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DEFORMATION-BASED DESIGN OF SEISMICALLY ISOLATED CONCRETE BRIDGES

K.I. Gkatzogias(1), A.J. Kappos(2)

(1) *PhD Candidate, Research Centre for Civil Engineering Structures, Dept. of Civil Engineering, City, University of London, UK, Konstantinos.Gkatzogias.1@city.ac.uk*

(2) *Professor, Research Centre for Civil Engineering Structures, Dept. of Civil Engineering, City, University of London, UK, Andreas.Kappos.1@city.ac.uk*

Abstract

A performance-based design (PBD) procedure, initially proposed for bridges designed for ductile behaviour of piers, is adapted herein to seismically isolated bridges. Accounting for multiple performance objectives, it initially identifies the critical hazard level and 'near-optimal' alternatives of the isolation system in terms of both economy and performance based on the inelastic response of a single-degree-of-freedom (SDOF) system. By incorporating nonlinear response history analysis (NLRHA) of the multi-degree-of-freedom (MDOF) system in a number of successive design steps that correspond to different performance levels (PLs), it subsequently leads (in a non-iterative way) to a refinement of the initial design solution through the control of a broad range of material strains and deformations. The efficiency of the proposed design methodology is evaluated by applying it to an actual bridge that was previously designed for ductile behaviour. Assessment of the design using NLRHA for spectrum-compatible motions reveals enhanced seismic performance and cost reduction in the substructure design, thus, rendering base-isolation an appealing design alternative.

Keywords: bridges; performance-based design; seismic isolation; nonlinear dynamic analysis

1. Introduction

Performance-based seismic design concepts aiming at the design of structures with a predefined structural response (hence a known level of safety, damage and loss) under a specific or, preferably, under multiple, seismic hazard levels have long been established. In fact, the PBD challenge can be addressed to a certain extent within a purely probabilistic framework (e.g. [1]); however, this approach cannot be justified (at least not yet) in a practical design context due to the associated increased computational effort. Instead, PBD procedures addressing bridge structures in general, and more specifically, seismically isolated bridges, with far less applications in the latter case (e.g. [2], [3]), have been mainly developed so far on the basis of linear equivalent-static/dynamic analysis (an example being the 'direct displacement-based design' method) in an attempt to reconcile the design principles of 'simplicity' and 'enhanced seismic performance'. Notwithstanding the underlying limitations (e.g. imposed lower-bound limits on the inelastic response derived from 'simpler' elastic analysis, absence of comprehensive guidelines), the adoption of NLRHA in modern codes for the seismic design of isolated (as opposed to designed for ductility) bridges seems to be in contrast with the previous approach, reflecting an emerging scepticism about the use of equivalent linearisation techniques due to certain associated pitfalls, such as, the requirement for iterations, the inaccurate estimation of inelastic response, due to the introduction of the ill-defined (i.e. non-physical) 'effective' isolation period [4], the 'equivalent' damping ratio [5], and the inconsistent treatment of systems that involve non-classical (or non-proportional) damping matrices [6].

In view of the available bridge design practice that involves advanced analysis tools, complex structural configurations and diverse loading conditions (more so in the case of 'important' bridges), the present study attempts to strike a balance among the aforementioned trends in PBD, presenting a rigorous design methodology for seismically isolated concrete bridges, capable of reflecting the current state-of-the-art in a performance-based context. This involves explicit design for multiple PLs and a broad range of design parameters, while providing the required tools for a direct comparative evaluation, at the early stages of design, of alternative isolation schemes that may consist of isolation and supplementary energy dissipation devices. The suggested approach, called Deformation-Based Design (Def-BD), originates from recent work presented by the authors on the seismic design of 'ductile pier' bridges [7], and earlier work of the second author and his associates focusing on buildings [8]. Detailed steps and required modifications (with regard to [7]) accounting for the peculiarities of isolated bridges (which arise from the use of passive control devices) and the associated performance objectives are put forward. The efficiency of the proposed methodology is subsequently investigated by applying it to an actual bridge previously used by the authors in the extension of Def-BD to the case of 'ductile pier' bridges. The suggested procedure and the resulting designs for different isolation schemes are evaluated in the light of NLRHA using a number of spectrum-compatible motions, whereas a comparison among different designs addressing both economy and structural performance is also presented.

2. Description of the Def-BD methodology

The target performance sought in Def-BD of seismically isolated bridges can be described with reference to the member (e.g. pier and isolator) limit states and the associated seismic actions. For common bridges, a 'frequent' earthquake event (denoted EQII and having a return period T_r=50-200yrs, see [7] for EQI, not used herein) is associated with the 'operationality' limit state of isolators, corresponding to non-disrupted service of the bridge, a 'rare' earthquake (EQIII: T_r=500-2500yrs) with quasi-elastic response of piers and minimal damage in the isolators without significant disruption of service, and a 'maximum considered' earthquake (EQIV: T_r>2500yrs) with ultimate response of isolators, limited inelastic response of piers, and limited service of the bridge. The range of T_r coupled with each PL should be seen as indicative of the widely varying requirements prescribed in different codes for common bridges [7]; modification either of the level of performance requirements or of T_r should be in order in the case of bridges of higher or lower importance. The target objectives are also in line with code specifications (e.g. [9]) regarding the necessity of limiting the inelastic response of the substructure, aiming at a proper performance of the isolation system, since it has been demonstrated [10] that when inelastic action develops, the effectiveness of the isolation system may be reduced, resulting in larger deformation demands in the isolated structure. In the light of the previous consideration, controlled inelastic response of the piers (e.g. associated with spalling of concrete cover) is allowed only under EQIV. It is also noted, that a strict approach would require analysis of the seismically isolated bridge for 3 different PLs and 2 different sets of mechanical (i.e. lower (LB) and upper (UB) bound design properties (DP)) of isolators and dampers, i.e. in total, 6 different sets of NLRHAs, each set consisting of 7 (or more) pairs or triplets of accelerograms in an appropriate orientation. The following scheme attempts to reduce the required sets of analyses to 3 (i.e. one per each PL).

