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Citation: Camara, A. & Astiz, M. A. (2014). Analysis and Control of Cable-Stayed Bridges Subject to Seismic Action. *Structural Engineering International*, 24(1), pp. 27-36. doi: 10.2749/101686614x13830790993762

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Analysis and Control of Cable-Stayed Bridges Subject to Seismic Action

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DOI: 10.2749/101686614X13830790993762

Abstract

Cable-stayed bridges are key points in transport networks and at present one of the most challenging structures for the civil engineering community. The integrity of these bridges should be guaranteed even under extremely large earthquakes. This paper begins with a discussion of the advantages of a new non-linear static “Pushover” procedure that includes the three-dimensional contribution of the governing vibration modes. The efficacy and the accuracy of the proposed Pushover in the non-linear seismic analysis of bridges with significant coupling between the towers, deck and cable system is verified. In the second part of this paper, the seismic responses of several cable-stayed bridges have been studied, verifying the influence of the tower shape, cable arrangement and the main span length on the structural behaviour under strong ground motions. Severe damage is identified at critical tower sections by means of extensive non-linear dynamic analyses. Finally, retrofit solutions with viscous dampers (VDs) and yielding metallic dampers (MDs) connecting the deck and the tower in the transverse direction are explored. The proposed connection with dampers effectively prevents yielding of the reinforcement and cracking in the tower legs.

Keywords: cable-stayed bridges; nonlinear seismic behaviour; retrofit; dampers; pushover analysis; tower shape; main span.

Introduction

Cable-stayed bridges have crucial importance in transport networks. Their failure due to natural hazards such as large earthquakes would lead to substantial social and economic losses. These structures are markedly flexible and present a reduced number of intermediate supports that would, in principle, imply a favourable response under seismic excitation. However, the combination of high flexibility and low inherent damping,¹ together with the modal coupling between the deck, towers and cable system² that usually accompanies these bridges, can significantly complicate their dynamic response.

The seismic behaviour of cable-stayed bridges received the attention of the academic community in recent years, with key contributions from Refs. [2,3]. Most of the studies on cable-stayed bridges are focused on specific phenomena, such as cable

vibrations,^{3,4} spatial variability of the seismic excitation⁵ and the use of seismic devices.⁶ Few studies are focused on the response of cable-stayed bridge towers to seismic effects. The towers play a paramount role in the global integrity of the structure and should survive large earthquakes.⁷ Guidelines on the conceptual design of the towers of cable-stayed bridges to withstand seismic ground movements have been provided in Ref. [8]. The seismic response of a relatively small bridge (284 m span) with metallic towers for three different shapes has been studied in Ref. [9]. However, there is a need for parametric studies on the seismic behaviour of cable-stayed bridge towers with different dimensions and cable arrangements. The present paper is focused on two essential aspects of the project of cable-stayed bridges in seismic areas that are relevant to engineers: (1) the inelastic analysis and (2) the design of the towers and their connection with the bridge deck.

The selection of the analysis strategy is an important step that should be decided in accordance with the relevance of the structure, the seismic risk and the stage of the construction project. There are several analytical strategies

available for designers to study the seismic behaviour of structures in the elastic¹⁰ and inelastic ranges,¹¹ with different levels of accuracy and associated computational costs. The most rigorous procedure when large inelastic response is expected is the non-linear response history analysis (NL-RHA). Unfortunately, the computational cost associated with this procedure is significant and often not justified at the early stages of the project. In such cases, non-linear static procedures (Pushover) may represent an ideal solution. The basic concept behind the Pushover analysis is the static application of an incremental loading up to a given target displacement employing a load pattern aimed at representing the distribution of inertial forces during an earthquake. Pushover methods help uncover structural weaknesses that may remain hidden during the elastic seismic analysis. Besides, the static approach can estimate the peak seismic demand in non-linear range with reduced computational cost.¹² Codes and design guidelines^{13–15} in Pushover procedures are generally based on the assumption that the fundamental vibration mode governs the structural response, which is far from accurate in the case of cable-stayed bridges. The modal pushover analysis (MPA) to consider the contribution of several vibration modes has been proposed in Refs. [16,17], but the three-dimensional (3D) nature of the earthquake and the possible modal couplings (characteristic of cable-stayed bridges) were not considered. Recent advanced Pushover strategies seek to overcome these limitations.¹¹

Analysis strategies are tools that provide information to design the structure or to assess its response. In terms of seismic design, it is widely recognised that the traditional force-based method fails to accomplish the goals of modern performance-based earthquake engineering. Instead, the direct displacement-based design (DDBD) method defines the structure to satisfy the performance limit displacement by explicitly considering the non-linear

Cite as:

Camara A and Astiz MA (2014). Analysis and control of cable-stayed bridges subjected to seismic action. *Structural Engineering International*. **24**(1): 27-36, DOI: 10.2749/101686614X13830790993762.

