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ASSESSMENT OF THE SEISMIC PERFORMANCE OF A SEISMICALLY ISOLATED BRIDGE

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ABSTRACT

The issue of the feasibility of using seismic isolation in bridges with rather long fundamental natural period is revisited herein, through the study of such a structure. The influence of the seismic isolation system as well as of foundation modelling assumptions on the overall dynamic behaviour is examined. The seismic performance of the structure is assessed using both standard and advanced analysis tools, for ground motion records scaled to the design-level earthquake and to higher intensities. Through comparing results from the aforementioned analyses, the applicability of linear analysis methods in such a context is investigated. All of the issues addressed might be of a broader interest to engineers involved in seismic design and assessment of bridges.

The results obtained quantify and highlight the impact of seismic isolation on the response of the bridge, while the effect of foundation modelling is minor. Equivalent linear analysis is found to be problematic, largely due to adopting the assumption of uniformly distributed damping along the structure. The earthquake performance of the structure is found to be satisfactory, with a noncritical for life safety failure expected for twice the design intensity level.

INTRODUCTION

Seismic isolation has been widely employed in bridge engineering, in order to mitigate the effects of earthquake loading and usually to ensure an elastic response of the structure. This is achieved by (CEN, 2005):

(a) increasing the flexibility of the structure, thus reducing the inertial forces, but at the same time increasing displacements,

(b) increasing structural damping, which reduces displacements and potentially also the forces, or

(c) combining both.

In the bridge under consideration, option (c) has been selected as the design option (a common choice in practice). It is accomplished by a combination of high flexibility devices (elastomeric bearings) at the deck – pier interface, which can sustain high deformations in a non-linear elastic manner, with energy dissipation devices (dampers) installed along the bridge. One of the main objectives of this study is to analyse and quantify the effectiveness of this solution, which has been the subject of previous works (e.g. Kappos and Dimitrakopoulos, 2005) for other bridges and/or contexts.

Another key objective is to evaluate the applicability of equivalent linear analysis methods for the design or assessment of such a highly non-linear structure. Although nowadays often supplemented by non-linear analysis in the case of important structures, the solution of conducting only linear analysis is still allowable by various aseismic codes (e.g. CEN, 2005).

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The assessment of the earthquake performance of a structure is nowadays typically conducted to check if the structure complies with new, generally stricter, performance demands (arising after the construction is completed), to check the influence of possible structural modifications, or to check the behaviour of the structure for various earthquake scenarios, usually of lower probability of exceedance than the design actions; the latter is the context of the study presented in this paper.

The Nestos bridge, addressed in this paper, is one of the largest among the over 500 bridges of the Egnatia Motorway. It is located between the cities of Kavala and Xanthi in Northern Greece, crossing the river Nestos (Fig.1).

Figure 1. View of the bridge crossing the river Nestos. The depicted part constitutes less than 50% of the whole structure.

The analyses presented herein refer to the seismic loading case, hence taking into account earthquake, self-weight and traffic loads; thermal actions, snow, wind or any other transient or accidental kind of load are not part of this study. Moreover, asynchronous earthquake excitation and soil-structure interaction phenomena were also beyond the scope of the work presented herein. Clearly, the results of this study do not necessarily apply to all seismically isolated bridges.

DESCRIPTION OF THE STRUCTURE

Basic data

The structure was designed by the Greek firm DOMI S.A. (Athens) and constructed adopting the popular in recent years configuration of two, almost identical, statically independent parallel bridges (Fig.2), each one carrying one direction of motorway traffic. Both branches comprise twelve spans of approximately 38 m each, resulting in a total length of 456 m. Two 250 mm expansion joints are located at the ends and one in the middle (330 mm).

In plan, half the bridge is a circular arc with a mild curvature (radius R = 1800 m) and the other half is on a clothoid (Fig.2). In elevation, it begins descending at a slope of 1.5%, decreasing to a constant 0.6% after the first three spans (Fig.2).

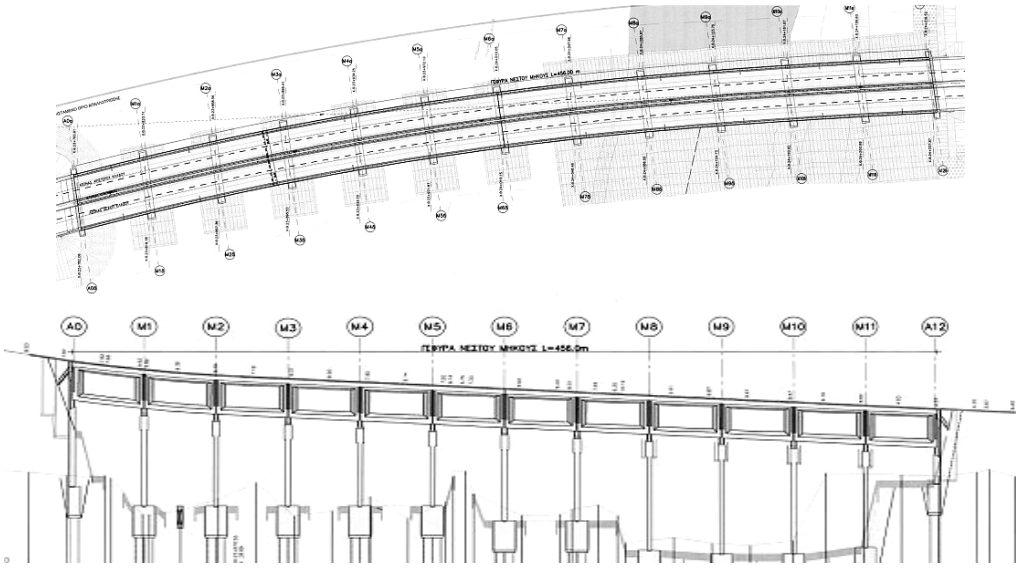

Figure 2. Plan (above) and longitudinal section (below) of the Nestos-bridge.

