear/flexural strength) pier of the bridge, in the considered direction, is reported. For existing bridges (see Fig. 4(a)), the most critical pier is identified by a given reinforcement ratio (ρ*). For new bridges, a suitable reinforcement ratio (ρ') is initially assumed, within the typical values for RC columns (1% < ρ' < 4%). Considering that an elastic response of all the piers is required, only the Performance Points (PP's) which fall inside the painted background of the ADRS diagrams can be preliminarily selected. Obviously, two further aspects affect the selection of the PP, i.e.: the clearance of the joints and the IS displacement capacity. Once a suitable PP has been selected, the associated damping ratio (hence IS type), design IS displacement, isolation period and (for new bridges) pier reinforcement ratio are identified.

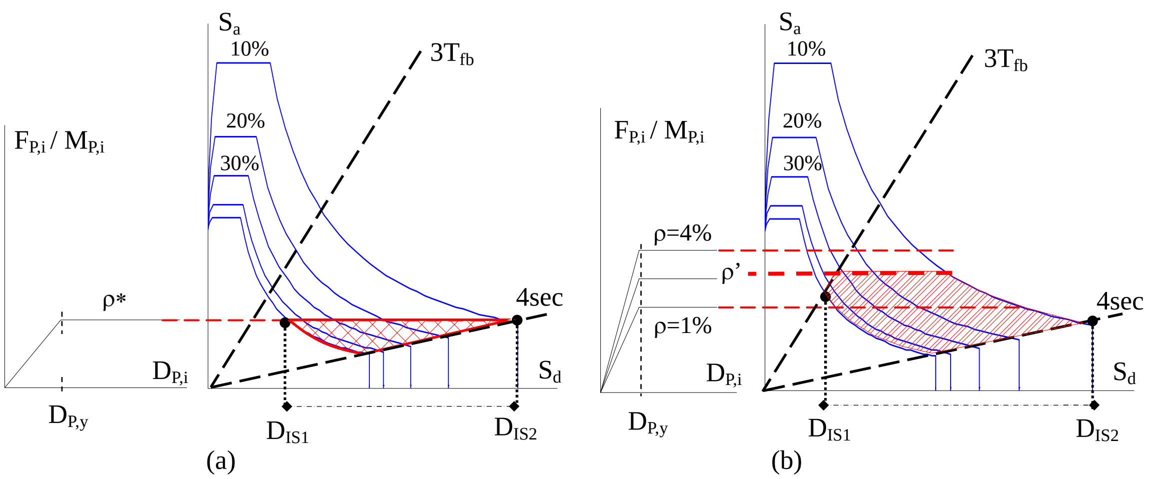


Figure 4 Preliminary selection of IS type, design IS displacement (and pier reinforcement ratio) for the (a) retrofit of existing bridges and (b) design of new bridges.

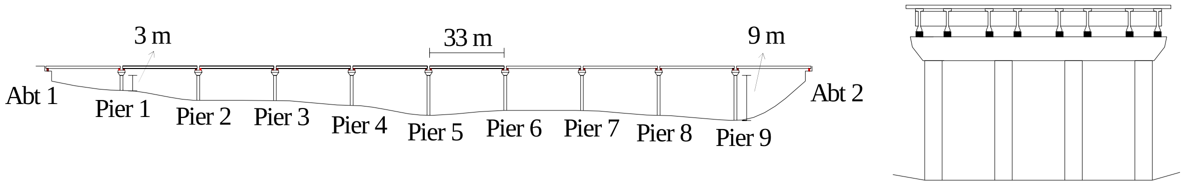
The proposed DDBD procedure consists of eight steps.
Step 1: input data are defined. They include bridge geometry, pier dimensions, bridge masses, pier reinforcement ratios (for existing bridges only), etc.. For what concerns the lateral stiffness and yield displacement of the piers ($D_{P,y}$), reference is made to the approach suggested by Priestley et al. (2007) based on extensive experimental results, according to which the effective elastic stiffness of cracked concrete sections is essentially proportional to strength and the yield displacement is constant and independent from strength. In the evaluation of $D_{P,y}$, the risk of premature shear failures is taken into account. Step 1 also includes the selection of