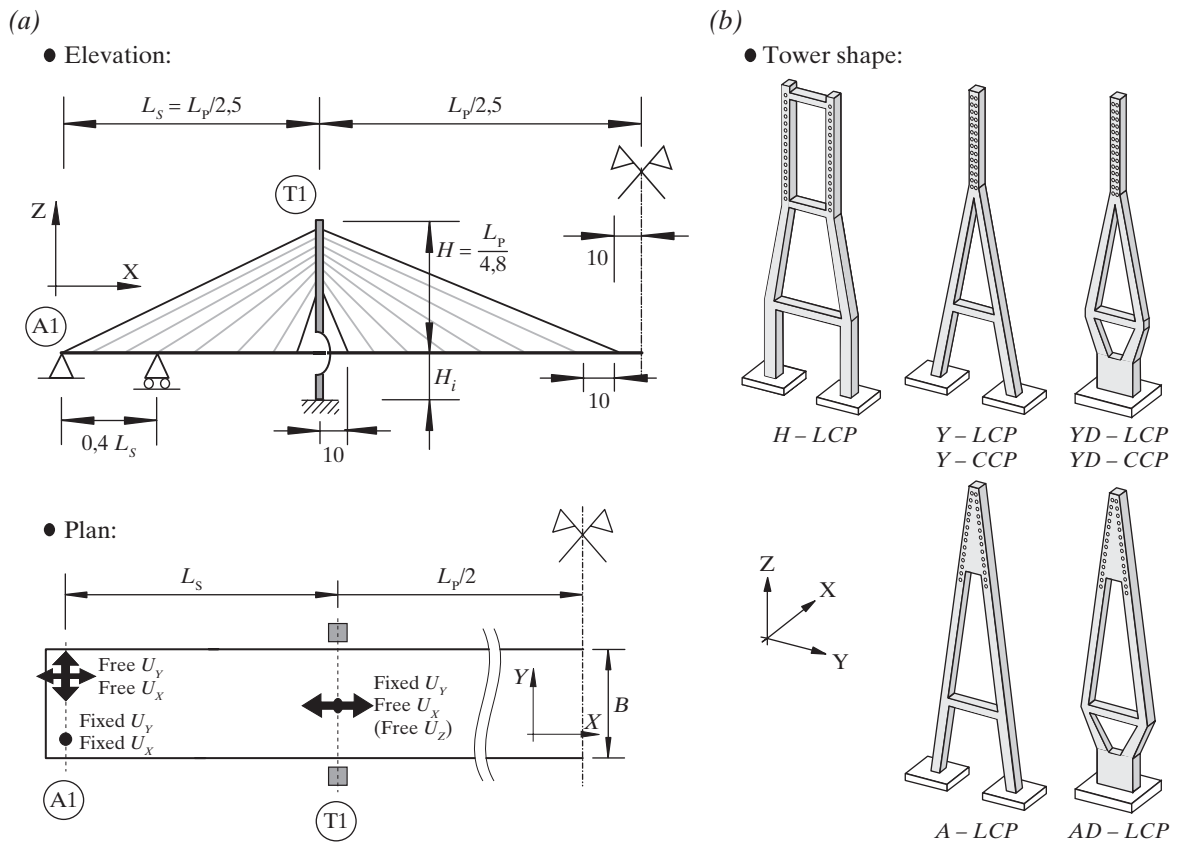


Fig. 1: (a) Schematic bridge elevation and plan with the support conditions (units in metres), the deck width $B = 25$ m; (b) types of towers considered and corresponding keywords

response.¹⁸ One of the key factors in the dynamic behaviour of a cable-stayed bridge is the connection between the deck and the towers.⁶ The current trend in the design of cable-stayed bridges in earthquake-prone areas is to incorporate seismic devices in the deck-tower connection. The objective is to concentrate or reflect the seismic force and to help maintain the towers in elastic range during the earthquake (e.g. Rion-Antrion^{19,20} or Sutong²¹ bridges, among others). Recently, the DDBD has been employed to design linear viscous fluid dampers (VD) that control the longitudinal response of cable-stayed bridge towers.⁸ The interaction between the towers and the deck in transverse direction is also of interest as has been shown by numerical studies.^{10,22} Furthermore, one of the few reported structural problems arising from ground shaking in a real cable-stayed bridge is the severe spalling of the Chi-Lu concrete tower in transverse direction.²³ In the second part of this paper, it is shown that a rigid connection between the deck and the tower in the transverse direction may lead to considerable cracking in the tower legs. The interest here is focused on how design decisions may affect the seismic behaviour of cable-stayed bridges.

The third and final part of the paper proposes solutions to minimise the dissipation of seismic energy through structural damage to the towers. These solutions are conceived as retrofit designs, maintaining the original towers and modifying exclusively the transverse deck-tower connection. Conventional VDs with a fuse restrainer (FR) are employed in the connection, and the solution is compared with that obtained with yielding MDs in the same position. MDs have been successfully proposed in building frames,²⁴ and its applicability to cable-stayed bridges is explored in this paper.

Proposed Structures and Seismic Action

A large number of canonical cable-stayed bridges with two concrete towers and main spans (L_p) ranging from 200 to 600 m have been studied. The sections and proportions of the proposed structures are parametrically defined in terms of the main span (L_p) and are borrowed from a previous compilation of the dimensions of cable-stayed bridges constructed worldwide. Figure 1a presents the elevation and plan of the bridges studied. The connection of the deck with

the abutments follows the configuration depicted in Fig. 1a, while the deck-tower connection is floating and exclusively constrains the transverse relative movement (in Y axis). The intermediate piers in the side spans only prevent the vertical movement of the deck. To cover the wide range of possibilities in the design of cable-stayed bridges, different tower shapes (shown in Fig. 1b), cable layouts (one central cable plane, CCP, or two lateral cable planes, LCP) and foundation soil conditions (rocky soil, TA, and soft soil, TD) have been considered.

Two sets of 12 synthetic far-field accelerograms are imposed to the structure supports in the three directions (X , Y , Z). Each set matches the rocky and soft soil Eurocode EN1998-1²⁵ design spectra with a ground acceleration of 0,5 g, representative of seismic-prone areas worldwide. The duration of the artificial records is 20 s, and their seismological features (strong pulse interval and Arias Intensity, among others) were validated through empirical models based on natural records of the PEER-NGA database.²⁶ The spatial variability of the earthquake could have a significant impact on long structures such as cable-stayed bridges due to the loss of synchronism of the

seismic action between both towers.^{2,5} A previous study on the proposed structures with several wave propagation velocities was conducted to assess the importance of this effect on the tower design.²² It was verified that in the tower anchorage area, the peak longitudinal response (X axis) under asynchronous excitation is larger than that obtained when the same ground motion was considered to be synchronous in all the supports. The increment in the response under asynchronous excitation varies with the span but the variation is generally below 20%. This effect is less noticeable in the transverse direction. Consequently, the spatial variability of the earthquake is ignored in this study and both towers have the same response due to the symmetric conditions.

The accurate representation of the non-linear response of the towers is paramount in the study of cable-stayed bridges under large ground shaking. Due to the possible simultaneous stiffness degradation in transverse (Y) and longitudinal (X) directions, besides the large variation of the axial load during the earthquake, the conventional moment–curvature models are not recommended. Instead, the towers are simulated through the rigorous beam-type fibre model²⁷ in this study. The position of the steel longitudinal reinforcement bars and concrete fibres is defined at each node in the finite element (FE) model of the towers. The fibre model conveniently accounts for axial load variation on the seismic response, as suggested in Ref. [8]. The sections of the towers are hollow and strongly reinforced to confine the concrete at the tower base and the strut connections: the transverse reinforcement ratio is 0,8%. A Finite Element software²⁸ has been employed in this study.

Relevant Eurocodes were considered to define the linear and non-linear constitutive relations of the steel and concrete in the whole structure. Note that in this work, the deformation with negative sign represents compression, while a positive sign denotes tension. The concrete in the towers has a characteristic strength (f_{ck}) of 40 MPa. The concrete model includes softening if the normal compressive strain exceeds $\varepsilon_{cy} = -0,1\%$ and tension stiffening to simulate cracking. The stress and strain corresponding to crack initiation are $f_{c,crack} = 3,5$ MPa and $\varepsilon_{c,crack} = 0,01\%$, respectively, whereas the contribution of the concrete is assumed null beyond

$\varepsilon = 0,035\%$. Degradation effects due to cyclic loading are not included in the concrete. The reinforcement steel is set to capture yielding when the strain reaches $\varepsilon_{sy} = 0,26\%$ (related to the yielding limit $f_{s,y} = 552$ MPa). The transformation of the steel yielding surface because of cyclic loading is considered (e.g. the Bauschinger effect). The pre-stressing steel in the cables is Y-1770 (elasticity modulus $E_s = 195$ GPa).