The deck consists of five simply-supported prestressed concrete (post-tensioned) double-T beams (2.30 m deep and 0.80 m wide). They support a continuous 14.45 m wide and 0.30 m thick slab, composed of a 20 cm cast-in-situ slab on 10 cm precast slab parts (Fig.3). Near the piers, the beam web thickness is gradually increasing until the cross-section becomes rectangular (0.80 × 2.30), to accommodate the increased shear forces near the support. Moreover, transverse connecting beams 0.60 × 1.80 are provided above the piers (Fig.3) for local strengthening of the deck section.

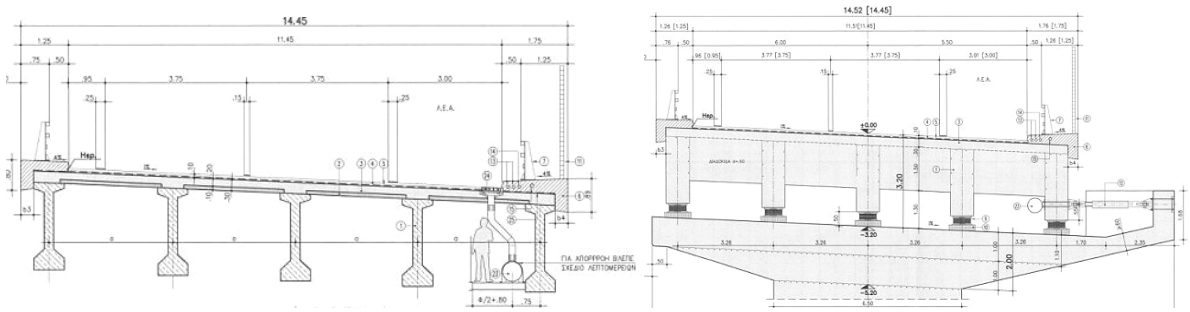

Figure 3. Typical deck cross-section at the span (left) and above the piers (right).

The 11 wall-type, rather squat, piers have an approximately elliptical cross-section (major axis of 6.5 m and minor axis of 2.5 m) and a height ranging from 2.81 m to 5.67 m (Fig.4). The foundation of the piers is a 3×3 group of Ø120 piles (2×4 at the abutments), with lengths ranging between 20.00 and 37.00 m, with a 2.0 m thick pile cap (Fig.4). Most piers are located either in the river or in its floodplain. Therefore, extended ground improvement with stone columns has taken place.

The materials used are concrete B35 (≈C30/37 in Eurocode 2) for the piers, B45 (C35/45) for the deck, and B25 (C20/25) for the foundation; ordinary (non-prestressed) steel S500s (B500C) and prestressing steel 1570/1770 are used for the reinforcement.

The seismic isolation system consists of 500×600, 154-187 mm thick, common (low-damping) elastomeric bearings, which support the deck's prestressed beams (Fig.5), and hydraulic viscous dampers, which provide supplemental energy dissipation. The latter are installed at both ends of every span in the transverse direction and at one end of each span in the longitudinal direction (Fig.5).

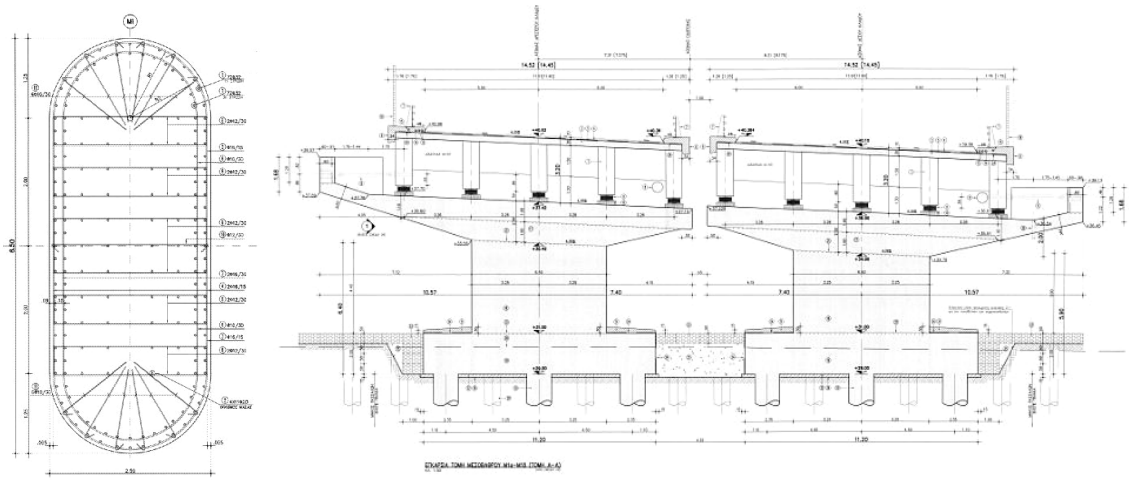

Figure 4. Pier cross-section (left) and transverse section (right).

