Analysis and assessment of a seismically isolated bridge

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Abstract

This paper focusses on the seismic response of a bridge currently being constructed, using the incremental launching method, as part of the Egnatia motorway in northern Greece. The bridge is seismically isolated via elastomeric and PTFE bearings, and has fluid dampers installed at its end abutments. The scope of the paper is firstly the seismic analysis (using methods adopted also in practical design), and secondly the assessment of the bridge’s earthquake performance (using advanced inelastic time-history analysis).

Keywords: seismic isolation, bridges, damping, viscous dampers.

1 Introduction

The structure, which is analysed herein (fig. 1), is curved in plan with a radius R=1175 m and the deck section is laterally inclined (in the transverse direction) by 4.5%. In elevation it has a slope of 4.9% while its piers are of unequal height that ranges from 12 to 28 m. The piers’ section is a rectangular hollow one, the same for all pies (3.0m×5.8m). The deck is continuous, made of prestressed concrete and the total span reaches 482 m (36.7+9×45.5+35.5). The deck is a single-cell box girder (fig. 2) with 8 different sections along its length.

All piers are connected to the deck via elastomeric or PTFE bearings, which are considered as low damping bearings. Two fluid dampers in each direction (longitudinal and transverse) are used at the end abutments, where the only joints of the bridge deck are located. The dampers have practically hysteretic behaviour, since they realize a velocity exponent $a (F=C \cdot v^a)$ as low as 0.05. The maximum strength they provide is 750 kN and 450 kN per damper, in the longitudinal and transverse direction, respectively. The design of the actual bridge was carried out by the firm T. Tsiknias & Associates (Athens).
1.1 Codes and standards used in design

A combination of state of the art regulations has been necessary for the seismic design of this bridge. Firstly, the seismic action was defined according to the Greek National Codes EAK2000 [1] and E39/99 [2]. For issues, not covered by the former codes, namely seismic isolation and energy dissipation devices, EC8-Part2 [3], AASHTO [4] and FEMA 273 [5] were used. It is worth noting here that a new Greek Code for Seismic Isolation of Bridges was introduced in mid-2004. The design spectrum was defined as prescribed in E39/99 [2] (it is the
same one as in EC8 [3]). According to E39/99, the behaviour factor of this seismically isolated bridge, is assumed equal to 1.0 in all directions \( (q_x = q_y = q_z = 1.0) \), the importance factor \( \gamma_i = 1.3 \), maximum ground acceleration in the area (code value) is 0.16g, and soil conditions class is B. As far as the equivalent damping ratio for the structural system (not including the damping devices) is concerned, a parametric analysis was conducted here, using three values: 2%, 3.5% and 5%; values close to 5% are commonly used for R/C, while 2 to 3% is normally used for prestressed concrete.

1.2 Programs used for the analysis

Various advanced analysis programs were used for the analysis of the bridge. For the elastic analyses, as well as some inelastic ones (sections 2 and 3 of the paper) wherein bearings and dampers were modelled inelastically, the program SAP2000 [6] was used. For the full inelastic dynamic analyses, wherein potential inelastic behaviour of the bridge piers is considered, the program RUAUMOKO 3D [7] was utilized. Finally, synthetic accelerograms needed for the design, were generated using the program SIMQKE-I [8].

2 Equivalent elastic dynamic analysis

2.1 Modelling of the elastic elements of the structure

A 3D stick model was constructed which required 237 linear 3D beam elements. In each span a finer mesh was used close to the deck-pier connections, to accurately capture stress concentrations, as well as the deformation of the deck.

![3D view of the model](image)

Figure 3: 3D view of the model.

A parametric study was conducted to estimate the influence of flexural cracking in the piers on the seismic performance; also the deck’s torsional stiffness was reduced by 20% to account for minor cracking.
2.2 Modelling of the inelastic elements of the structure

Both bearings and dampers were modelled using joint–type spring elements, with 6 degrees of freedom, which, in the case of elastic analysis, remain elastic. Bearings were modelled using their actual height and effective properties for their five significant degrees of freedom, (their rotational stiffness around the vertical axis was considered negligible), according to Naeim and Kelly [9].