Seismic Analysis Strategies in Non-Linear Range

Both NL-RHA and non-linear static (Pushover) procedures have been studied and compared. In both cases, the analysis begins with the deformed configuration of the bridge after the application of its self-weight and the pre-stress of the cable system.

In NL-RHA, the triaxial accelerograms (in X , Y , Z directions) are imposed at the supports, and the equation of motion is directly solved using the Hilber–Hughes–Taylor algorithm.²⁸ The analysis is repeated for each of the 12 independent accelerograms to obtain statistically meaningful results, and the time domain response is post-processed to extract the peak value. The average peak seismic response is finally obtained. Except where otherwise stated, the results presented in this work correspond to the average value of the peak response obtained with the set of 12 independent triaxial accelerograms. For comparison purposes, NL-RHA is considered as the “exact” solution.

A New Pushover Analysis for the Seismic Analysis of Cable-Stayed Bridges

The original MPA proposed in Refs. [16,17] has been included in FEMA-440¹⁵ and has recently been adapted for analysis of cable-stayed bridges.¹¹ Unlike simplified Pushover procedures included in most of the codes, the MPA considers the contribution of a set of important vibration modes in the structure. However, this method neglects the contribution of modes in directions other than the dominant one. *Figure 2* shows the first transverse vibration mode in one of the cases studied in which the transverse flexures of the towers and the deck are significantly coupled with vertical flexure and torsion of the deck in the central span. The original MPA considers this mode as purely transverse, and hence the significant contribution of the earthquake in vertical direction is ignored.

The objective of the new Pushover procedure described here is to account for the 3D nature of the important vibration modes in cable-stayed bridges. Like MPA, the proposed method considers the load pattern as the modal expansion of the excitation vector, but in this case, it is extended to the three dimensions. The load pattern of the n th-mode in direction j (with $j = X, Y, Z$) is represented in *Fig. 2* with components along the three axes. Consequently, the static incremental analysis is no longer bi-dimensional:

$$s_n^j = \Gamma_n^j m \phi_n \quad (1)$$

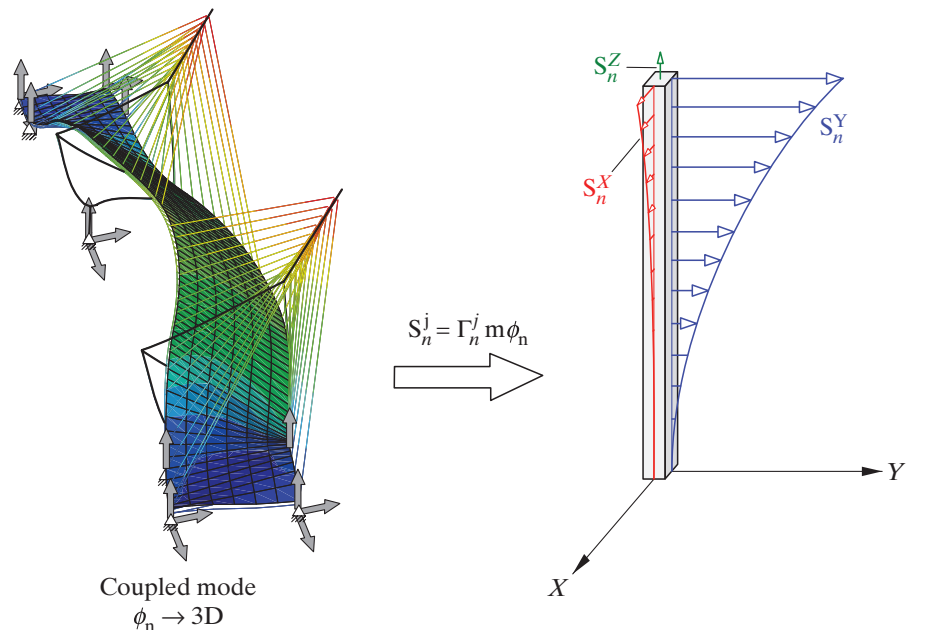


Fig. 2: Typical transverse mode coupled with vertical and torsional flexure of the deck, alongside the 3D load pattern proposed for the YD-LCP bridge with 200 m main span

where $\mathbf{s}_n^j$ and Γ_n^j are, respectively, the load pattern and participation factor (scalar) in direction j (where $j = X, Y, Z$) corresponding to the n th mode. $\mathbf{s}_n^j$ is a vector with dimensions $[N \times 1]$, with N being the number of degrees of freedom of the structure. $\mathbf{m}$ $[N \times N]$ is the mass matrix of the structure and ϕ_n $[N \times 1]$ is the n th mode shape.

The capacity curve relates the base shear with the displacement of the target point. Again, similar to other Pushover procedures, this curve is obtained in the incremental static analysis, but it has three normal components (X, Y, Z). The capacity curve of the n th mode describes the non-linear response of a single-degree-of-freedom (SDOF) system subject to an equivalent acceleration history. This equivalent accelerogram comes from the 3D definition of the ground motion and the modal participation factors of the structure:

$$\ddot{u}_{g,n}^*(t) = \Gamma_n^X \ddot{u}_g^X + \Gamma_n^Y \ddot{u}_g^Y + \Gamma_n^Z \ddot{u}_g^Z \quad (2)$$

where $\ddot{u}_g^j(t)$ ($j = X, Y, Z$) are the longitudinal, transverse and vertical components of the triaxial accelerogram representing the seismic excitation. The SDOF system response is integrated in time domain, and its peak displacement is considered the target displacement that defines the non-linear seismic demand in the n th mode.

It has been reported that the 3D extension of MPA significantly improves the accuracy of the original procedure in the analysis of large cable-stayed bridges.¹¹ This may be explained by the simultaneous contribution of the seismic excitation in the three dimensions in vibration modes with strong coupling between the deck and the towers, similar to the one shown in Fig. 2.