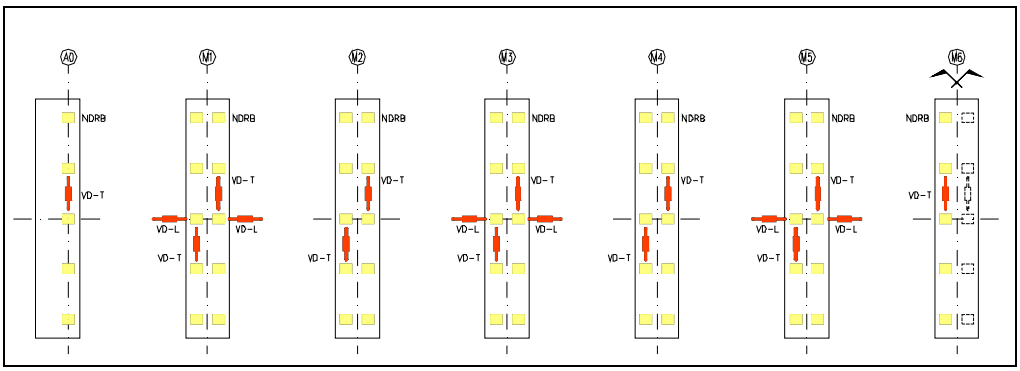

Figure 5. Seismic isolation system arrangement sketch. There are 10 rubber bearings and 2 transverse dampers on each pier (5 and 1, respectively, on abutments), and 2 longitudinal dampers on every second pier. Half the bridge is shown, as the arrangement is symmetrical in the other half.

Regarding the seismic hazard characteristics of the site, it belongs to the seismic zone I of Greece, i.e. peak ground acceleration (PGA) of 0.16g. This value is increased to 0.20g, due to its proximity (a bit more than 1 km) to a possibly inactive fault. Furthermore, the design PGA reaches 0.26g, since an importance factor of $\gamma_I = 1.30$ was selected.

Structural Analysis Models

Analysis was conducted with the aid of the structural analysis software *SAP2000* (CSI, 2010). The deck slab was modelled using shell elements, while all beams and piers were modelled with line (3D beam-column) elements. The foundation was modelled with equivalent linear springs, whose values were obtained from the initial geotechnical study.

Linear Analysis

In applying the 'standard' linear response-spectrum analysis, the effect of energy dissipation devices was approximated by an effective viscous damping uniformly distributed along the structure. Its value was calculated, using the procedure adopted by Eurocode 8, as 27.3% of the critical damping. The bearings were modelled using their equivalent linear effective stiffnesses, according to Kelly and Naeim (1999). Expansion joints were not included in the model and maximum displacements were checked in the analysis making sure they are lower than the joint width.

Non-Linear Analysis

On the other hand, in carrying out non-linear response-history analysis, the non-linear behaviour of various structural components was explicitly taken into account. The viscous dampers were modelled with one of the various types of *SAP2000* non-linear link elements (NL-links). It is based on the Maxwell model of viscoelasticity, i.e. a non-linear damper in series with a spring (CSI, 2010). The necessary parameters were calculated according to the initial design of DOMI SA. The elastomeric bearings were simulated with NL-links, based on the bilinear smooth model used in *SAP2000* (CSI, 2010) and with parameters calculated according to Kelly and Naeim (1999). The potential closing of expansion joints was explicitly modelled through an NL-link, which simulates such gap-behaviour (CSI, 2010).

Using the in-house developed section analysis software *RCCOLA.NET* (Kappos and Panagopoulos, 2011), moment-curvature relationships and bending moment-axial load interaction diagrams were calculated for the pier section (Fig.6). These were used to define the properties of plastic hinges situated at the bottom of the piers, in the frame of the standard lumped plasticity approach adopted by *SAP2000* (CSI, 2010).

The shear strength of the piers was calculated conservatively (using design parameters instead of characteristic) as per Eurocode 2 provisions (CEN, 2004); it was found to be 9.44 MN in the short and 18.64 MN in the long direction.