the IS type and the design displacement of the deck (Δ_d) in the critical direction of the bridge, based on the indications of the preliminary design procedure sketched in Fig. 4. As a matter of fact, the DDBD approach is applied twice, the first time in the critical direction, to derive the IS design characteristics, the second time in the other direction, to verify the suitability of the bridge response.
Step 2: a trial optimal distribution of the IS effective stiffnesses ($K_{IS,i}$) is assigned. it guarantees a uniform rigid translation of the deck in the transverse direction. This automatically determines the contributions of each pier and IS to the design displacement of the deck (i.e. $\Delta_d = D_{P,i} + D_{IS,i}$). The optimal stiffness distribution is determined by centring the centre of stiffness of the pier-IS systems with respect to the centre of mass of the bridge. In the first cycle of iteration the mass of the piers is neglected. In the following cycles, the analysis is repeated referring to the effective masses of the pier-IS-deck systems (see Eq. (1)).
Step 3: the 2-DOF pier-IS systems are examined, deriving the associated equivalent stiffness (see Eq. (2)) and equivalent damping ratio (see Eq. (3)).
Step 4: the MDOF model of the bridge is converted into an equivalent SDOF model. The equivalent stiffness of the SDOF model of the whole bridge (or single span for multi-span bridges) is obtained by summing, in parallel, the equivalent stiffness values of each pier-IS system (i.e. $K_{eq} = \sum K_{eq,i}$). The same is done for the effective mass (i.e. $m_e = \sum m_{e,i}$). The global equivalent damping is derived by combining the contributions of each pies-IS system weighted with their effective mass (i.e. $\xi_{eq} = \sum \left(m_{e,i} \cdot \beta_{eq,i}\right) / \sum m_{e,i}$).
With the target displacement Δ_d assumed in Step 1, the displacement response spectrum at ξ_{eq}(%)-damping is entered (**Step 5**), to determine the equivalent period (T_{eq}) of the SDOF system (see Fig. 1(c)). A new value of global equivalent stiffness (K^*_{eq}) can be then derived and compared to that used in Step 4. If they differ more than a given tolerance (i.e. $|K^*_{eq} - K_{eq}| > \varepsilon$), the equivalent stiffnesses of the IS's ($K_{IS,i}$) are revised according to K^*_{eq} and steps 2-5 repeated until convergence is reached.
At the end of the iterative process, the verifications of piers and joints are carried out (**Step 6**), based on the maximum displacements provided by the analysis (i.e. $D_{P,i} < \alpha D_{y,i}$ and $|d_{J,i}| < \beta d_{gap}$, with $\alpha,\beta < 1$). If pier verifications are not satisfied, a new design displacement Δ_d is selected and steps 2-6 repeated. In the last step of the design procedure (**Step 7**) the mechanical characteristics of each IS are fully specified, based on their equivalent linear maximum response and the mechanical parameters (e.g. friction coefficient, post-yield hardening ratio, viscous damping ratio) assumed at the beginning of the analysis.
Obviously, the number of isolation devices with different mechanical characteristics must be limited to one or two, over the whole bridge, in order to contain costs and reduce installation difficulties. As a consequence, at the end of the design procedure, the IS characteristics must be properly revised and adjusted to the common practice. Referring to this last IS configuration, a Linear Static Analysis (LSA) is performed (**Step 8**), modelling the IS's through their effective stiffnesses, in order to verify deformations and stresses in all the structural members, including IS's, piers, abutments, joints and foundations.
In any case, a final verification through time-history analysis is always recommended.
The procedure for the design of new bridges with seismic isolation is practically the same as for the retrofit of existing bridges, except for the pier reinforcement ratios, which are still unknown at the beginning of the analysis and must be preliminarily designed in Step 6 and then verified through LSA.

4. APPLICATION TO A CASE STUDY AND COMPARISON WITH TIME-HISTORY RESULTS

As a preliminary verification, the proposed design procedure has been applied to a case study, given by a bridge of the late 60's in the A16 (Napoli-Canosa) Italian Highway. In order to evaluate the reliability and accuracy of the proposed procedure, the design predictions have been compared to the results of Nonlinear Time-History Analyses (NTHA). The NTHA have been carried out with the finite element program SAP2000-Nonliner (Computers and Structures, 2002), using 7 different accelerograms, compatible (on average) with a corrected version (T_D = 4 sec instead of 2.5 sec) of the displacement response spectrum of Eurocode 8 (CEN, 1998) for soil type C, PGA = 0.35g.
The bridge is made of a 10-span simply supported deck of 330m total length (see Fig. 5(a)). The decks, realised by 8 prestressed concrete beams, are supported by 9 frame-type piers (see Fig. 5(b)), characterised by 4 RC columns with 1.25m diameter circular cross section and free heights ranging from 3m to 9m. The longitudinal

reinforcement ratio of each column is equal to 1.4%. The transverse reinforcement is realised by 10mm diameter hoops at 20mm spacing. The yield strength of the reinforcing steel (type AQ50-60) is equal to 230 N/mm^2. As far as concrete is concerned, a compression strength of 35 N/mm^2 is expected, based on the design data available.