To obtain the total seismic response using MPA, it is assumed that the interaction between vibration modes in non-linear range is negligible. The modal contribution is simply superimposed through standard modal combination rules, such as those employed in the conventional elastic spectrum analysis. However, this is conceptually incorrect, as the tower damage caused by the longitudinal flexure inevitably affects the transverse response, and vice versa. This interaction is taken into account in the coupled non-linear static pushover (CNSP).¹¹ The load patterns of the most significant longitudinal and transverse vibration modes are combined in CNSP, and

only one incremental static analysis is conducted with the resulting load distribution. The procedure considers the non-linear interaction of the two governing modes (i.e. in-plane and out-of-plane). Contribution of other vibration modes is assumed purely elastic. This assumption leads to an estimation of the peak seismic response that is typically on the safer side, a definite advantage in the seismic design of any structure. Furthermore, CNSP reduces the computational time because only one incremental non-linear static analysis is conducted, while the MPA typically requires 10–15 static analyses for cable-stayed bridges.¹¹

The peak seismic response along the height of the tower obtained with CNSP is shown in Fig. 3: a good agreement with the “exact” NL-RHA solution is observed. If the response of the structure was strongly dominated by the governing transverse and longitudinal vibration modes (the ones that are combined in CNSP to obtain the load pattern), the accuracy of this procedure would be typically better than the MPA, because the mode interaction is considered. The figure shows that CNSP accurately estimates the peak transverse reaction of the deck against the towers. This reaction is responsible for large increase in the transverse shear force at the level of the deck–tower connection and causes significant damage, as will be discussed in the following sections. On the other hand, if different modes (apart from the governing ones) significantly contributed to the non-linear response,

CNSP would tend to over-predict the seismic response.

Advanced Methods versus Code-Compliant Pushover Analysis in Cable-Stayed Bridges

Pushover approaches in standards and guidelines reduce the structure to an inelastic SDOF system typically related to the fundamental vibration mode. Different load patterns are proposed in these documents, for example the “uniform” distribution proportional to the mass ($s_k = m_k$, where m_k is the mass associated with the node k of the model) and the “principal mode” distribution ($\mathbf{s} = \mathbf{m}\phi_1$, where ϕ_1 is the shape of the fundamental mode).

Figure 3 compares the peak transverse shear force in the tower along its height obtained with code-based methods employing the uniform and principal mode load patterns. The improvement in the results obtained with advanced pushover methods (MPA and CNSP) is clear in comparison with the simplified strategies proposed by different codes. This is especially true for the method based on the uniform load pattern where the distribution of inertial forces is not predicted realistically. The dominating transverse and longitudinal modes present sign reversals in their modal displacements along the tower height due to the constraint exerted by the tower geometry (in transverse modes) and the cable system (in longitudinal modes). The uniform load pattern ignores this important effect and leads

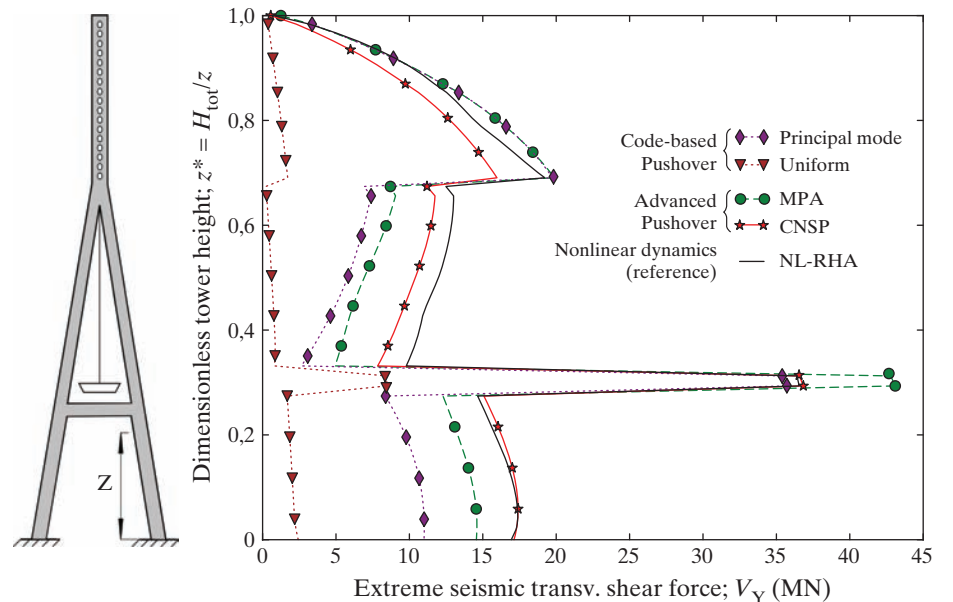


Fig. 3: Peak transverse shear (V_Y), obtained by means of different analysis procedures. Y-CCP model with main span $L_P = 400$ m. Soft soil category (TD)

to inadmissible peak forces that are up to 90% lower than the reference values (NL-RHA). The solution obtained with the method based on the principal mode is more accurate than that with the uniform load pattern as it properly accounts for the sign reversals of the inertial forces during earthquake. However, the principal mode method also underestimates the response in comparison with the advanced procedures where the contribution of vibration modes other than the fundamental one (ϕ_1) is considered. This advantage of the advanced pushover methods over the code-based methods employing the uniform and principal mode load patterns was observed in all the cable-stayed bridge models studied. However, it is also recognised that Pushover methods proposed in seismic codes and guidelines are not directly applicable to cable-stayed bridges. Eurocode 8 EN1998-2²⁹ discourages the Pushover procedure in structures with large percentage of mass concentrated in the piers, which is the case of the towers in cable-stayed bridges.

Seismic Response of the Towers

Advanced Pushover analysis can accurately predict the peak seismic response. However, the NL-RHA is more accurate. Therefore in the following sections, NL-RHA is considered to compare the non-linear response of the towers.

The objective now is to explore how the structural configuration of a cable-stayed bridge affects the seismic response of its towers. Figure 4 presents the peak deformation (positive in tension and negative in compression) recorded in the longitudinal reinforcement bars of the tower sections during an earthquake (including the compression caused by the self-weight) in different models. The elastic limits of compression and tension of the concrete and reinforcement steel are also included. Cracking is considered inadmissible if the reinforcement yields in tension ($\epsilon_{\text{tot}} > 0,26\%$).