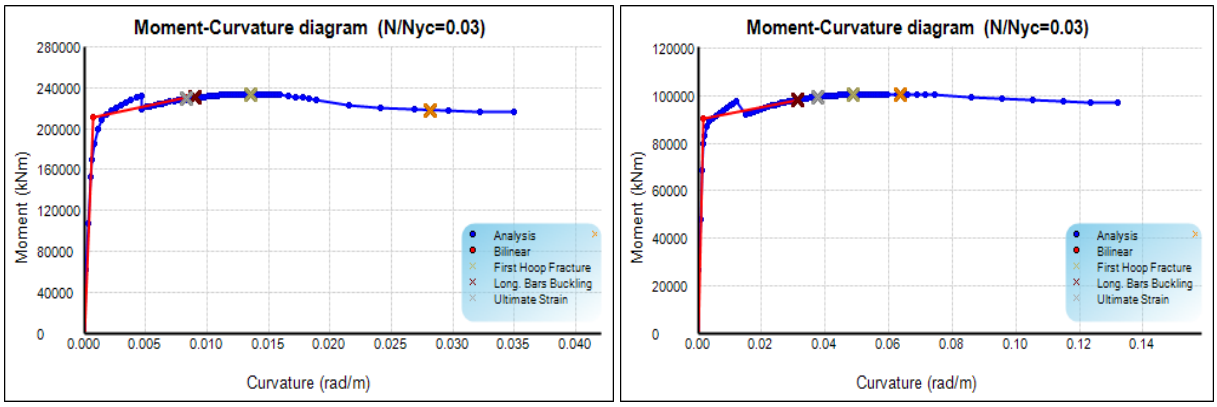

Figure 6. Moment – curvature (M-φ) diagrams of the pier cross-section for the design axial load; about the short side (left) and about the long side (right). The actual behaviour is depicted with the blue curve, while the bilinearised one with the red.

ANALYSIS RESULTS

Modal Analysis

Initially, the modal characteristics of the bridge were identified using the previously described model. The first four modes involve deck translations (Table.1) while the rest involve bending and torsional deformations of the spans or the piers. For subsequent analyses, 130 modes were considered, so that the sum of the effective modal masses amounts to more than 90% of the total mass of the structure (CEN, 2005).

Table 1. Modes pertinent to displacements of the deck (natural periods, cumulative participation mass ratios and a brief description are provided for each one).

Mode	T (s)	ΣUx (%)	ΣUy (%)	Description
1	1.824	75.67	0.13	Translational - longitudinal
2	1.815	76.99	59.67	Translational - transverse
3	1.812	78.70	78.66	Translational - transverse
4	1.140	78.70	78.69	Rotational - torsional

Subsequently, the effect of specific modelling assumptions and design choices on the behaviour of the structure was investigated. To this purpose, comparative modal analyses of five alternative models of the bridge were conducted. The models (see caption of Fig.7) differ in:

- the value of the bearing shear modulus that varies with ageing and temperature (the minimum being equal to 1.0 MPa and the maximum to 1.50 MPa),
- the fixity conditions of the pier and abutment foundation (fixed support or equivalent linear springs) and
- the connection between pier and deck (monolithic or through elastomeric bearings, as in the actual bridge).

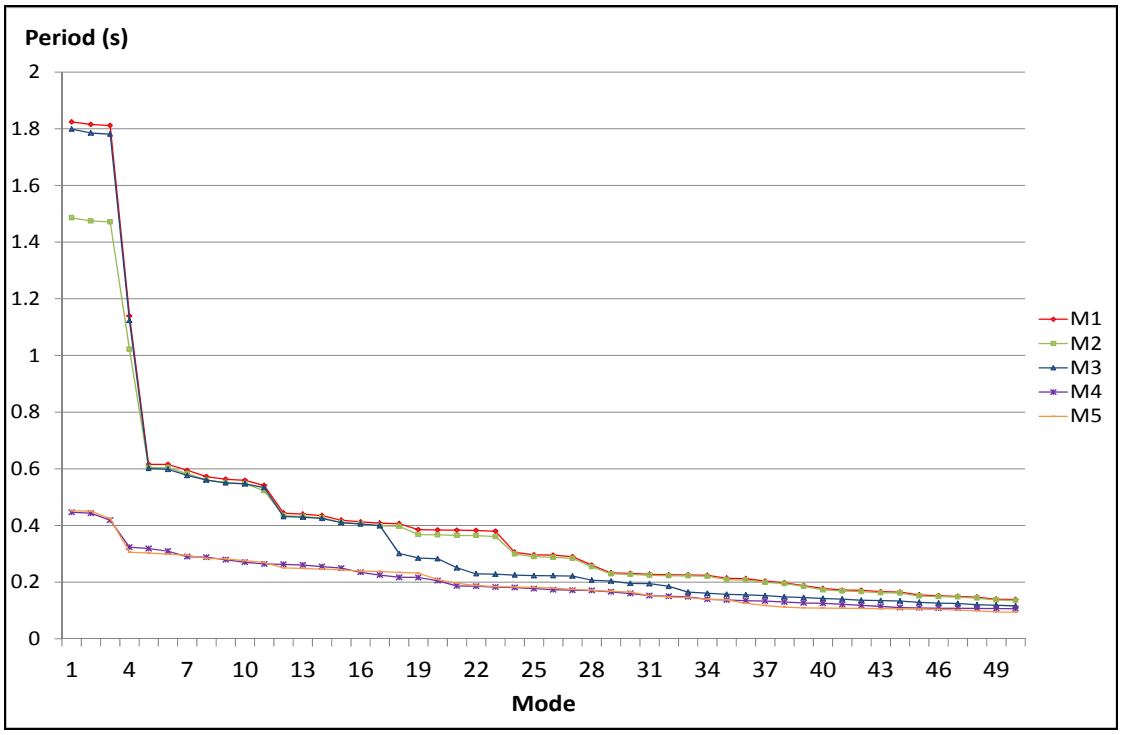

Figure 7. Natural period of 50 modes for five different models: (M1) Elastomeric bearings with lower bound shear modulus supporting the deck and equivalent linear springs at foundation; (M2) similar to M1, but with upper bound shear modulus; (M3) similar to M1 but with fixed supports at foundation; (M4) similar to M1, but with monolithic connection of the piers with the deck. (M5) monolithic connection of the piers with the deck and fixed supports at foundation.