(a) (b)

Figure 5 Case study (Macchione bridge): (a) longitudinal profile, (b) pier transverse section.

The clearance of the deck-abutment joints is equal to 300mm in the longitudinal direction and 200m in the transverse direction. The clearance of the joints between adjacent decks is equal to 100mm in the longitudinal direction and 200mm in the transverse direction.
Pier n. 9 is the most critical (lowest flexural strength) pier of the bridge (see Fig. 5), for transverse actions. Pier n. 9 has been then considered in the preliminary selection of IS type and design displacement (see Fig. 4(a)). The following design parameters have been assumed at the beginning of the DDBD procedure: (i) design displacement of IS (and deck) in the longitudinal direction equal to 250mm, (ii) HDRB/LRB IS types, (iii) 20% effective damping ratio and (iv) maximum pier displacement equal to 82mm, corresponding to 70% of its yield displacement.
The aforesaid selection resulted in 20 identical isolation units, arranged between the deck and the top of piers/abutments, and characterised by 8 identical HDRB/LRB devices (one for each deck beam, see Fig. 5(b)) with effective stiffness at the design displacement (250mm for the devices located on the abutments) equal to 0.69KN/mm. Based on the proposed design procedure (see paragraph 3.), the isolation system thus defined satisfies all the required performance objectives and associated compliance criteria.
In Figure 6 the limit/target values of the displacement response of the bridge are compared to the results (maximum values averaged over 7 accelerograms) of accurate NTHA. The comparison is made in terms of relative displacements of the joints in the longitudinal direction (Fig. 6(a)) and absolute displacements of the decks in the transverse direction (Fig. 6(b)). It should be noted that the relative displacements of the deck-abutment joints, shown in Fig. 6(a), practically coincide with the absolute displacements of the corresponding isolation devices. In the diagrams of Fig. 6, the limit/target values of the deck displacement are indicated with tick continuous/dashed lines. In the transverse direction, the limit displacement follows a step-like trend because it takes into account the maximum displacements of each pier.
In the numerical model adopted for NTHA, the piers have been modelled by elastic beam elements with distributed mass and 5% viscous damping ratio. Rigid link elements have been used to model the pier-caps and simulate the real distance between the deck intrados and its barycentric axis. The deck has been modelled as a rigid horizontal diaphragm. The masses of pier-cap and deck have been lumped in a suitable number of joints, based on tributary areas. The isolation devices, finally, have modelled with bi-directional spring elements characterised by a visco-elastic (HDRB) or elasto-plastic with hardening (LRB) force-displacement behaviour.
Figure 6(a) points out a good accuracy in the attainment of the target displacement in the longitudinal direction of the bridge, with percent errors less than 5-10% for deck10 (228 vs. 250 mm for LRB and 239 vs. 250 mm for HDRB, precisely) and less than 25% for deck1 (188 vs. 250 mm for LRB and 191 vs. 250 mm for HDRB, precisely). The aforesaid discrepancies, between NTHA results and design predictions, must be ascribed to the significant variability of pier heights and to the design simplification of adopting only one type of isolation device throughout the bridge. It is worth to mention that such discrepancies were expected (and then accepted), based on the LSA results (Step 8 of the proposed procedure).
Figure 6 shows that all the maximum displacements of the joints comply with the available clearance, both in the longitudinal and transverse direction. In addition, the deformed shape of the bridge in the transverse direction (see Fig. 6(b)) is characterised by an almost uniform translation of all the decks (maximum rotations lower than 0.02 degrees), with negligible relative displacements at the joints. Finally, all the piers remain elastic (see Fig. 8).