Currently, the resistance of the structure to the peak seismic demand is not the only concern of the designer. The evolution of the energy balance during an earthquake and the percentage of the seismic energy that is dissipated as structural damage are also considered important. This study addresses both these important aspects. The following damage ratio summarises the accumulated structural damage in the towers

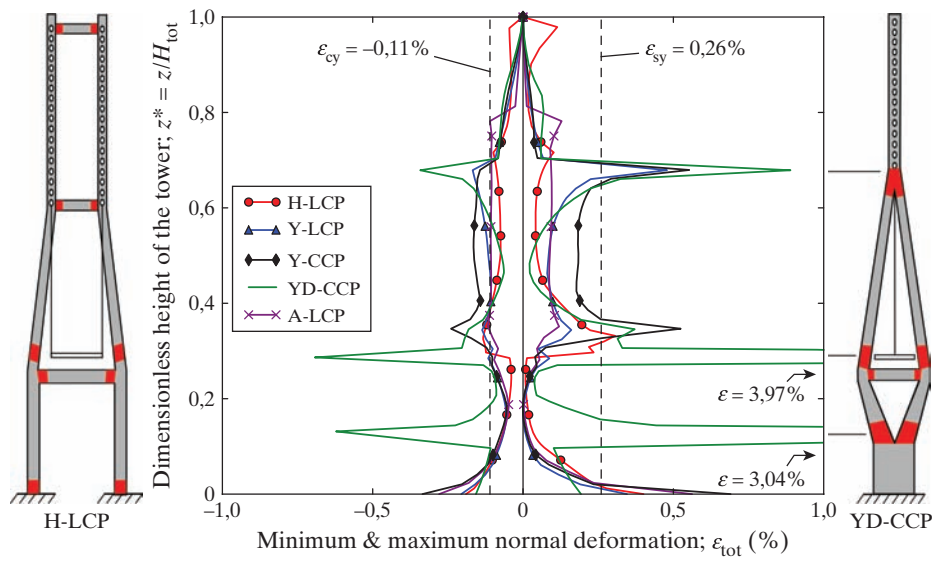


Fig. 4: Peak deformation in the reinforcement along the tower height for different cable-stayed bridges. The schematic representation of the reinforcement yielding (in red) in the whole of the tower is included in two bridges; 200 m main span. Soft soil category (TD); keywords in Fig. 1b

Tower shape	Cable layout	Main span length		
		$L_P = 200$ m	$L_P = 400$ m	$L_P = 600$ m
H	LCP	39	31	32
	CCP	18	19	10
Y	LCP	38	19	15
	CCP	87	3	0
YD	LCP	75	16	0
	CCP	32	27	15
A	LCP	57	2	0

Table 1: Damage ratio Ω (%) in different cable-stayed bridges without energy dissipation devices. Soft soil category (TD). The keywords referring the tower shape are described in Fig. 1(b). LCP and CCP stand for lateral and central cable systems respectively.

during an earthquake by means of a simple scalar measure:

$$\Omega = \frac{E_{Sp}}{E_W} 100 \quad (\%) \quad (3)$$

where E_W is the time integral of the total work done by the seismic inertial forces during the earthquake. The parameter E_{Sp} is the work done by the forces associated with plasticity in the towers. A comprehensive description of the components involved in the energy balance is included in Ref. [30].

The damage ratios in cable-stayed bridges with different main span lengths, tower shapes and cable layouts are listed in Table 1. The damage ratio provides a global understanding of the response, but no distinction is made between the energy dissipation at the tower legs and the struts. As a consequence, the maximum allowable damage ratio depends on the tower shape and the number of transverse struts connecting the lateral legs. In general,

problematic cracking associated with longitudinal reinforcement yielding at key sections in the tower legs (normally at the base) was observed when the damage ratio was above 25%. It is suggested that when the structure is subject to the design ground motion, the percentage of energy dissipated by plasticity in the tower sections should remain below $\Omega_{\text{max}} = 25\%$.

Influence of the Tower Shape

It is observed from Fig. 4 that all the bridges studied with reduced span (200 m) presented inadmissible cracking levels at key tower locations, such as the points of connection with the deck or the tower base, exceeding the reinforcement yielding limit. A significant aspect of the tower shape in terms of the seismic response is the configuration of the piers below the deck. It is remarkable to note from Table 1 that in the studied bridges with lower diamond and 200 m main span, more

than 50% (in some cases, up to 87%) of the total energy introduced by the earthquake is dissipated by structural damage to the towers. In this case, the damage ratio is well beyond the limit ($\Omega_{\max} = 25\%$) and is deemed inadmissible due to the special importance of the towers in the global integrity of the bridge. However, this unadvisable response is corrected in bridges with larger main spans because there is more available space to accommodate the lower diamond. The design of the tower should avoid abrupt changes in the slope of the lateral legs that otherwise concentrate the seismic damage.

The damage factor in H-shaped towers is larger when compared with other tower shapes without the lower diamond, and it is nearly independent of the main span length. This result is explained by the localization of the reinforcement yielding in the strut-leg connections of H-shaped towers (Fig. 4). The H-shaped towers usually require several transverse struts to provide enough stiffness in transverse direction, which may result in larger seismic damage if the transition between sections at the strut-leg connections is not carefully designed.

From the view point of seismic response, A- and Y-shaped towers represent superior solutions in cable-stayed bridges with main spans ranging from 200 to 600 m. The geometry in these towers constraints the transverse displacement at the point where the lateral legs are connected above the deck. This effect is more pronounced in small bridges with Y-shaped towers due to the larger inclination of the legs. The geometric constraint in Y-shaped towers favours the cantilever response in transverse direction of the vertical anchorage area, which in turn concentrates cracking at the level of the lower cable anchorage (Fig. 4). Cracking in the anchorage area is conveniently avoided in homologue A-shaped towers, although in this case the global damage ratio is higher due to reinforcement yielding in the strut (Table 1). Again, smooth transitions between the legs and the anchorage area sections would help reduce cracking.

Influence of Cable Arrangement

Figure 4 and Table 1 show, respectively, the significant increase in the peak deformation and the amount of seismic energy dissipated by structural damage in bridges with central cable arrangement (CCP), in comparison

with homologue models with two lateral cable planes (LCP). This is explained by an increase in the transverse reaction of the deck against the towers in CCP bridges during the earthquake, which rounds to 30%. In bridges with inclined cable planes (LCP), the cables carry part of the transverse inertial forces in the deck to the tower anchorages. However, in CCP bridges, the cables lie in a vertical plane, and the transverse action of the deck is exclusively transmitted to the tower at the deck level through the deck-tower connection. This is the main reason behind the increase in the reaction of the deck against the towers in CCP bridges, especially prominent in the structures with moderate main span (200 m). The central cable arrangement is not recommended in seismic areas.

Influence of Main Span Length

With the exception of H-shaped towers, the structural damage is reduced with an increase in the main span (Table 1). There are two main reasons for this effect: (1) the tower sections must increase in size with the main span length to support the self-weight of the structure and (2) with an increase in the distance between towers, the vibration periods of the significant deck modes also increase, with corresponding decrease in acceleration values in the design response spectrum.

Retrofit with Energy Dissipation Devices

The significant tower damage observed in the tower examples shown in the preceding sections is clearly undesirable for the overall bridge stability. The transverse reaction of the stiff (essentially rigid) deck-tower connection during an earthquake was identified in this study as the main source of structural damage (see Fig. 4). Therefore, it is not surprising that current cable-stayed bridge designs aim at providing a partially rigid connection to ensure sufficient stiffness under non-seismic forces and flexibility, ductility and energy dissipation in the event of a strong earthquake.⁸ In this study, two solutions based on energy dissipation devices are proposed: viscous fluid dampers (VDs) and yielding metallic dampers (MDs). The design of the devices is conceived as a retrofit solution for the structures

presented previously, in which only the transverse connection between the deck and the towers is modified. In the design of the dampers included in this section, only the transverse response of the towers (Y axis) is considered.