The major role that seismic isolation plays in the flexibility of the structure is clearly shown in Fig.7, particularly when comparing the response of models M1 and M4. The much shorter natural periods of M4 translate into much higher acceleration and hence inertial forces acting on the structure. This becomes even more evident when examining the design spectrum (Fig.8). The spectral acceleration for M4 would be approximately quadruple that for M1, which would lead to approximately quadruple inertia forces acting on the (fundamental mode dominated) bridge. Hence, much higher ductility would be required in M4 to maintain a force level similar to M1.

Comparing M1 with M2 (Fig.7), it can be observed that - as expected - the shear modulus of the bearings influences substantially the modes related to deck translation. Specifically, the first three

natural periods decrease from 1.82 s to 1.48 s. The fourth mode period decreases from 1.14 s to 1.02 s, while the influence on the rest of the modes is negligible.

A comparison between M1 and M3 (Fig.7) reveals that foundation fixity conditions of the bridge affect almost exclusively higher modes. These are mostly related to deformation of the piers and are associated with very low participating mass ratios. Consequently, it can be deduced that foundation modelling has only a minor effect on the global structural response. Considering the close similarity between M5 and M4, it becomes evident that these conditions would play an even lesser role, in the case of an equivalent non-isolated bridge.

Linear Dynamic Analysis

Elastic analysis was conducted for a site-specific elastic spectrum, developed in the frame of the actual design of this bridge, which is classified as an important one. This spectrum is more demanding than that of the 2003 Greek Aseismic Code (Fig.8) as well as of Eurocode 8. The initial design adopted a behaviour factor q = 1.0, in line with (conservative) common practice.

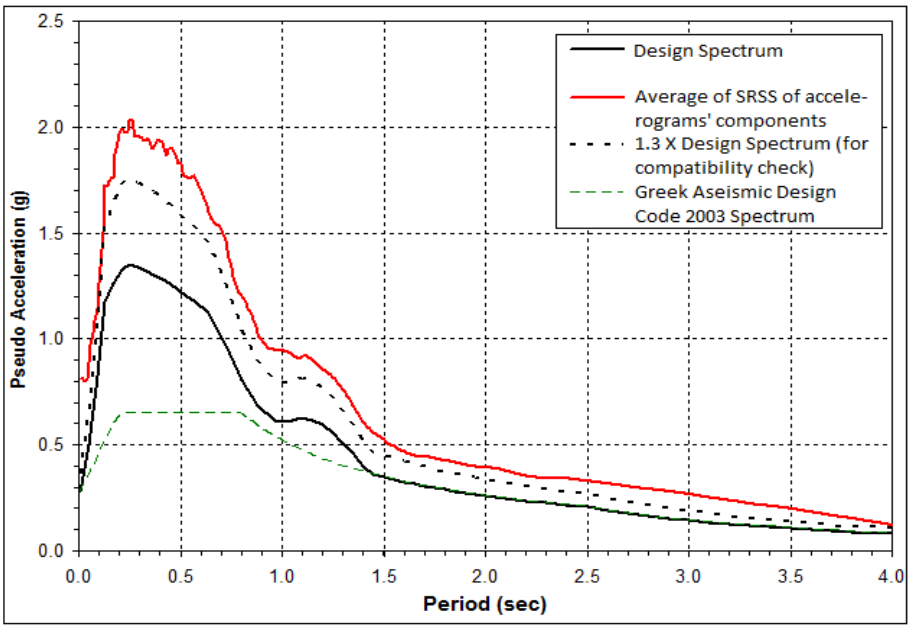

Figure 8. Design spectrum plotted against the spectrum of the 2003 Greek Aseismic Code for this site. Also, the average of the SRSS spectra of the accelerograms used is compared to 1.3 times the design spectrum.

To quantify the contribution of the supplemental damping system, different analysis cases were carried out. In some of them, the damping offered by the viscous dampers was taken into consideration, whereas it was ignored in the others (32.3% of the critical damping ratio or 5.0%, respectively). Of course, the higher effective damping was applied only to modes with a natural period greater than $0.8T_{eff}$ (CEN, 2005), i.e. the first three modes.

Deck displacements in both directions were found to be representative of a single-degree-of-freedom system. In other words, the deck is displaced as a rigid body, reaching 13.1 cm in the longitudinal and 13.3 cm in the transverse direction. When neglecting the supplemental damping, these values increase to 24.4 cm and 24.6 cm, respectively. The displacements of the bearing supported deck are almost equal in the two directions due to the provision of joints at the abutments in both directions, while the bearings have the same shear stiffness in both directions and damping is applied uniformly along the structure.

The structural response, in terms of shear forces and bending moments of the piers, is depicted in Fig.9 for both cases of damping (with/without the viscous dampers). The axial forces are approximately equal to 18.0 MN in all piers. Considering the average response in each case, the ratio of 'high-damped' over 'low-damped' displacements was equal to 0.54, while the internal forces were

reduced by approximately 25%. This underlines the vital contribution of the damping system to the overall structural response, especially in terms of displacement control, which is of paramount importance in the case of structures characterised by a long fundamental period.