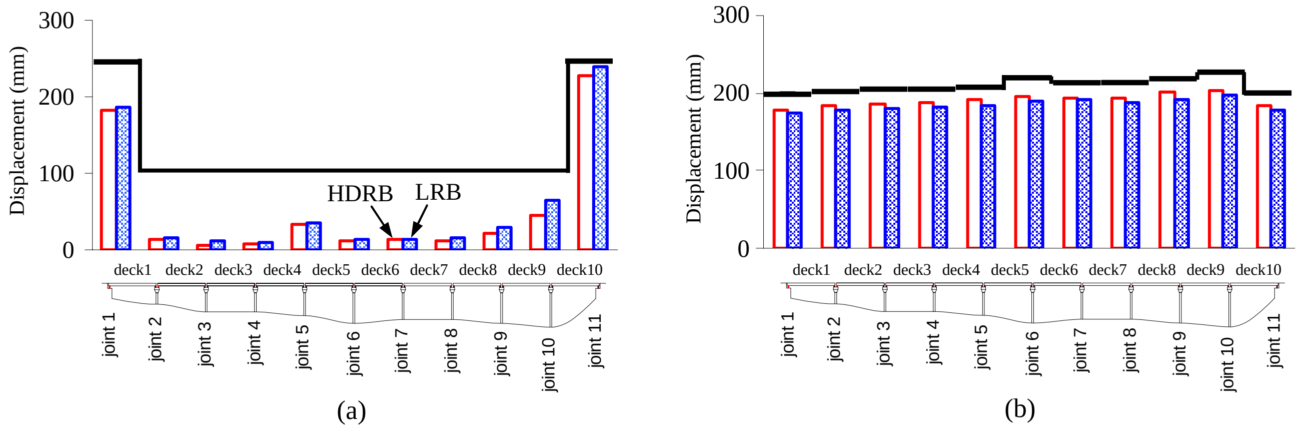


Figure 6 Comparison between the maximum displacement response from NTHA and the corresponding limit/target values: (a) relative displacements of the joints in the longitudinal direction and (b) absolute displacements of the decks in the transverse direction. Isolation system realised by HDRB/LRB devices only.

The IS configuration under examination is very simple, since it relies upon an unique type of isolation device. The only drawback is that the bridge response is not optimised in both directions, as the little margin with respect to gap closure of joint1 and joint11 in the transverse direction reveals (see Fig. 6(b)). For this reason, another IS configuration has been examined. It is based on the combination of HDRB/LRB devices with unidirectional Pot-Sliding-Bearings (PSB) placed on the top of a number of IS devices. The use of PSB's allows to decouple the IS response in the longitudinal and transverse direction, thus giving the possibility to select two different target displacements.

Fig. 7 shows the seismic response of the bridge equipped with an IS (realised with HDRB/LRB + PSB) that exhibits a different behaviour in the two directions. In the example of Fig. 7, the target displacement of the deck have been set equal to 250mm in the longitudinal direction and 150mm in the transverse direction, respectively.

In the longitudinal direction (see Fig. 7(a)), the percent differences between target displacement and maximum displacements from NTHA do not exceed 10% for both HDRB and LRB. The relative displacements between adjacent decks, moreover, are considerably reduced, compared to the first IS configuration (see Fig.6(a)).

Also in the transverse direction (see Fig. 7(b)), the percent differences between target and maximum displacements do not exceed, on average (over 10 decks), 10% for both HDRB and LRB, thus guaranteeing an adequate margin with respect to the gap closure of the joints. Moreover, the deformed shape of the bridge is characterised by an almost uniform translation of all the decks (maximum rotations lower than 0.02 degrees). Finally, all the piers remain elastic (see Fig. 8).