Step 1 - Preliminary selection of an isolation scheme with a 'near-optimal' performance under a reference earthquake event: The following preliminary design aims at the identification of the critical (in terms of economy and performance) PL and at a first 'near-optimal' estimation of the basic parameters of the isolation system, namely, its strength ($\overline{v}_0$), post-elastic stiffness (k_p) or isolation period (T_p), and damping ratio (ξ). $\overline{v}_0$ represents the ratio of $V_0/(mg)$, where V_0 is the shear capacity of the isolator at zero displacement and m is the isolated deck mass. The 'near-optimal' isolation solution is defined herein as the one that results in 'near-minimum' peak total acceleration ($\ddot{U}$) of the superstructure while keeping within allowable limits the deformations of the isolation system and the substructure [11]; different approaches can be explored by duly exercising engineering judgement. Analytical investigations [12] imply that an isolated structural system designed for optimal performance under a 'rare' event (denoted as optEQIII) results in suboptimal response with regard to peak relative displacement demands (u) when subjected to stronger earthquake events (e.g. EQIV), compared to a system optimised for the increased hazard intensity (i.e. optEQIV). On the other hand, increased $\ddot{U}$ (and hence base shears) are obtained in the case of the optEQIV system when the latter is subjected to more frequent events (e.g. EQIII). In design terms, the previous observation may be translated into an increased cost of isolators in the first case and an increased cost of reinforcing steel in the piers in the second; hence, the decision on the reference hazard level to be used (i.e. EQIII or EQIV) should be made cautiously. As a means to this end, an approach proposed by Ryan & Chopra [13] for the direct estimation of seismic demand in SDOF systems isolated with bilinear isolators was extended with a view to (a) addressing a wider range of seismic isolation systems that may consist of linear and bilinear isolators (e.g. low/high damping elastomeric bearings (LDRB/HDRBs), lead-rubber bearings (LRBs), sliding bearings (FSB) and friction pendulum bearings (FPBs)), supplementary energy dissipation devices (i.e. linear viscous dampers (VDs)), as well as combinations thereof, (b) developing generalised 'design equations' for the direct estimation of the inelastic demand under code-compatible spectra [14] in terms of both u and $\ddot{U}$, since the maximum force of the system ($m\ddot{U}$) is not directly associated with u due to the introduction of VDs. In this respect, the governing equation of an SDOF system, representing an isolated rigid deck of mass m mounted on isolators and VDs while disregarding the substructure effect, was reduced in the normalised form of Eq. (1) (see also [13]);

$$\ddot{\bar{u}}(t)+2\omega_p\left(\xi_e+\xi_d\right)\dot{\bar{u}}(t)+\omega_p^2 z\left(t,k_e,u,\dot{u}\right)+\omega_p^2\bar{u}(t)=-\left(\omega_p^2\big/\eta\omega_D\right)\ddot{\bar{u}}_g(t),\ \ \ddot{\bar{U}}(\mathrm{t})=\ddot{\bar{u}}(t)+\left(\omega_p^2\big/\eta\omega_D\right)\ddot{\bar{u}}_g(t) \quad (1)$$

$$\ddot{\bar{u}}(t)=\ddot{u}(t)/u_r\,,\ \dot{\bar{u}}(t)=\dot{u}(t)/u_r\,,\ \bar{u}(t)=u(t)/u_r\,,\ \ddot{\bar{u}}_g(t)=\ddot{u}_g(t)/\dot{u}_{g0}\,,\ u_r=\eta\omega_D\dot{u}_{g0}\big/\omega_p^2 \quad (2)$$

$$\eta=\bar{v}_0 g\big/\omega_D\dot{u}_{g0}=\omega_p^2 u_r\big/\omega_D\dot{u}_{g0} \quad (3)$$

In Eqs. (1-3), $\dot{u}(t)$, $\ddot{u}(t)$ are the relative velocity and acceleration of m, respectively (symbols with bars represent normalised quantities), z is a dimensionless parameter (i.e. a function of u, $\dot{u}$, and the initial stiffness (k_e)) that represents the fraction of the applied V_0 (maxima of ±1) [13], whereas ξ_e, ξ_d (corresponding to constant damping coefficients of $2m\omega_p\xi_{e/d}$) introduce viscous damping originating, respectively, from elastomer-based isolators and VDs. $\bar{v}_0$ can be seen as the acceleration at yield of a rigid system with strength V_0 and yield displacement u_r equal to the residual displacement under which the system can be in static equilibrium, i.e. V_0/k_p. The normalised strength (η), which characterizes the system strength relative to the PGV ($\dot{u}_{g0}$), is defined according to Eq. (3); the frequency ω_D, included to make η a dimensionless quantity, corresponds to the period T_D marking the transition from the velocity-sensitive to the displacement-sensitive region of the target spectrum. The EC8 'Type 1' [14] elastic spectrum constituted herein the basis for seismic design, adopting, nevertheless, a T_D of 4.0s in line with recent research findings [15]. By solving Eq. (1) for a range of parameters (ξ=0-0.3, η=0-1.5, T_p=1-5s) under suites of artificial records closely matching the considered spectrum, it was found that the seismic intensity has a negligible effect on the median normalised response. This significant property allows the development of equations that provide direct estimates of peak response under different PLs associated with target spectra of common frequency content but different intensity. In this respect, linear regression equations were fitted to the log-transformed data of the median normalised u, $\ddot{U}$ derived from NLRHA (Fig. 1). Due to space limitations and pending a future publication, 'design equations' presented as Eq. (4) (systems of $\xi\geq$0.05, $\eta\geq$0.25) and Eq. (5) ($\xi\geq$0.05, η=0) are restricted to elastomer-based isolators (yield displacement of u_y=1cm [13]) under a target spectrum with a frequency content corresponding to subsoil class 'C' [14] noting that the seismic intensity is expressed for convenience in terms of PGA on the rock (i.e. a_g in m/s^2).

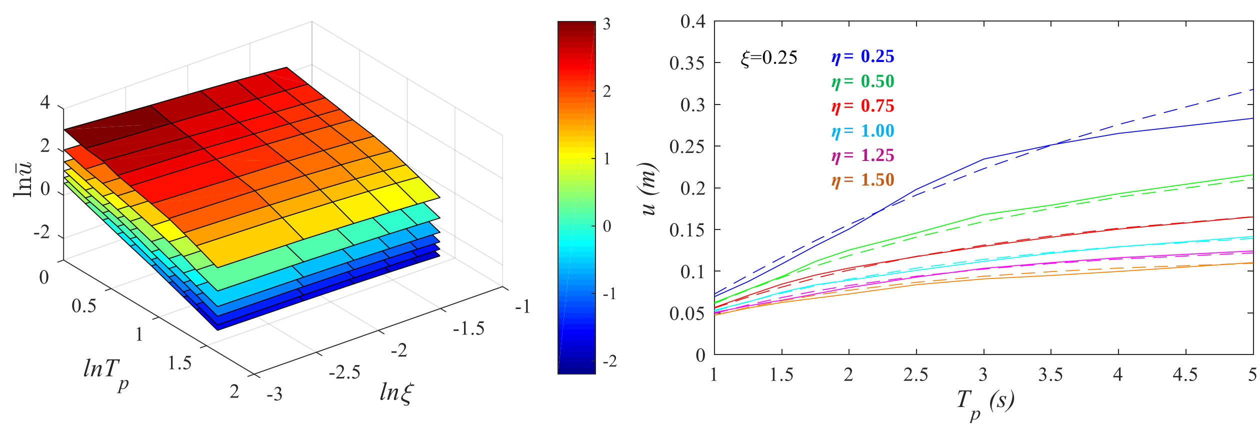

Fig. 1 – Log-transformed normalised median relative displacements ($\ln\bar{u}$) derived from parametric NLRHAs ($\xi\geq$0.05, $\eta\geq$0.25) (left), and median relative displacements (u) (solid lines) derived from NLRHAs (PGA=0.42g and ξ=0.25) compared to values predicted from linear regression model (dashed lines)