VDs are implemented according to the scheme as shown in Fig. 5(a), based on the design of Rion-Antirion cable-stayed bridge³¹ (Greece). The FR is designed to fail when the load transmitted to the tower through the dampers exceeds the maximum deck-tower reaction under service conditions (wind loading and moderate earthquakes). Hence, the FR prevents the activation of the dampers under moderate dynamic actions.

As an alternative solution, the MDs in the deck-tower connections are also explored (Fig. 5b). MDs rely on the hysteretic properties of metals to dissipate energy, at the same time acting as stiff members to reduce structural deformation under moderate seismic forces.

Prior to the designing of the energy dissipation devices, the engineer should select the desired level of seismic protection in the structure. A non-linear static analysis is conducted by incremental rise in the transverse force of the deck that is transmitted through the damper to the tower. The objective is to obtain the critical damper force that would cause a drop in the tower stiffness induced by concrete cracking, F_{crack} . The MDs should yield and VDs should release pressure before the tower is significantly damaged. A safety factor of about 1.3 to limit the maximum reaction of the seismic device has been suggested in Ref. [32], therefore:

$$F_{d,\max} = F_{Y,MD} = 0,765 F_{\text{crack}} \quad (4)$$

where $F_{d,\max}$ and $F_{Y,MD}$ are, respectively, the damper force that activates the pressure control in the VD and the yielding force of the MD.

Design of VDs

The VD design follows the DDBD proposed in Ref. [8] to control the longitudinal response with linear dampers between the deck and the tower. However, when the VDs are applied transversely, the following modifications for the design procedure are considered:

1. The central part of the deck in the main span sways freely in the transverse direction, without interacting

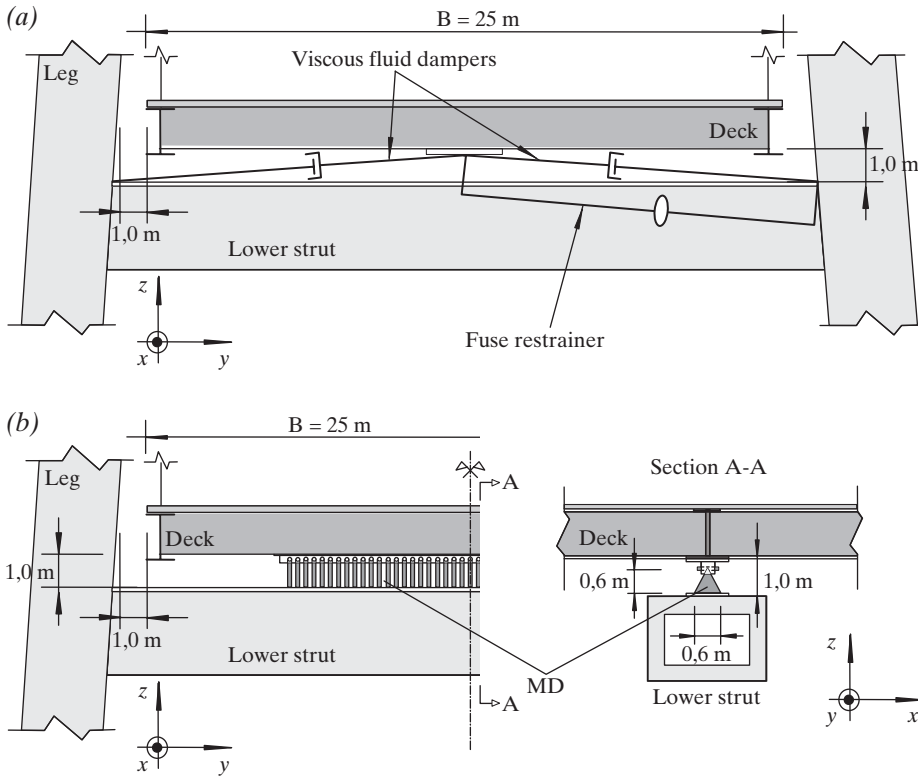


Fig. 5: Energy dissipation devices located at the transverse connections of the deck: (a) VDs; (b) yielding MDs

with the towers. As a consequence, the length of the deck that affects the tower in transverse direction during the earthquake ($L_{Y,deck}$) is shorter than the total span. This effect is more significant for large spans due to the smaller restraint of the towers to the transverse movement of the deck.²² This is evident from the data given in Table 2 in which the main span (L_P) and the corresponding effective length of the deck ($L_{Y,deck}$) are compared. The effective mass of the deck in transverse direction is also considered in the design of the VDs.

2. The damper design displacement (Δ_d) is less than the one that could be considered in longitudinal direction to prevent the contact between the deck and the tower legs. The space constraints shown in Fig. 5a determined design damper displacement of less than 1,0 m. $\Delta_d = 0,3$ m in this study.
3. The design damper force is limited to the critical load that would introduce inadmissible cracking in the tower ($F_{d,max} = 0,765 F_{crack}$). However, this condition never limited the design in this study. This can be appreciated from Fig. 7, where the peak damper force is well below $F_{d,max}$ during the earthquake. The critical design condition is the peak damper displacement.

Considering these criteria, the DDBD methodology proposed in Ref. [8] was applied to design the transverse dampers with a design damping level of 30% ($\xi = 0,3$). Table 2 presents the damping coefficient (C_d , for each VD unit) that relates the damper force and velocity. Only linear dampers were considered.

The FR is designed to fail (fuse) when the force in the deck–tower connection reaches 10% above the peak force expected under strong wind loading in the whole deck: F_{wind} (considering 30 m/s as basic wind speed³³). Prior to failure, the FR remains completely elastic. The elastic stiffness of the FR ($K_{el,FR}$) should be large enough to prevent relative movements of the dampers. In

this case, $K_{el,FR}$ was interpolated from the experimental testing of the FRs in Rion-Antirion bridge, as reported in Ref. [31]. The values of $K_{el,FR}$ and F_{wind} considered in this study are included in Table 2.