Dampers were modelled (in this analysis) through an equivalent viscous damping ratio and their secant stiffness at the maximum displacement as shown in figure 5, instead of directly modelling their hysteretic behaviour that is not feasible in an elastic analysis. Equivalent viscous damping ratio was calculated using equation (1), and assuming SDOF system behaviour in the longitudinal and MDOF behaviour (figure 4) in the transverse direction, respectively. Calculations in the second case were based on appropriately modified equations (2) and (3), derived from those of FEMA [5] which, however, refer to buildings.

\[
\zeta_d = \frac{\sum W_j}{2\pi W_K}
\]  \hspace{1cm} (1)

where the energy damped and the elastic energy of the structural system restored per response cycle (Figure 5) are respectively (in the following equation, \( F_{dj} \) is the force of the \( j \)-th damper):

\[
\sum W_j = 4 \cdot \sum F_{dj} (\Delta_{2,j} - \Delta_{i,j})
\]  \hspace{1cm} (2)

\[
W_K = \frac{4\pi^2}{T^2} \sum m_i \phi_i^2
\]  \hspace{1cm} (3)

![Figure 4: Transverse direction – MDOF approach.](image-url)

The dampers’ secant stiffness was estimated through an iterative procedure, in every step of which, the spectrum in each direction, was recalculated according to the damping ratio that corresponded to the predefined stiffness, and after the analysis the stiffness was re-evaluated so as to match the damper’s calculated displacement. Finally this (high) damping ratio was applied to the design spectrum only for periods greater than 0.8 \( T_{pr} \), according to EC8 [3].
(fig. 5), where \( T_{pr} \) is the predominant period in each direction. The equivalent damping ratios and the secant stiffness of the dampers derived from this procedure are shown in Table 1.

![Figure 5: a) Energy absorption and dissipation per cycle b) design spectra.](image)

Table 1: Damping (\( \zeta \) and \( \eta \)) and stiffness values.

<table>
<thead>
<tr>
<th>Equivalent Damping Ratio - Damping Reduction Factor</th>
<th>Damper secant stiffness</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \zeta_0 ) (%) - ( \eta )</td>
<td>( \zeta_{eff,x} ) (%) - ( \eta_x )</td>
</tr>
<tr>
<td>5.0</td>
<td>1.00</td>
</tr>
<tr>
<td>3.5</td>
<td>1.13</td>
</tr>
</tbody>
</table>

2.3 Modal analysis

The modal analysis was conducted using the aforementioned effective stiffness for the bearings and dampers, since their true values vary during response and consequently so do the dynamic characteristics of the system; this can always be useful for a critical and qualitative insight of the bridge’s seismic behaviour.

![Figure 6: 1st longitudinal mode - 2.35 sec.](image)

It is worth mentioning that the dominant period in the case of the rigid connection between piers and deck for this bridge was found to be around 0.65 sec.
2.4 Elastic dynamic analysis (modal superposition and time history)

Elastic analyses were conducted using the following approaches [6]:
- modal spectral analysis and the EC8 spectrum ($\zeta_{\text{eff}}$ from Tab. 1 and fig. 5b)
- modal time-history, i.e. modal analysis with peak values from the calculated response to 5 artificial records (fig. 10), ($\zeta_{\text{eff}}$ from Tab. 1 and fig. 5b)
- direct integration-based time history (Hilber – Hughes – Taylor algorithm)

All time history analyses were carried out for the generated artificial records (fig. 10), while in modal analyses, the equivalent damping values were used for each mode, separately. Because the number of the accelerograms used was less than 10, each mean value of the results was statistically adjusted with the correction factor $(1 + 0.352/\sqrt{N} = 1 + 0.352/\sqrt{5}) = 1.16$, suggested in EC8 Part 2 [3].

As expected, results (figure 9) from modal superposition (grey line) are in good agreement with adjusted mean values (MV) from modal history analysis (“1.16MV-modal history” line) and time history analysis (“1.16MV-time history” line). Moreover the seismic displacement in the longitudinal direction is about 12cm for the entire deck (hence SDOF behaviour), while in the transverse direction it ranges from 5 to 12 cm (hence MDOF behaviour). Moments at the bottom of the piers varied from about 60000 to 40000 kNm in the longitudinal and transverse directions, respectively.

The thin black line in figure 9 (1.16MV- no dampers) corresponds to the case that supplemental damping from the viscous dampers is ignored (calculated for comparison purposes only), i.e. the only difference between the two black lines is damping (secant stiffness of the dampers is considered in both analyses). It is therefore clear from figure 9, that equivalent damping in the case of elastic analysis uniformly reduces the bridge displacements. It will be shown from
inelastic analysis in the next section that this is not the case with the real
dynamic behaviour of the structure.

Figure 9: Displacements from elastic analysis.

Figure 10: Synthetic accelerograms used, and typical hysteresis loops for the
components of the passive system.
3 Inelastic analysis

At a second level, seismic analysis was repeated using a more advanced, but still not uncommon in the design of such important structures, method, i.e. inelastic time history analysis, wherein the response of the passive system is modelled through appropriate elastoplastic models (figure 10), while the rest of the structure is assumed to remain elastic (which is actually true, unless the earthquake intensity considerably exceeds the design one, as shown in the next section). Both time integration (Hilber – Hudghes – Taylor) and mode superposition algorithms (Fast Nonlinear Analysis) were used, in order to check the results. These analyses were based on five synthetic accelerograms, which were in good agreement with the design spectrum as shown in figure 10.