$$u = 0.009\xi^{\left(-0.262+0.092\ln\eta+0.104\ln T_p\right)}\eta^{\left(-0.111-0.221\ln T_p\right)}T_p^{\left(1.081-0.203\ln T_p\right)}a_g,$$
$$\ddot{U}=0.68\xi^{\left(-0.108+0.099\ln\eta+0.13\ln T_p\right)}\eta^{\left(0.317+0.271\ln T_p+0.205\ln\eta\right)}T_p^{-0.323}a_g,\ \eta = 4.05\bar{v}_0 g/a_g \tag{4}$$

$$u = 0.011\xi^{-0.406}T_p^{\left(1.554-0.32\ln T_p\right)}a_g,\ \ddot{U} = 0.484\xi^{\left(-0.428+0.193\ln T_p\right)}T_p^{-0.52}a_g \tag{5}$$

'Design equations' in the form of Eqs. (4, 5) are used in this step to identify both the reference event and a 'near-optimal' isolation scheme. u and $\ddot{U}$ inelastic spectra for the adopted EQIII and EQIV seismic intensities can be readily established by plotting Eqs. (4, 5) in a u-$\bar{v}_0$ and $\ddot{U}$-$\bar{v}_0$ format (e.g. Fig. 3 in Section 3), enabling comparisons in terms of both economy and performance among isolation systems with a 'near-optimal' performance under different earthquake intensities, and systems consisting of different isolation and energy dissipation devices. It is noted here that the 'near-optimal' value of η will be the same for a system with a certain ξ and T_p irrespective of the considered PL; nevertheless, it will correspond to different $\bar{v}_0$ in accordance with Eqs. (3, 4). The selection of a system with a 'near-optimal' performance may also encompass additional design constraints, such as, a target T_p, a maximum value of ξ or $\bar{v}_0$, and a target u, accounting for both economy and market availability of the selected dampers, isolators, and expansion/contraction joints. Note also that the decision on the devices required to materialise the selected system will normally follow the selection of ξ, η, T_p, apart from the case when specific restrictions apply. An example of the above procedure is presented in Section 3.

Selecting an isolation system will result in a first estimation of the geometrical and mechanical (LB, UB) properties of devices to be used in subsequent steps, so long as, ξ, η, and T_p, of the selected system are properly distributed to a sufficient number of units located at the piers and abutments, accounting for the weight distribution of the deck to the substructure, and aiming at the minimisation of the eccentricity between the centre of stiffness of the substructure-isolation system and the centre of mass of the supported deck to avoid torsional effects. Uniformity of the stiffness of piers with different height can be achieved to some extent by tailoring the isolator properties so that the bearing stiffness counterbalances the difference in pier stiffness [16]. Notwithstanding the importance of the previous factors, reliability and cost issues will normally dictate the above distribution, e.g. selection of two isolators per pier/abutment is the most reliable and cost-effective design solution in the case of box girder section decks [17], while identical devices are preferable in small-to-moderate bridges since the cost for testing of devices is minimised. It is worth noting that the constraint of maintaining classical normal modes (i.e. distribution of damping constants proportional to the lateral stiffness of the substructure) does not apply here due to the use of NLRHA, hence, optimal distributions of dampers [18] may be explored. Distribution of properties of the selected isolation system and determination of LB/UB-DP of devices will also provide an estimate of the pier strength required to ensure the target performance under the selected reference event, i.e. quasi-elastic pier response of piers under EQIII or controlled inelastic response under EQIV. Regarding the second case, the strength at pier ends should be established to retain the effectiveness of the isolation system under an 'extreme' event through proper consideration of the range within which the inelastic deformations should fall (associated with the degree of damage allowed under EQIV). To meet the aforementioned objective the procedure used in Step 1 of Def-BD for 'ductile pier' bridges [7] to ensure that the bridge remains operational during and after EQII, can be fully implemented herein under EQIV without requiring an elastic analysis; pier column forces and chord rotations can be estimated from the maximum shear transferred through the isolator to the pier top and a proper estimation of the pier equivalent cantilever height (h_{eq}). The reader is referred to [7] for further details on the calculation of reduced design moments that are directly related to allowable deformations. In case the longitudinal reinforcement demand (ρ_l) is found to be less than the minimum requirement, reduction of cross sections is in order.

Step 2 - 'Operationality' verifications: During this step a partially inelastic model (PIM) of the structure is set up, wherein hysteretic isolators and dampers are modelled as yielding and dashpot elements, respectively. In the same model, the remaining parts of the bridge are modelled as elastic members. The flexural stiffness of prestressed concrete deck elements is calculated assuming uncracked deck sections, while reinforced concrete pier column stiffness should correspond either to yield (e.g. M-φ analysis of sections based on ρ_l estimated in Step 1) or, more rationally, to the gross section. NLRHA of the PIM also requires the definition of a suite of ground motions, which, in a design context, should be compatible with the selected design spectrum. Selection and scaling of input motions can be performed according to the procedures described in [7]. The selected earthquake motions will be used for both this step and the following ones, and they should be properly scaled to the level associated with the limit state considered; alternatively, different suites of motions can be established for each PL based on different selection criteria and shape of target spectra in a more refined approach. Verifications under the EQII event should be carried out in terms of both 'operationality' and 'structural performance' of the bridge, hence, limit state design criteria in this PL should ensure both 'full' service of the bridge (i.e. no closure of the bridge) and 'negligible' (or preferably no-) damage of the isolation devices. The 'operationality' requirement can be satisfied by providing an adequate restoring capability, a design strategy common in most codes [17], by dictating the presence of devices that can inherently apply re-centring forces to the superstructure, thus preventing substantial residual displacements and accumulation of displacements during a sequence of seismic events or under earthquake input containing pulses. The parameter mainly affecting the restoring capability of typical bilinear seismic isolation systems, defined as u_{res}/u, is the ratio u/u_r [19], where u_{res} is the residual displacement bounded by u_r (i.e. $-u_r \leq u_{res} \leq u_r$). Due to the non-monotonic variation of u_{res} with respect to the ratio u/u_r, the restoring capability should be assessed on the basis of design charts (e.g. [19], [20]) derived from statistical analysis of responses from a large number of NLRHAs since the absence of u_{res} in NLRHA results under few records is not always indicative of sufficient restoring capability [19]. In view of the previous remark, the isolation system design should aim at 'full' service of the bridge at this PL (i.e. near-zero u_{res} and accumulation of residual displacements under EQII) and at 'limited' service at the PL associated with EQIII. Engineering judgement will be required at this stage in defining allowable u_{res} values associated with the 'closure/non-closure' state of the bridge (e.g. horizontal offsets of approximately 20 and 30 mm were associated with 'non-' and 'brief-closure' in [21]), noting that the capability of bilinear isolation systems is expected on average to be relatively lower for seismic motions involving small-to-moderate displacements and UB-DP. A more stringent ('force-based') 'operationality' criterion can be the limitation of the isolation base shear below V_0 assuming UBDP; this will ensure zero u_{res} of the deck in the case of sliding bearings, and thus, 'full' serviceability of the bridge, but is more difficult to apply in LRBs due to the actual gradual transition from the elastic to the inelastic range of response (i.e. uncertainty with regard to the definition of u_y); nevertheless, the previous criterion can be applied by conservatively estimating u_y. Considering the requirement for 'negligible' damage of isolators, an upper limit of deformation corresponding to the yielding of the steel shims (e.g. shear strains due to lateral deformation γ_q lower than 1.0 [22]) should be applied in the case of elastomer-based isolators and LB-DP.