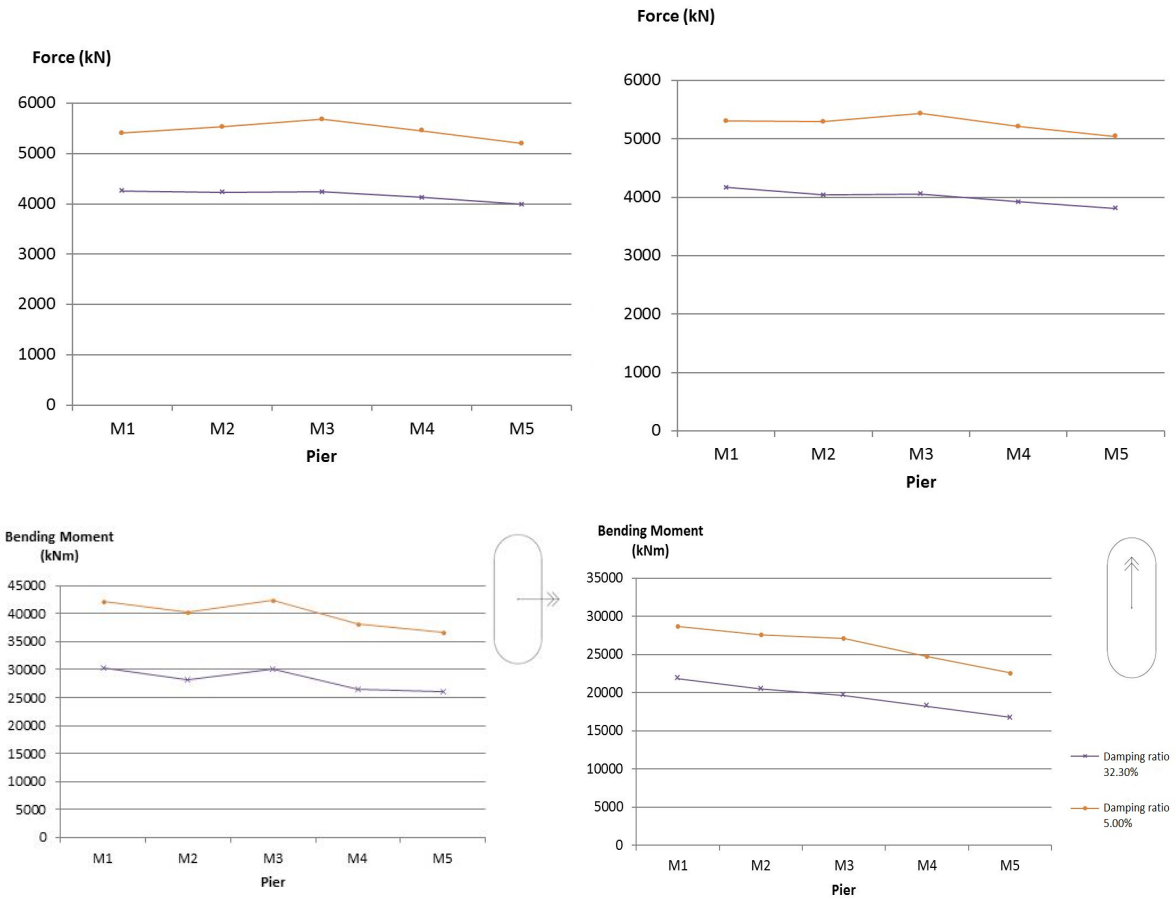

Figure 9. Shear forces and bending moments at the pier bottom section for a damping ratio of 32.3% (purple lines) and 5.0% (orange lines); shear force along the short side (top left) and in the long side (top right); bending moment about the short side (bottom left) and about the long side (bottom right).

Non-Linear Dynamic Analysis

Non-linear response history analysis was carried out using a suite of seven normalised natural accelerograms. The average spectrum of the SRSS of the two components of the accelerograms is greater than 1.3 times the design spectrum in the range of $0.2T_{eff}$ to $1.5T_{eff}$, with T_{eff} ranging between 1.45 s and 1.75 s (Fig.8). Thus, the set of accelerograms is deemed appropriate to use in the non-linear analysis of the structure. The direct-integration method based on the Hilber-Hughes-Taylor algorithm (CSI, 2010) was selected.

Evaluation of Linear Method

The results of this more refined (and demanding) analysis are used to evaluate those of the 'code-type' linear method. The evaluation focuses on whether the latter are consistently on the safe side, something desirable for equivalent elastic analysis methods, so long as over-conservatism is avoided. A key point in this respect is the validity of assuming uniformly distributed damping in the structure (Kappos and Dimitrakopoulos, 2005).

With respect to deck longitudinal and transverse displacements, as well as pier shear forces and bending moments in the transverse direction, linear analysis is found to be systematically on the safe side without overestimating the response of the structure, hence deemed satisfactory. Nevertheless,

discrepancies arise in the case of internal pier forces in the longitudinal direction, since the equivalent linear analysis fails to yield conservative results for most of the piers (Fig.10).