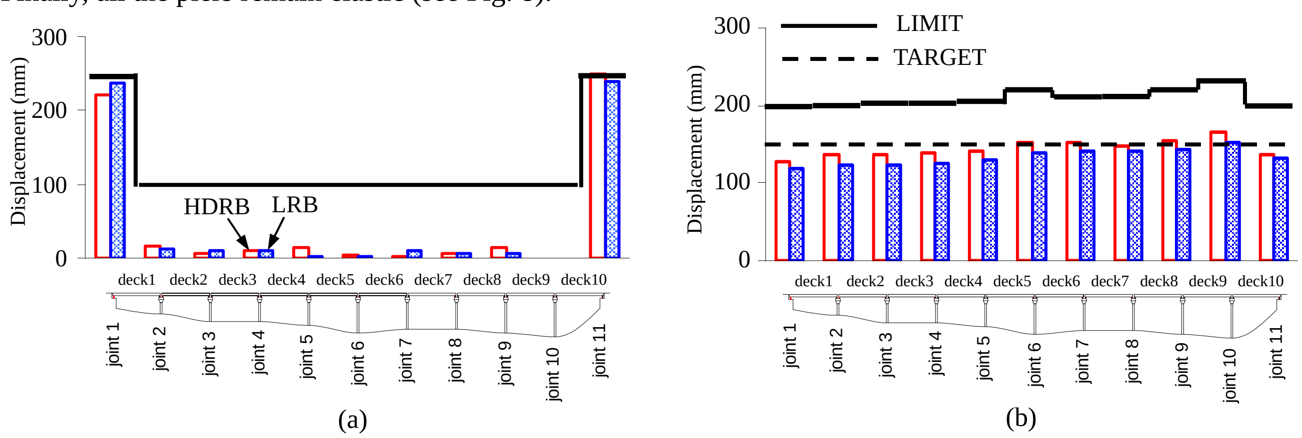


Figure 7 Comparison between the maximum displacement response from NTHA and the corresponding limit/target values: (a) relative displacements of the joints in the longitudinal direction and (b) absolute displacements of the decks in the transverse direction. Isolation system realised by HDRB/LRB devices coupled with unidirectional Pot-Sliding-Bearings.

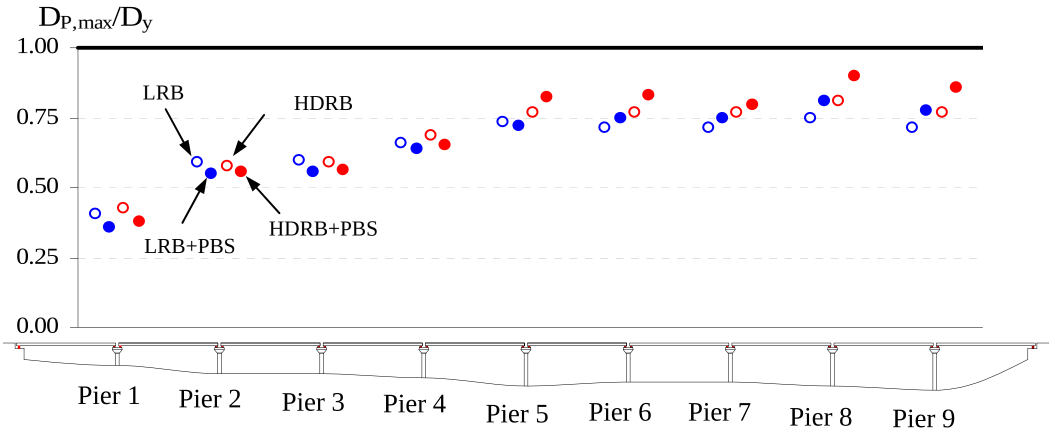


Figure 8 Ratios between the maximum displacements of piers from NTHA ($D_{P,max}$) and the corresponding yield displacements (D_y), for different IS types and IS configurations.

5. CONCLUSION

A Displacement-Based Design (DBD) procedure for continuous and simply-supported deck bridges with seismic isolation has been described. It has been derived from the Direct DBD method proposed by Priestley and co-workers. The key aspect of the proposed procedure is the definition of a uniform target displacement profile of the deck, in both the longitudinal and transverse direction of the bridge. The target displacement amplitude is assigned by the designer to accomplish a series of performance objectives, which can be summarised as follows: (i) elastic behaviour of the piers, (ii) maximum IS displacement lower than its ultimate displacement capacity, (iii) adequate margin with respect to the gap closure of the joints. Obviously, a suitable compromise between the optimal IS configuration provided by the design procedure and a number of economic/practical needs must be pursued. Basically, this means to limit the number of different types of IS devices to one or two, thus accepting some errors in the attainment of the target displacement profile and target displacement amplitude of the deck.
As a preliminary verification, the proposed design procedure has been applied to a 10-spans simply supported deck bridge of the Italian A16 Highway, characterised by a significant variability of the pier heights. The design predictions have been found in good accordance with the results of accurate Nonlinear Time-History Analyses (NTHA): all the design performance objectives are satisfied with an adequate margin, although the maximum displacement response of the bridge is a little overestimated.

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