Use of UB- or LB-DP of devices during the analysis in this step, should be in order for the verification of 'force-based' (including restoring capability) or 'deformation-based' operationality criteria, respectively. In the (common) case when both types of criteria are involved, analysis should be based either on UB- or LB-DP depending on the most critical one (Section 3). Verifications associated with mechanical properties not accounted in the analysis should be conducted using conservative estimates of the demand implicitly related to analysis results through proper modification factors; the latter can be calculated as the ratio of the relevant UB/LB design quantities derived from Eqs. (4, 5) (Section 3). If the adopted limit state criteria are not satisfied, mechanical properties of devices should be modified without performing additional NLRHAs; conformity to the requirements of Step 1 (i.e. performance under the reference event) can be evaluated using Eqs. (4, 5). When the required modifications do not satisfy the target performance set in Step 1, alternative (albeit, probably, less economical) design options can be explored, such as adding sacrificial devices that can restrain the relative movement of the deck to the piers for the relatively low shear forces under EQII. In any case, operationality verifications at this step are not expected to be critical for piers, as the latter are designed for responding quasi-elastically up to the next PL.

Step 3 - 'Minimal damage' verifications: During analysis in this step, the PIM of Step 2 should be used with pier stiffness corresponding to yield and UB-DP of devices (as modified in Step 2). Verifications under a 'rare' event should ensure that the extent of damage is such that the bridge can be easily repaired after the earthquake without causing any significant disruption of service. Regarding the isolation system, the previous requirement can be expressed as an adequate restoring capability allowing for u_{res} that can result in a 'brief' closure of the bridge, and 'minimal' damage in the isolators (e.g. γ_q~2) both evaluated according to the previous step. Required modifications of the isolator mechanical properties should be evaluated based on the requirements of Steps 1, 2; some additional control over the restoring capability requirements can be gained through the adjustment of the substructure stiffness (increase of pier stiffness will normally increase the relative deformation of the isolation system). The performance sought for the substructure at this PL refers to essentially elastic response of piers. When design for flexure is carried out in terms of design values of material strength (hence using commonly available design aids), pier column moment and shears derived from analysis (based on mean values of strength) should be properly reduced similarly to Def-BD of 'ductile-pier' bridges [7]. The final ρ_l ratio should be selected by adopting the highest demand derived from Steps 1 and 2.

Step 4 - 'Life-safety' verifications: Verification of deformations in both the isolation system and the substructure under EQIV constitutes the primary objective in this step. Adoption of LB-DP, followed by an explicit calculation of the deformation demand in the isolation system, or adoption of UB-DP, that will provide an accurate estimation of the deformations in the substructure, should be selected in accordance with the available capacity of substructure members and devices and the estimated response from Eqs. (4, 5). For example, if the estimated displacement demand of the isolators from Step 1 is close to the capacity of the devices selected in subsequent steps, it is preferable to adopt LB-DP during the analysis under EQIV with a view to assessing accurately (through analysis) the deformation demand in the isolation system and implicitly (through modification factors) the pier deformation demand. In any case, the designer may opt for analysing the isolated bridge under both LB- and UB-DP ensuring that neither the devices nor the substructure members are overdesigned. Finally, pier strength and stiffness should be calculated from M-φ analysis using ρ_l determined in Step 3. Verification of the 'near-optimal' performance sought in Step 1 and subsequently modified in Steps 2, 3 is of particular importance for the isolation system; the deformation capacity of isolation and energy dissipation devices should be checked for ultimate deformations (also accounting for residual displacements [20]), uplift, and stability. With regard to piers, it should be verified that the deformation demand is consistent with the limit state values allowing for controlled inelastic response of piers under EQIV calculated using section analysis; detailing of piers for confinement, anchorages and lap splices, should be carried out with due consideration of the expected level of inelasticity [7]. Moreover, shear design of piers should be carried out for seismic actions corresponding to this PL [7] and UB-DP. Finally, in case the selected isolation devices are not supported by technical approvals provided by the manufacturer, checking of stresses in reinforcing shims (internal plates) and design of end plates should also be performed during this step (e.g. [17]).

3. Pilot case study

To evaluate the proposed procedure a 3-span bridge of total length L=99m was selected (Fig. 2); the 10m wide prestressed concrete box girder deck has a 7% longitudinal slope and it is supported by two single column piers of cylindrical section and heights of 5.9 and 7.9m. The deck rests on piers and abutments through isolators allowing movement of the deck in any direction. The bridge rests on firm soil and both piers and abutments have surface foundations (footings). The selected structure is similar to the T7 Overpass previously used by the authors as a case study of Def-BD for bridges of ductile behaviour [7]. Apart from the modification of the pier-to-deck connection (monolithic in T7) the clear height of the piers is reduced herein to accommodate the pier cap (height of 1.5m). For the sake of consistency and with a view to enabling a meaningful comparison with T7, certain design parameters were defined in line with [7]. In particular the EC8 'Type 1' elastic spectrum (T_r=475 yrs) for a PGA of 0.21g was the basis for seismic design (i.e. EQIII), corresponding to subsoil class 'C' and assuming T_D=4.0s (i.e. compatible with Eqs. (4, 5)), whereas EQII and EQIV were selected as half and twice the spectrum of EQIII, respectively. Furthermore, the output of the Def-BD methodology in [7] regarding the geometry of the piers (i.e. D=1.2m) was used as a starting point, focusing on the transverse response of the bridge and ignoring SSI effects. Response-history analysis was carried out using Ruaumoko 3D [23].