In line with the assumptions used for the design of the actual bridge, results from the time-history analyses were combined assuming that 100% of one horizontal artificial record acted simultaneously with 30% of the other one (and 30% of the vertical one); as already noted, mean values of the results were adjusted with the correction factor 1.16.

![Figure 11: Transverse displacements - elastic and inelastic analyses.](image)

As seen in figure 10, hysteresis loops for the dampers (Maxwell model) were almost rectangular because of the extremely low velocity coefficient they realise ($F_D = C \cdot v^a$, $a = 0.05$). For the elastomeric bearings, the hysteresis loop (Park et al. model [10]) was narrow due to the assumption of their low damping. For the PTFE bearings that were included in this analysis (Wen model [11]), their low maximum force and stiffness justifies why they were ignored in the ‘standard’ elastic analyses. This assumption was further confirmed by a parametric study,
wherein the bridge was analysed with and without considering the PTFE bearings, and differences in the resulting displacements were close to 1%.

In the longitudinal direction, all methods were found to be in good agreement, as displacements from elastic analysis were similar to those of inelastic analysis, i.e. uniform seismic displacements of about 12 cm. However, in the transverse direction (displacements from all cases are compared in fig. 11) it is evident that the more refined inelastic method (black line) results in displacements that differ substantially from those from elastic analysis (grey lines). This discrepancy is not so large with regard to the value of the maximum displacement, but rather in the distribution of calculated displacements along the bridge axis, i.e. values from elastic analyses grossly overpredict displacements near the abutments, where the dampers are located, and underpredict (to a lesser extent) the displacement close to the centre of the deck. This is attributed to the non–proportional nature of the damping mechanism that arises from the MDOF behaviour of the system in this direction. Furthermore, elastic analysis fails to produce slightly conservative results as one would expect from equivalent, code-type, analysis and, on the contrary, conveys the impression that the use of stronger dampers would have led to an improvement of the seismic performance. This is not true, though, because as results from inelastic analyses reveal, the displacements at the location of the dampers are minimal, hence the supplemental damping provided is practically the maximum feasible. Finally, it is worth noting that differences were not so important in terms of member forces, mostly because the response of the piers is little affected, due to the seismic isolation, by the deck’s movement.

4 Assessment of seismic performance

At the final stage of this study, full inelastic analyses were performed, wherein potential inelastic behaviour of the bridge piers was considered, in addition to all other features included in the inelastic dynamic analysis (section 3). The analyses were carried out for five natural records from Greek earthquakes, as well as for the aforementioned five artificial accelerograms (fig. 10), which were scaled to the same seismic intensity, i.e. design earthquake (0.2g) and twice the design earthquake (0.4g). Mean values for both the material properties and the calculated results were used, since the objective of deterministic assessment is to obtain the most probable response quantities.

From these analyses, which, due to lack of space herein, are presented in detail elsewhere (Dimitrakopoulos and Kappos [12]), it was found that for the design seismic intensity of 0.20g, the structural system (excluding the passive devices) remains in the elastic range for all records used in the analyses, including the natural ones that were not considered at the design stage. On the other hand, in the case of twice the design earthquake intensity (0.4g), representing a very low probability event (about 2%/50yr.), some critical sections of the piers enter the inelastic range; still, the calculated ductility demands are low, and the overall performance of the bridge is judged as adequate.
5 Conclusions

From the presented study of a real isolated bridge, it became clear that due to the high supplemental damping provided by the passive system, variations of structural damping values within the normal range considered in design (between 2 and 5%) do not affect the results when performing an equivalent elastic analysis.

Furthermore, it became clear that equivalent damping in elastic analysis decreases the response quantities in a uniform way. However, as clearly shown in the more refined (inelastic time-history) analysis, the dampers placed in the abutments, resulted in a non–proportional damping mechanism, which standard elastic analysis is incapable of capturing (figure 11), no matter how accurately the equivalent damping ratio is calculated. Hence, apart from the method to estimate an equivalent value for supplemental damping, in the case of passive systems the effect of damping non–proportionality should also be accounted for.

It is also worth noting that, worldwide, there is still a need for improvement of design codes tackling passive systems combining seismic isolation and energy dissipation devices. There is also much to be done to provide a rigorous regulatory platform for time-history-based design, such as the issue of combination rules in different directions in which the records are applied, as well as the issue of scaling natural records.

References