The assumption of uniformly distributed damping in the structure, adopted in response spectrum analysis, is identified as the main reason for these inconsistencies. The arrangement of dampers in the transverse direction (Fig.5) leads to effectively uniform damping along the bridge, hence rendering the assumption valid. Nonetheless, the longitudinally arranged dampers follow an 'every-other-pier' pattern, i.e. one pier with no damper is followed by one pier with one damper installed on each side and so forth (Fig.5). This approach results in the rather increased internal forces observed in every second pier, hence the characteristic 'zigzag' pattern shown in Fig.10. If they followed a similar pattern to the transverse ones, the result would have been an essentially uniform damping along the structure; however the solution selected by the designer is a more economic one.

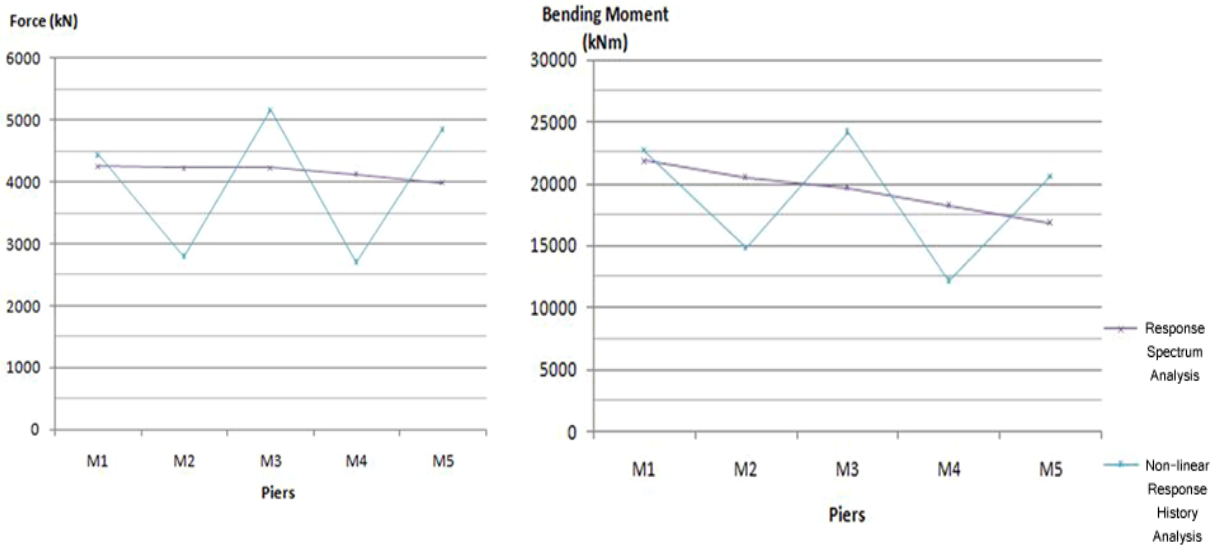

Figure 10. Comparison of response spectrum analysis (purple lines) and non-linear response-history analysis (light blue lines) with regard to the distribution of shear forces in the longitudinal direction (left) and bending moments about the transverse direction (right) along the length of the bridge.

Seismic Performance Assessment

The seismic performance of the structure is assessed for two levels of seismic action, i.e. the design-level and twice the design intensity, the latter corresponding to a very low probability of exceedance (less than 2% in 50 years), hence representing a particularly severe performance test for the bridge. For simplicity, response-history analysis for the higher intensity is carried out by directly scaling the selected natural records, as usually done in similar studies.

For the design-level seismic action, the performance of the bridge is assessed as fully satisfactory. The piers do not sustain shear failure or even flexural yield, as intended by the design (recall that q=1 was selected). The displacements are such that there is neither failure of the bearings nor impact with the neighbouring branch of the bridge or closing of the expansion joints.

For twice the design-level seismic action, the internal forces are increased considerably (Fig.11), nevertheless the wall-type piers still remain elastic. The resulting displacements (Fig.11), albeit much higher than before, are not high enough for the deck to sustain impact; however, closure of the expansion joints is observed and, more importantly, significant damage due to excessive shear deformation of the elastomeric bearings is predicted. This type of damage is not deemed critical for life safety; it will, however, require temporary closure of the bridge for replacement of the bearings.