The options described in the following are representative of just a few of the available isolation schemes and design criteria that can be explored (Section 2). During Step 1, Eqs. (4, 5) were used to plot inelastic spectra in the form of Fig. 3 corresponding to different intensities (EQIII, EQIV) and an isolated deck mass (m) of 2545tn; similar spectra were plotted for different combinations of ξ, η, T_p and PGA representing different isolation schemes under various PLs. In Fig. 3, the $\ddot{U}(opt)$ curve represents a visualisation of the design criterion of minimum $\ddot{U}$ per T_p, while $u(opt)$ indicates the corresponding relative displacements of the isolation system. $\ddot{U}opt$ and $uopt$ under EQIII and EQIV are presented for selected T_p values in Table 1 (in blue); the upper part of the table includes also the peak responses (in black) under EQIV of the SDOFs (optEQIII) that were optimally selected for the EQIII event, while the lower part includes the peak responses under EQIII (in black) of the optEQIV SDOFs (all derived from Eqs. (4, 5)). In the last 2 columns non-optimal responses are compared with their optimal counterparts. It is seen that the isolated system with optimally selected $\overline{v}_0$ under EQIII (optEQIII) will result in larger u (~50%) and $\ddot{U}$ (~10%) demand when subjected to EQIV, compared to the response of a system designed with the same ξ, T_p, but with $\overline{v}_0$ aiming at the minimisation of $\ddot{U}$ under EQIV. On the other hand, an optEQIV system subjected to EQIII, will result in ~35% smaller u, and ~10% larger $\ddot{U}$ (and hence base shear) demand compared to an optEQIII system. In a design context, the first approach entails an increased required deformation capacity of isolation and energy dissipation devices, while the second results in increased ρ_l ratios in the piers. On the other hand, the decision on the reference event has no effect on systems of ξ>0.05 and η=0, due to their inherent linearity. In general, the adopted approach should be based on the evaluation of data in the form of Table 1 for different isolation schemes accounting for both economy and market availability of materials and devices. Herein, EQIV was set as the reference event and two different isolation schemes were investigated. The first of ξ=0.05, $\overline{v}_0$ =0.046, T_p=3.0s (No. 2.2 in Table 1) materialised by LRBs, and the second of ξ=0.25, $\overline{v}_0$ =0, T_p=2.5s (No. 2.4) materialised by the combined use of LDRBs and linear VDs. The first scheme was selected on the grounds that the distributed base shear ($m\ddot{U}$) to the piers resulted in pier reinforcement ρ_l larger than the minimum required, and the second as an alternative approach resulting in similar u under EQIV (considering LB-DP) and $m\ddot{U}$ under EQIII (and UB-DP). It is noted that Eqs. (4, 5) can be applied irrespective of the properties considered (UB or LB) provided that the relevant input (i.e. ξ, $\overline{v}_0$, T_p corresponding to the considered DPs) is used.

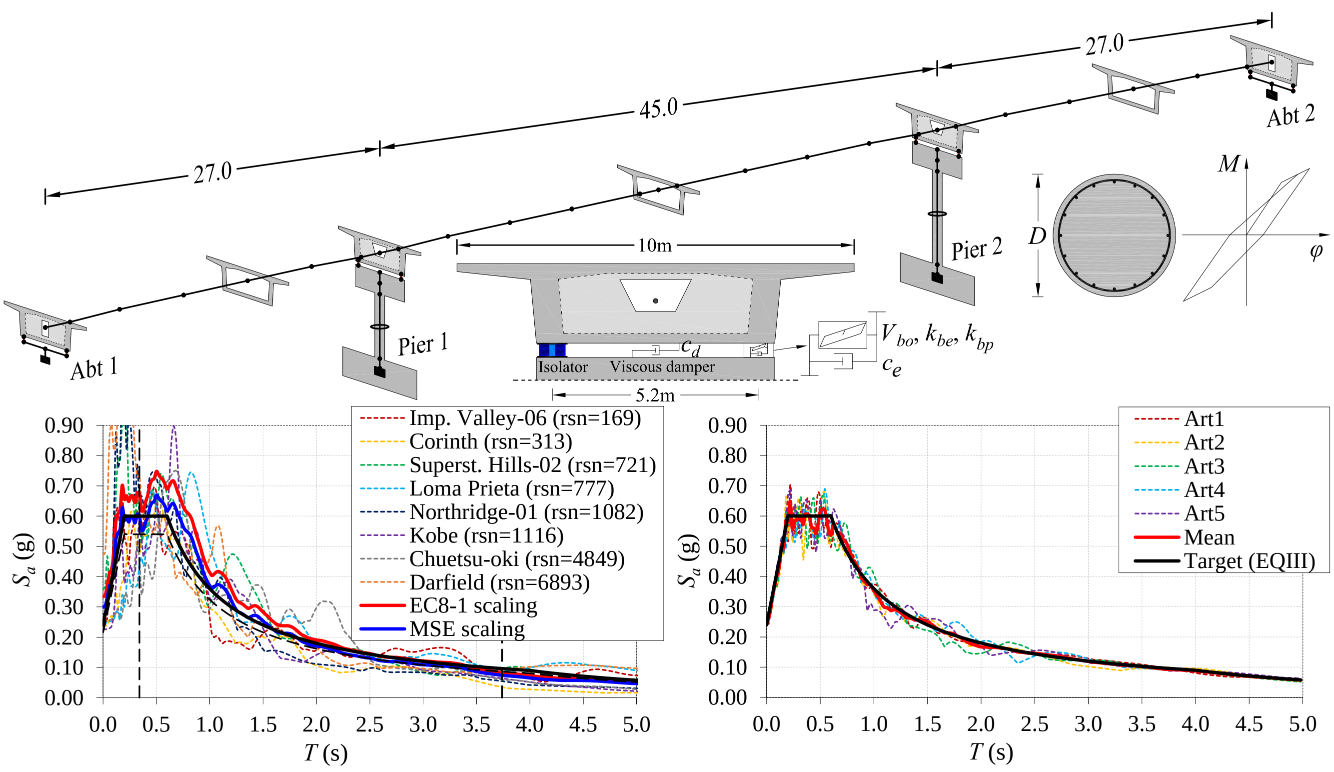

Fig. 2 – Configuration and modelling of studied bridge (top), and spectral matching of the scaled mean response spectrum to EQIII for a suite of natural (bottom left) and artificial recordings (bottom right)