Design of Yielding MDs

The MDs are designed to yield before the critical force ($F_{y,crack}$) is transmitted to the tower through the deck–tower connection. The objective here is to protect the main structure in the event of a large earthquake by concentrating the inelastic deformations in the MDs. Due to restrictions in the space between the strut and the deck that is required to accommodate the damper, the design of the MD begins with selection of a reasonable plate width ($B_p = 0,6$ m) and height ($H_p = 0,6$ m) that facilitates construction (see Fig. 5b). The number of plates (N_p) in the device is obtained by allowing yielding to occur at the maximum allowable force ($F_{Y,MD}$); therefore:

$$N_p = \frac{4F_{Y,MD} H_p}{f_{s,y} t_p^2 B_p} \quad (5)$$

where $t_p = 0,03$ m is the plate thickness. The yielding limit of the steel in the MD is $f_{s,y} = 552$ MPa, $E_s = 210$ GPa.

The distance between consecutive plates (0,03 m) and N_p determine the length of the MD as approximately $L_{MD} = N_p(t_p + 0,03)$. The values of the resulting length of the MDs in bridges with H-shaped towers are included in Table 2. The length is below the deck width (25 m) and satisfies the space constraints (Fig. 5b). It can be observed from the data in Table 2 that the MDs are designed to yield well beyond the peak reaction under wind loading ($F_{Y,MD} > F_{wind}$) to prevent excessive permanent movements under frequent events and fatigue problems in the MD.

		Main span length		
		$L_P = 200$ m	$L_P = 400$ m	$L_P = 600$ m
Effective deck length	$L_{Y,deck}$ (m)	108	73	55
Deck–tower reactions	F_{wind} (MN)	0,91	2,42	4,45
	F_{crack} (MN)	13,32	23,59	38,42
VD properties	$F_{d,max}$ (MN)	10,19	18,05	29,39
	C_d (MNs/m)*	2,89	1,93	1,49
	$K_{el,FR}$ (MN/m)	334,4	889,3	1635,3
MD properties	$F_{Y,MD}$ (MN)	10,19	18,05	29,39
	$K_{el,MD}$ (MN/m)	215,3	381,4	621,2
	L_{MD} (m)	3,92	7,72	13,2

*VD stiffness given per damper unit.

Table 2: Effective length of the deck in transverse direction, deck–tower reactions under different loading conditions (wind: F_{wind} and tower damage: F_{crack}) and damper properties; soft soil (TD); H-shaped towers

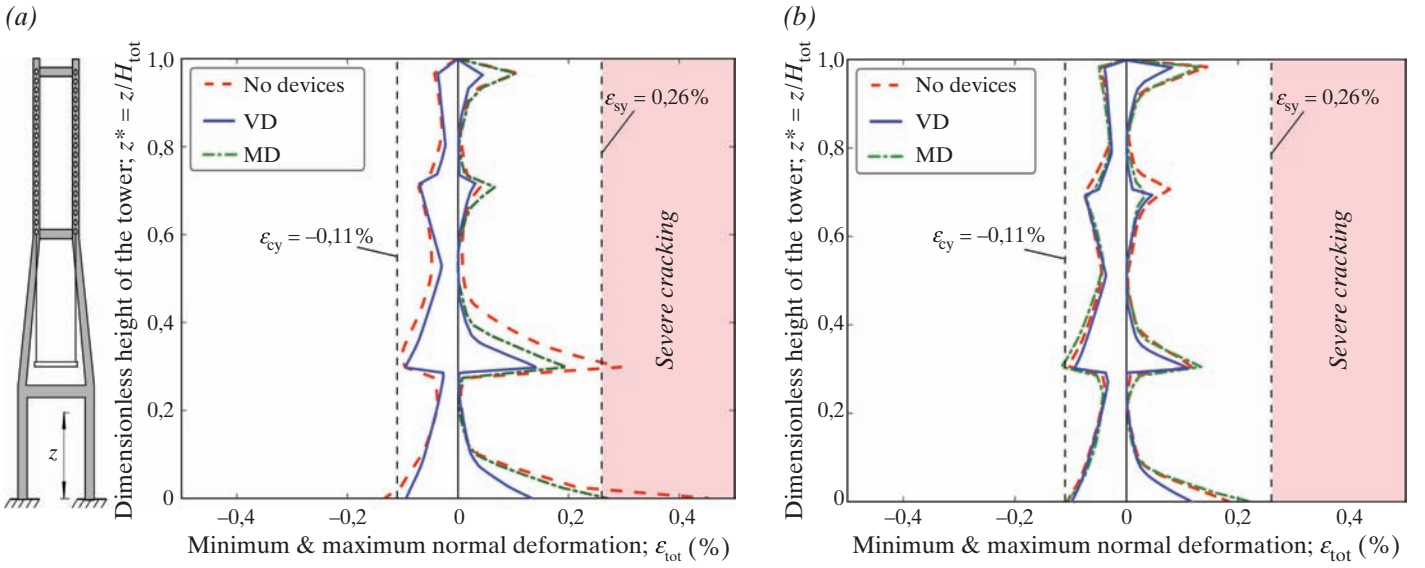


Fig. 6: Peak deformation recorded in the reinforcement along the height of the tower for bridges with and without energy dissipation devices. H-LCP model with (a) 200 m main span; (b) 400 m main span. Soft soil category (TD). H-LCP model

Finally, the stiffness of the MD in elastic range (K_{el}) is calculated as

$$K_{el,MD} = \frac{N_p E_s B_p t_p^3}{6H_p^3} \quad (6)$$

Verification of the Seismic Response

The response of the cable-stayed bridges retrofitted with dampers (summarised in Table 2) was compared with that obtained in the original bridges without dampers employing NL-RHA.

In the FE model, the VD's are represented with linear dashpot elements between the deck and the towers. The FR is included by means of an elastic connector element that is removed from the analysis when its failure load is exceeded. On the other hand, a single unit-length "truss" finite element has been defined to represent the global response of the MD in the numerical model. A moderate hardening is provided in this element according to the experimental results reported elsewhere,²⁴ through a combined kinematic/isotropic rule.

Figure 6 shows the peak deformation of the longitudinal reinforcement along the tower height for bridges with 200 and 400 m span. It can be appreciated that the response is effectively controlled by means of energy dissipation devices in bridges with 200 m main span. In this case, VD's and MD's reduce the peak deformation in the tower reinforcement by 70% and > 40%, respectively. In the bridge with 200 m main span, the proposed devices prevent the yielding of the tower reinforcement, avoiding the severe cracking at the tower base observed

in the absence of the dampers. It was observed that dampers reduced the tower damage significantly in small bridges with different tower shapes. However, the efficiency is reduced in bridges beyond 400 m span. As the main span is increased, the effective length of the deck that interacts with the towers in transverse direction during the earthquake is reduced, as was observed in Table 2. Consequently, in the long-span bridges, the mass of the tower is very large in comparison with the effective mass of the deck. For controlling the transverse response of the towers by dampers, the deck-tower connection is not the ideal location for their installation in bridges above 400 m main span.