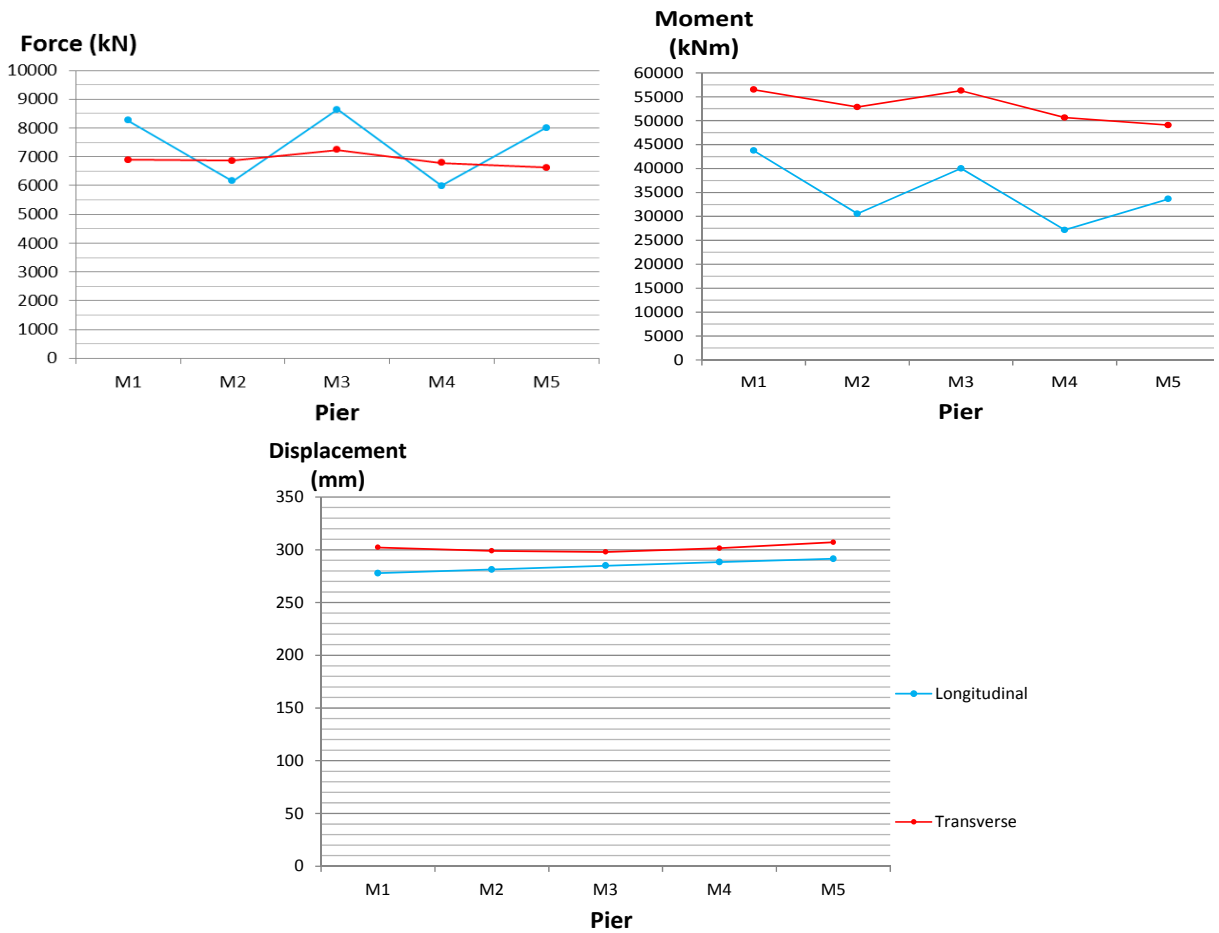

Figure 11. Response of the bridge to earthquake with twice the design-level intensity, for both directions. Pier shear forces (top left), bending moments (top right) and deck displacements (bottom). In the case of bending moments, 'longitudinal' means in the longitudinal direction, i.e. about the transverse axis of the pier; analogously 'transverse' means in the transverse direction.

CONCLUSIONS

A thorough analysis of a large seismically isolated bridge was conducted, using modal, response-spectrum, and response-history analysis. The main conclusions drawn from these extensive analyses are summarised as follows:

1. Seismic isolation plays a major role in the flexibility of the structure, as intended by design. As a comparison, an equivalent partially integral bridge (monolithic pier-to-deck connections) would sustain approximately quadruple inertial forces. Conversely, much higher ductility would be required for specific structural elements, in order to maintain a force level similar to that in the seismically isolated bridge.

2. With ageing and temperature, the shear modulus of the rubber bearings varies between a minimum and a maximum. This will influence substantially the first modes, which are related to translations of the deck. In bridges like the one studied here, these modes have a cumulative participation mass ratio of around 80%; hence, variation of the bearing shear modulus affects considerably the global structural response.

3. Modelling assumptions regarding the foundation fixity conditions of the bridge influence almost exclusively its higher modes. Thus, they do not have a major effect on the response of the structure.

4. The utilisation of hydraulic viscous dampers led to a reduction of internal forces by approximately 25% and of displacements by 46%. The latter is essential in the case of structures characterised by a long fundamental natural period, which would otherwise develop excessive seismic displacements.
5. It is well-known that use of dampers at the abutments only leads to non – uniform response reduction along the bridge and equivalent damping based approaches do not work well in such cases (Kappos & Dimitrakopoulos 2005). Interestingly, in the bridge studied herein where dampers were distributed along its length, the assumption of uniformly distributed damping along the structure still cannot be considered as fully valid in both directions. In the longitudinal direction dampers provide a rather non-uniformly distributed damping, because of their arrangement at every second pier (Fig.5), leading to a 'zig-zag' type of force distribution in consecutive piers (Fig.10).
6. The performance of the bridge was assessed for up to twice the design-level earthquake intensity. Severe damage due to excessive shear deformations of the elastomeric bearings is predicted. This type of failure is not deemed critical for life safety, since the ample seating lengths at both the abutments and the piers prevent unseating; it will, however, require temporary closure of the bridge for replacement of bearings.

ACKNOWLEDGEMENTS

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