The required characteristics of isolation devices were defined considering an isolation system with the properties of Table 1 and assuming LB-DP. Due to the relatively small length of the studied bridge it was deemed appropriate to use 8 identical isolators (i.e. 2 per abutment/pier) and 4 identical VDs (i.e. 1 per abutment/pier); it is recalled that only the transverse response is addressed herein (normally, VDs will also be provided in the longitudinal direction). In the LRB system, the required strength (V_0= 0.046mg) was distributed among pier and abutment isolators (V_{b0}), hence providing the required diameter of the lead core (D_L), by considering that the yield stress of lead (f_{Ly}) in abutment bearings is 25% lower than f_{Ly} in pier bearings to account for the low confinement of lead due to smaller vertical loads. The diameter of the isolators (D_b) was defined assuming an allowable vertical stress $\sigma_{v,max}$=12MPa, while the required $k_p=4\pi^2 m/T_p^2$ of the isolation system was evenly distributed to all isolators ($k_{bp}=k_p/8$) providing the required height of the elastomer $t_R=\pi G(D_b^2- D_L^2)/(4k_{bp})$, where G is the shear modulus of the elastomer. By adopting LB values of G_{LB}=0.77MPa [9], and $f_{Ly,LB}$=10MPa for pier bearings, D_b=0.75m, D_L=0.145m, t_R=0.234m were specified. Similar considerations provided D_b=0.75m, t_R=0.165m in the case of LDRBs. Assuming that ξ_e=5% is provided by the elastomer of LDRBs ($c_e=4\pi m\xi_e/T_p$), the LB damping coefficient of VDs was defined by considering ξ_d=20% and equating the energy/cycle of the 4 dampers to the energy/cycle of the single damper as $c_{d,LB}=(4\pi m\xi_d/T_p)/4$ (i.e. per damper). UB-DP were calculated based on the previous properties of devices and G_{UB}=1.12MPa, $f_{Ly,LB}$=22.5MPa [9], $c_{d,UB}=1.35c_{d,LB}$ (i.e. ±15% variability of the nominal c_d).

Using the LB- and UB-DP of isolators and VDs in Eqs. (4, 5), the displacement and shear response shown as Step 1 in Table 2 was calculated. Shear forces per abutment and pier (V_i) were calculated according to Eq. (6) that assumes a rigid horizontal movement of the deck accounting for both the hysteretic part of the isolator (1st term) and the damping forces due to the elastomer of the isolators and the VDs (2nd term). Use of UB shear forces along with estimated values for h_{eq} of pier columns (based on preliminary analysis) within the procedure described in [7] provided an estimate for the required pier strength associated with an allowable 'serviceability'-related concrete strain (i.e. 3.5~4.0‰). Table 2 summarises the adopted reinforcement ratio ρ_l and the corresponding pier yield moments (M_y) defined through M-φ analysis carried out using RCCOLA.NET [24] and considering a minimum transverse mechanical reinforcement ratio (ρ_w) for limited ductile bridges [9].

$$V_i = 2\left(V_{b0,i} + k_{bp,i}u\right) + \left(m\ddot{U} - 2\sum_{i=1}^{4}\left(V_{b0,i} + k_{bp,i}u\right)\right)\Big/4 \tag{6}$$

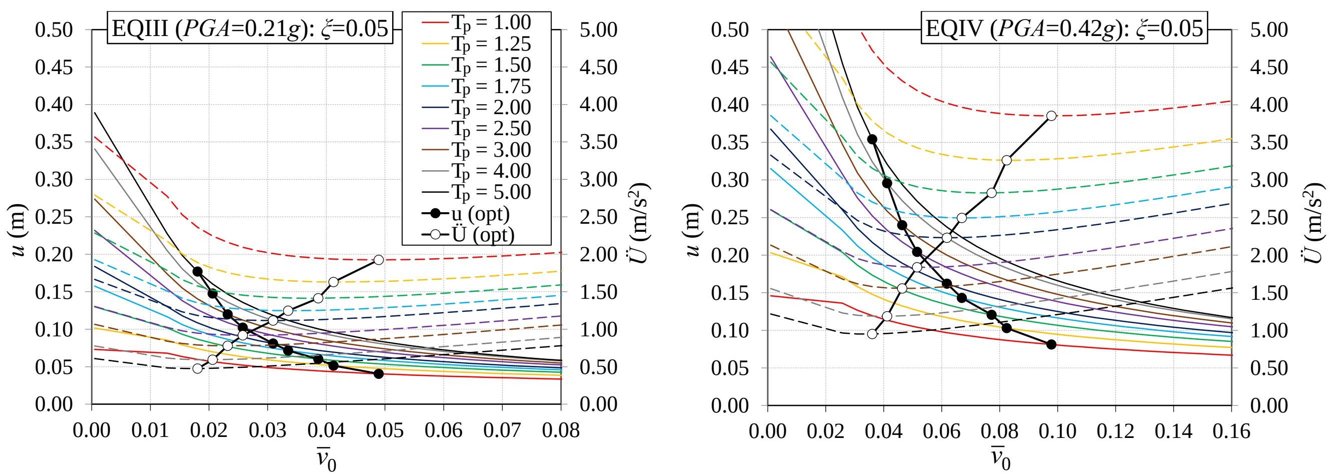

Fig. 3 – Peak (absolute) relative displacements (u) (solid lines), peak (absolute) total accelerations ($\ddot{U}$) (dashed lines), optimal peak (absolute) total accelerations ($\ddot{U}opt$) and corresponding relative displacements ($uopt$) of deck under EQIII (left) and EQIV (right) plotted for ξ=0.05 and different values of $\overline{v}_0$ and T_p