It is noted that VD's are more efficient than MD's when controlling the tower response. This may be explained by the fact that VD displacement is inherently out of phase with velocity.³⁴ The peak response of the tower and the extreme reaction introduced by the VD never occur at the same time, which would be the case if MD's were employed. However, MD's have other relative advantages associated with their lower construction and maintenance costs. Furthermore, higher efficiency levels may be achieved if the design of MD's is not constrained by limitations of space, as is the case in new bridge designs.

The behaviour of the dampers during earthquake is verified by means of their load-displacement response presented in Fig. 7 (model with 200 m span). This figure is particularly useful to check whether the peak damper response is below the admissible levels.

The following can be observed from Fig. 7: firstly, the peak relative displacement between the deck and the tower (design damper displacement) is within the available space between the deck and the tower legs (i.e. less than 1 m) and the impacts are prevented for both VD's and MD's; and secondly, the maximum damper reaction is below the critical level. In the case of VD's, the pressure control is not required ($F_{d,VD} < F_{d,max}$). The extreme reaction due to wind loading (F_{wind}) is depicted in Fig. 7 to verify that the FR does fail as expected, activating the VD. It can also be observed that the yielding level of the MD is above the F_{wind} threshold, as intended. Although the relative displacements during the earthquake are larger with VD's, this solution is more stiff under transverse loading in service conditions due to the FR, as is observed from the data given in Table 2 ($K_{el,FR} > K_{el,MD}$).

From Fig. 7, the observed ductility of the MD is $\mu_{MD} = u_{max}/u_y = 0.14/0.047 = 3$. According to Ref. [32], this ductility level corresponds to a damping of 29%, which is very similar to the values considered in the design of VD's (i.e. $\xi = 0.3$). The resistance of the MD's to low-cycle fatigue is verified through the Coffin-Manson and Palmgren-Miner rules.³⁵ The maximum permanent displacement observed in the deck with MD's after the earthquake is 0.15 m, which can be easily corrected by means of hydraulic actuators without interrupting the traffic.

As has been discussed, the incorporation of dampers reduces the energy dissipated by damage in the towers (E_{Sp}). In the case of VD's, this reduction of the

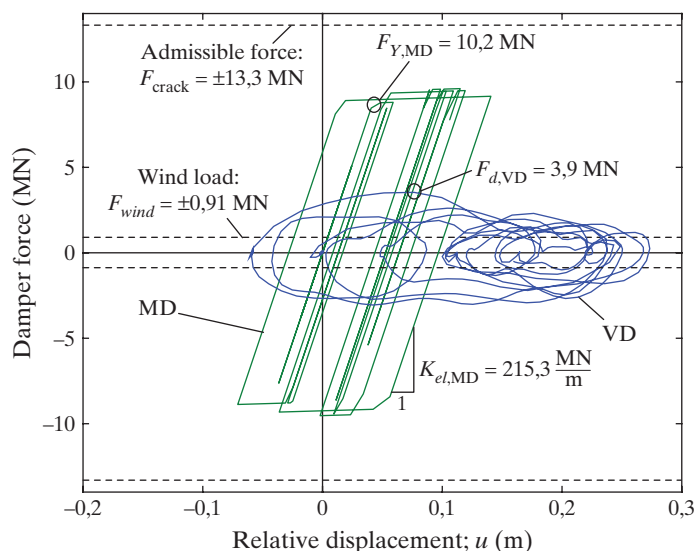


Fig 7: Load–displacement history of the energy dissipation devices at deck–tower connections in the H-LCP bridge. Main span $L_P = 200$ m. Soft soil category (TD). The VD force is the sum of the two damper units

structural damage is due to increase in the viscous energy dissipation. When VDs are included, the damage factor (see Table 1) is reduced from $\Omega = 39$, 31 and 32% to $\Omega = 10$, 19 and 21% in H-LCP models with 200, 400 and 600 m main span, respectively. As for the retrofit with MDs, the tower is protected against seismic damage at the expense of the plastic energy that is dissipated in the metal plates by hysteresis. In the same bridges but with MDs, the damage factor is $\Omega = 24$, 30 and 31%. The efficiency of the energy dissipation devices in the bridge with 200 m main span may be appreciated by simply comparing the damage ratios in the structure with and without the dampers. By employing VDs or MDs in the bridge with moderate span ($L_P = 200$ m), the damage is reduced below the maximum admissible value (i.e. damage factor $\Omega_{\max} = 25\%$). However, in long-span bridges (above $L_P = 400$ m), the proposed MDs are unable to reduce the damage factor to the admissible levels due to the reinforcement yielding in the transverse struts. Alternative retrofit solutions should be considered in these cases.

Conclusions

A large number of cable-stayed bridges with different structural configurations have been studied by means of rigorous finite element models with different analysis procedures. The following conclusions are drawn:

1. Unlike the code-based Pushover methods, the advanced Pushover procedures that include the effect

of high-order vibration modes can accurately estimate the peak seismic response. Pushover procedures notably reduce the computational cost when compared with direct non-linear dynamic analysis. A new advanced Pushover method (CNSP) that accounts for the 3D nature of the seismic excitation and the interaction between vibration modes in the non-linear range is discussed. CNSP significantly improves the estimation of the peak response in cable-stayed bridges due to the strong modal coupling. Another advantage of the procedure is that it typically falls on the safer side.

2. The non-linear seismic response of different cable-stayed bridges has been compared to obtain design criteria for these structures in the earthquake-prone areas. The cable system layout with one cable plane anchored to the centreline of the deck section is not recommended because the transverse reaction of the deck into the towers is increased in this structure. The detailed transition between sections and the smooth change of the slope of the legs is very important, especially in towers with lower diamond. Towers with “A” and inverted “Y” shapes represent good design solutions, reducing the risk of cracking and rebar yielding in the critical areas. It has also been observed that generally, the expected seismic damage in the towers is smaller in larger bridges, in comparison with the structures with short-to-medium spans.
3. Retrofit solutions with transverse energy dissipation devices at the

deck–tower connection have been designed to minimise the energy dissipated by structural damage in the tower. It has been observed that VDs are more efficient than yielding MDs with distributed plates if their height is constrained by the allowable space between the tower and the deck. For the bridges of moderate span (200 m) studied, the proposed deck–tower connection with dampers efficiently prevented yielding in the reinforcement of the tower legs. Regardless of the damper typology, when it connects the deck and the tower in transverse direction, the efficiency to mitigate the tower damage is reduced in long-span bridges.

Acknowledgements

The authors thank the support extended by the Technical University of Madrid (UPM, Spain) and Eduardo Torroja Research Institute (IETec, Spain) in the course of this research project. The comments of Dr Christian Malaga-Chuquitaype are also highly appreciated.

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