In Step 2, eligible records were selected from the PEER NGA-West2 database [25] excluding records containing long velocity pulses. Adopted preliminary search criteria were magnitude M_w=6.5~7, closest distance to the ruptured area R_{rup}=10~25km, and average shear wave velocity $V_{s,30}$=180~360 m/s (Ground Type C). The sample of eligible events was further constrained by assessing the similarity of spectra of the selected records to the target spectrum over the period range of (0.2~1.5)T_{eff} [9] quantified by the mean-squared-error (MSE) of the differences between the spectral accelerations (S_a) of the record and the target spectrum (http://ngawest2.berkeley.edu/). A suite of 8 eligible pairs of records was finally selected resulting in the spectral matching depicted in Fig. 2 for EQIII (scaling factor SF=1.20 using H1-components) when scaled according to the procedure of EC8-1 [7], [14]. A PIM of the structure was subsequently set up; the strength and stiffness of pier columns and LRBs were modelled using the modified Takeda (α=0.5, β=0) and the bilinear inelastic hysteresis, respectively [23], in line with the output of Step 1. The system damping matrix was assembled from the damping matrices of the different subsystems [6]; a stiffness proportional damping matrix [26] was adopted for the structural members of the bridge, whereas dashpot members were used to model viscous damping resulting from the elastomer of isolators and VDs. NLRHAs of the bridge were performed under the selected suite of records scaled to the intensity corresponding to EQII (Step 2) and subsequently to EQIII (Step 3); results representing the mean response are given in Table 2. 'Operationality' and 'minimal-damage' verifications included specific limits for u_{res} of bilinear isolators (estimated according to [19]) and γ_q (=u/t_R). Analyses for the LRB scheme under EQII and EQIII were based on UB-DP since during the selection of bearings in Step 1, k_{bp} and D_b (related to $\sigma_{v,max}$), rather than limit-state strains, were found to control t_R. UB-DP facilitated an explicit (UB-E) calculation of u_{res} but required an implicit calculation of LB deformations (LB-I) that are critical in checking γ_q. A modification factor (MF) equal to u_{LB}/u_{UB}=0.077/0.045 (derived from Eq. (4) and EQIII) was used to implicitly estimate LB deformations from UB analysis results. Although not required by the suggested procedure, γ_q calculated by explicitly considering LB-DP (LB-E) are also provided in Table 2 (in grey). The restoring capability was not checked in the LDRB+VD scheme, thus, LB-DP properties were adopted under EQII. None of the isolation system verifications were found to be critical in Steps 2, 3; similarly, pier strength requirements under EQIII were lower than those of Step 1, hence, it was deemed appropriate to proceed to Step 4 without further modifications.

Table 1 – Comparison of peak responses among SDOF systems optimised for different earthquake intensities

No.	ξ	$\bar{v}_0$	T_p (s)	η	u (m)	$\ddot{U}$ (m/s^2)	η	u (m)	$\ddot{U}$ (m/s^2)	Δu (%)	$\Delta\ddot{U}$ (%)
	opt EQIII			opt EQIII			EQIV			(EQIV - optEQIV) / optEQIV	
1.1	0.05	0.031	2.00	0.60	0.081	1.116	0.30	0.236	2.463	45.5	10.4
1.2	0.05	0.023	3.00	0.45	0.120	0.779	0.23	0.367	1.727	53.2	10.8
1.3	0.05	0.018	5.00	0.35	0.177	0.476	0.18	0.552	1.041	55.8	9.4
1.4	0.25	0.001	2.50	0.01	0.121	0.871	0.01	0.241	1.741	0.0	0.0
	opt EQIV			EQIII			opt EQIV			(EQIII - optEQIII) / optEQIII	
2.1	0.05	0.062	2.00	1.20	0.056	1.231	0.60	0.162	2.231	-31.3	10.4
2.2	0.05	0.046	3.00	0.90	0.077	0.855	0.45	0.240	1.558	-35.4	9.7
2.3	0.05	0.036	5.00	0.70	0.106	0.535	0.35	0.354	0.951	-40.3	12.4
2.4	0.25	0.001	2.50	0.01	0.121	0.871	0.01	0.241	1.741	0.0	0.0

Analysis under EQIV in Step 4 was carried out in both schemes for UB-DP for the reason stated in Step 3, thus enabling an explicit calculation and verification of the response in the substructure. Shear design (rather than confinement requirements) was found to control transverse reinforcement ρ_w in piers while the ultimate curvature (φ_u) in pier sections was found somewhat lower than the value corresponding to ε_{cu}=4.0‰ verifying the target performance set in Step 1 for controlled inelastic response of the substructure. UB-DP were also used to check tensile stresses (σ_t) of isolators [17], while verifications of bearing strains due to vertical compression (γ_c) and lateral deformations (γ_q), and bearing stability (ratio of the buckling load under compression and lateral deformation P'_{cr} to the peak compressive force $N_{b,max}$ per bearing) [17], required an implicit estimation of $u_{i,LB}$ (Table 2, MF=0.239/0.134); the latter was conservatively [19] increased by $u_{i,res}^5$ to account for the accumulation effect of 5 past EQIII events in the LRB scheme. Isolator demands were found to lie within the adopted limits, with the stability criterion on LRBs being the most critical one, while tensile stresses were kept in any case below the stress corresponding to cavitation [17]. It is noted that an attempt to reduce t_R aiming to match more closely the strain limits of Table 2 is obstructed by $\sigma_{v,max}$ (i.e. a reduction of t_R requires a reduction of D_b to obtain a target k_{bp}) which does not pose a strict limitation nor is it a code requirement (inasmuch as total strains γ_{tot} lie within allowable limits) but it is considered good common practice and is typically recommended by manufacturers. The required force capacity of VDs ($N_{d,max}$) was also estimated in this step as 774 kN; the designer may choose to use more than one dampers per abutment/ pier location in order to reduce $N_{d,max}$ without affecting analysis results.

Assessment of the designs was carried out to evaluate the efficiency of the proposed procedure for the 3 different PLs and the considered range of DP of devices (i.e. LB, UB), accounting for all possible combinations of isolator properties and PLs. Since the primary objective of the assessment was the study of the transverse response of the bridge under a seismic excitation that matches as closely as feasible the 'design excitation' (i.e. the design spectrum), NLRHAs were performed for two different suites of records (Fig. 2); the first consisted of 5 artificial records used to develop the design equations in Section 2, generated [27] to fit the design spectra associated with the EQIII PL, and scaled appropriately when a different PL was considered, and the second consisted of the natural records used during design, each one scaled to minimise MSE (Fig. 2) over the period range of interest (i.e. different *SF* per record). M-φ analyses based on final detailing of reinforcement were also performed for each pier critical section.

Table 2 – Comparative evaluation of Def-BD for two different isolation schemes

Scheme				LRBs				LDRBs + VD				Design Criterion
Step	EQ	Response	DP	Abt1	Pier1	Pier 2	Abt 2	Abt1	Pier1	Pier 2	Abt 2	
1	IV	u (m)	LB	0.239	0.239	0.239	0.239	0.241	0.241	0.241	0.241	
		V (kN)		958	1040	1040	958	1108	1108	1108	1108	
		u (m)	UB	0.134	0.134	0.134	0.134	0.192	0.192	0.192	0.192	
		V (kN)		1199	1385	1385	1199	1252	1252	1252	1252	
		ρ_l (‰)		-	6.08	12.15	-	-	4.34	9.55	-	> 2.5
		M_y (kNm)		-	4666	5867	-	-	4341	5343	-	
2	II	u_{res} (m)	UB-E	0.008	0.007	0.006	0.008	-	-	-	-	< 0.015
		γ_q (%)	LB-I	26	19	15	37	-	-	-	-	< 100
			LB-E	22	20	19	26	40	40	38	42	< 100
3	III	u_{res} (m)	UB-E	0.009	0.009	0.009	0.008	-	-	-	-	0.015-0.030
		u_{res}^{5} (m)		0.010	0.009	0.010	0.009	-	-	-	-	
		γ_q (%)	LB-I	64	57	52	77	85	82	77	89	100-150
			LB-E	51	49	47	56	81	79	76	84	
		M_y (kNm)	UB-E	-	4492	5236	-	-	3243	3535	-	
4	IV	u (m)	UB-E	0.195	0.174	0.167	0.226	0.223	0.205	0.191	0.235	
			LB-I	0.348	0.310	0.297	0.404	0.282	0.258	0.241	0.296	
		u_{acc} (m)	LB-I	0.357	0.319	0.307	0.413	-	-	-	-	
		γ_q (%)		153	136	131	176	171	156	146	179	< 250
		γ_c (%)		133	272	272	181	112	240	237	125	
		γ_{tot} (%)		286	408	403	357	282	397	383	304	< 700
		$P'_{cr}/N_{b,max}$		2.71	1.33	1.33	2.00	5.46	2.53	2.57	4.89	> 1.10
		u (m)	LB-E	0.289	0.279	0.269	0.301	0.267	0.255	0.244	0.277	
		σ_t (MPa)	UB-E	0.62	-	-	0.98	0.66	-	-	0.94	< 2.3 (3G)
		$N_{d,max}$ (kN)		-	-	-	-	748	724	675	774	
		V (kN)		1386	1434	1375	1515	1318	1300	1256	1383	
		ρ_w (‰)		-	8.10	7.56	-	-	7.09	7.09	-	5.15
		φ_u (m^{-1})		-	0.0082	0.0069	-	-	0.0072	0.0072	-	0.0084
A	II	u_{res} (m)	UB-E	0.006	0.005	0.004	0.007	-	-	-	-	< 0.015
		γ_q (%)	LB-E	16	13	12	18	36	35	34	38	< 100
	III	u_{res} (m)	UB-E	0.008	0.008	0.007	0.008	-	-	-	-	0.015-0.030
		γ_q (%)	LB-E	37	35	33	39	72	71	68	75	100-150
	IV	u (m)	LB-E	0.265	0.258	0.248	0.275	0.239	0.232	0.223	0.248	
		γ_{tot} (%)		199	332	331	214	224	347	346	237	< 700
		$P'_{cr}/N_{b,max}$		4.42	1.66	1.63	3.93	7.52	2.87	2.83	6.80	> 1.10
		σ_t (MPa)	UB-E	-	-	-	0.09	0.21	-	-	0.48	< 2.3 (3G)
		$N_{d,max}$ (kN)		-	-	-	-	670	658	620	694	
		V (kN)		1167	1223	1299	1245	1177	1191	1198	1234	
		φ_u (m^{-1})		-	0.0040	0.0036	-	-	0.0052	0.0044	-	0.0084

In Table 2, selected results are provided (denoted as Step A) for the LRB and the LDRB+VD scheme assessed using the suite of artificial and natural records, respectively. Regarding the reliability of the design procedure, the design was found to be safe, in that it satisfied the limit-state criteria associated with each PL since the deformation demand derived from assessment was in general lower than that derived at the design stage. This is attributed to a certain degree of conservatism introduced when all recordings are scaled using the same *SF* in accordance with the procedure prescribed by EC8 [14] as opposed to the approach adopted during assessment

(Table 2, u: 4/A-IV-LB-E), and to the implicit (and conservative) estimate of deformations during design (u: 4-IV-LB-I/E); the latter can be eliminated with an explicit calculation of u in Step 4, noting however, that the implicit approach does not necessarily result in overdesigning members and devices. In this context, deformations derived from assessment are closer to the values estimated during Step 1 somewhat increased due to torsional effects of the deck, the elimination of which would require devices of different properties at each pier and abutment location, an approach not justified in small-to-moderate bridges. Shear forces (explicitly calculated) were found to be close to the values obtained during design.

In summary, both systems exhibited similar performance satisfying all adopted design criteria under the studied PLs with the LDRB+VD scheme resulting in relatively lower isolator and deck displacement demand, and lower reinforcement demands ρ_l (23.8%) and ρ_w (9.5%) in piers. The final decision on the scheme to be adopted should account for the cost and availability of relevant materials and devices. Last but not least, both isolation schemes resulted in reductions in pier ρ_l (12.5% in the LRB and 33.3% in the LDRB+VD scheme) and ρ_w (31.9% in the LRB and 38.3% in the LDRB+VD scheme) compared to the design of the T7 Overpass (i.e. 'ductile' pier response [7]), indicating that higher performance objectives adopted in isolated bridges do not necessarily result in higher initial cost of substructure design when the isolation system is quasi-optimally selected.

4. Conclusions

A deformation-based design procedure initially proposed for bridges designed for ductile behaviour of piers was extended herein to seismically isolated bridges aiming at efficient structural design for multiple PLs, via the control of a broad range of design parameters and with the aid of advanced analysis tools. A key issue in this extension was the identification of the critical PL and the comparative evaluation of different isolation schemes at the early stages of design, thus providing the designer with the quantitative tools required to select a 'near-optimal' isolation system in terms of both economy and structural performance, also accounting for various design criteria. Further issues addressed involved the realisation of the selected scheme through base isolation and energy dissipation devices along with the treatment of the variability of their DPs, the proper consideration of the intended plastic mechanism of the substructure under the relevant PLs, and the direct (non-iterative) estimation of seismic demand during preliminary design. The validity of the procedure was investigated by applying it to the transverse direction (biaxial excitation is currently under investigation) of a bridge previously used by the authors to develop the Def-BD method for 'ductile' bridges. The following conclusions were drawn from the pilot study presented herein:

- 'Life-safety' verifications under EQIV governed in general the bridge design in both isolation schemes considered. More specifically, stability considerations and allowable vertical stresses were found to control the characteristics of the isolators, while the requirement for controlled inelastic response and shear forces controlled reinforcement ρ_l and ρ_w in piers. Assessment of the design by NLRHA using suites of records closely matching the design spectrum associated with each PL, revealed that the suggested procedure predicted well the structural response while resulting in safe design in the sense of respecting the adopted design criteria.
- Among the isolation schemes investigated, i.e. LRB and LDRB+VD, the second resulted in relatively lower seismic demand (for the bridge type and seismic scenario considered herein). In addition, both schemes resulted in lower pier reinforcement ratios compared to the design for ductile response, indicating that cost reductions in substructure design of optimally selected isolation systems may be able to compensate for the initial cost of the isolation system, thus rendering base-isolation an appealing design alternative.
- Def-BD of seismically isolated bridges requires design-equations in the form of Eqs. (4, 5) that can be provided for code-prescribed target spectra using the procedure proposed herein. Although development of regression models is required in cases wherein spectra of different frequency content are adopted, the procedure can be easily automated based on a relatively small number of NLRHAs under spectrum compatible records.

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