ASSESSMENT OF SEISMIC PERFORMANCE OF SEISMICALLY ISOLATED BRIDGES WITH INVERTED T-CAP BEAMS

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ABSTRACT

Seismic performance of seismically isolated bridges with inverted T-cap beams are not studied in depth until now. In practice, design engineers usually prefer to utilize elastic response spectrum analysis in design. Such practical analysis method can not account for seismic pounding of adjacent spans. In this study, it has been demonstrated that analysis with elastic response spectrum method may result in significant underestimation of design forces. Nonlinear time history analysis performed with response spectrum compatible earthquake records are usually more representative of design forces with pounding effects. The pounding effect may be mitigated by use of different design details at bridges with inverted T cap beams.

Keywords: Inverted T Cap Beam, Seismic Isolation, Pounding
INTRODUCTION

Bridges, important piece of highway network systems shall remain in service after earthquakes for emergency use. In many countries, instead of demolishing and rebuilding structurally deficient old bridges, retrofitting with seismic isolation is performed successfully. There are distinct differences in design details of the bridges in Turkey and U.S although bridges in Turkey are designed per USA bridge specifications.

Inverted T cap beams supporting adjacent simply supported spans of highway bridges are very popular in Turkey, as shown in Figure 1. The 25 to 30 meters spans are formed by placing prestressed I girders over bearings located at bottom flanges of inverted T cap beams. Typically, a cast in-place concrete deck is placed over girders. The deck may be selected to be continous over the inverted T cap beams of the bridge pier. It is believed that such design may prevent pounding of adjacent girders to each other and any associated damage due to pounding.

The focus of this study is to evaluate the seismic performance of seismically isolated bridges with inverted T cap beam bridge piers. Typically, low damping elastomeric bearings are selected in design of Turkish highway bridges. As an alternative, use of friction pendulum system (FPS) and lead rubber bearings (LRB) was evaluated to determine if there is a shift in seismic response of bridges with inverted T cap beam bridge piers. In one other case called “multi (or mix)-type”, low damping elastomeric bearings (ELAS) are placed at abutments and lead rubber bearings are placed over piers.

The most popular and time efficient analysis method for evaluation of seismically isolated bridges is response spectra analysis, being an elastic dynamic analysis, it requires iterations for convergence on effective stiffness of the isolation system. In such analysis method, highly nonlinear behaviour of pounding of adjacent spans can be very difficult to model and can result in misleading evaluations. Even if it is not time efficient as response spectra analysis, nonlinear time history analysis can be the right tool to understand the highly nonlinear behaviour of pounding and isolation bearings.

Fig. 1 A Standard Highway Bridge Having Inverted T Cap Beam
DESCRIPTION OF THE BRIDGE

A 180 m long, standard highway bridge is modeled as shown in Figure 2. Geometric properties are presented in Figure 2.

Fig. 2 Model of the Bridge Used in the Analysis

<table>
<thead>
<tr>
<th>Definition</th>
<th>Property</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height of Pier 1</td>
<td>4 m</td>
</tr>
<tr>
<td>Height of Pier 2</td>
<td>6 m</td>
</tr>
<tr>
<td>Height of Pier 3</td>
<td>15 m</td>
</tr>
<tr>
<td>Height of Pier 4</td>
<td>15 m</td>
</tr>
<tr>
<td>Height of Pier 5</td>
<td>4 m</td>
</tr>
<tr>
<td>Diameter of the Pier X-Section</td>
<td>2.5 m</td>
</tr>
<tr>
<td>(Circular)</td>
<td></td>
</tr>
<tr>
<td>Span Length (Simply Supported)</td>
<td>30 m</td>
</tr>
<tr>
<td>Girders</td>
<td>Prestressed I140 (Spacing-140cm)</td>
</tr>
<tr>
<td>Cap Beam</td>
<td>Inverted T (X Section Area- 4.6m²)</td>
</tr>
<tr>
<td>Intermediate Expansion Joint</td>
<td>At Axis 4</td>
</tr>
<tr>
<td>Number of Spans</td>
<td>6</td>
</tr>
<tr>
<td>Deck Width</td>
<td>12 m</td>
</tr>
<tr>
<td>Deck Thickness</td>
<td>20 cm</td>
</tr>
</tbody>
</table>

Abutments with 3 meters height are considered to be rigid. Bridge superstructure consists of 9 equally spaced simply supported I-girders. In order to control transverse movement in case of a seismic event, shear keys are placed between I girders over the bottom flanges of inverted T cap beams. Initial gap between the shear keys and the I girders are considered as 10 cm.
In nonlinear time history analysis, lead rubber bearings and friction pendulum bearings are modeled having bilinear horizontal force-displacement relation as presented in Figure 3. Properties of the bearings used in the analysis are presented in Table 2.

![Bilinear Horizontal Force-Displacement Relation of the Lead Rubber Bearings and Friction Pendulum Bearings Used in Nonlinear Time History Analysis](image)

### Table 2 Properties of the Bearings

<table>
<thead>
<tr>
<th>Definition</th>
<th>Low Damping Elastomeric Bearing (ELAS)</th>
<th>Lead Rubber Bearing (LRB)</th>
<th>Friction Pendulum Bearing (FPS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dimensions</td>
<td>400mmx400mmx125mm</td>
<td>Φ350mmx107mm</td>
<td>-</td>
</tr>
<tr>
<td>Total Rubber Height</td>
<td>96 mm</td>
<td>77 mm</td>
<td>-</td>
</tr>
<tr>
<td>Hardness</td>
<td>60</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Shear Modulus</td>
<td>1.06 MPa</td>
<td>0.62 MPa</td>
<td>-</td>
</tr>
<tr>
<td>Axial Load (DL+0.5LL)</td>
<td>340 kN</td>
<td>340 kN</td>
<td>340 kN</td>
</tr>
<tr>
<td>Bulk Modulus</td>
<td>2000 MPa</td>
<td>1500 MPa</td>
<td>-</td>
</tr>
<tr>
<td>Elastic Stiffness</td>
<td>1750 kN/m</td>
<td>7350 kN/m</td>
<td>11400 kN/m</td>
</tr>
<tr>
<td>Post-Yield Stiffness</td>
<td>-</td>
<td>735 kN/m</td>
<td>670 kN/m</td>
</tr>
<tr>
<td>Characteristic</td>
<td>-</td>
<td>42.7 kN</td>
<td>27.2 kN</td>
</tr>
<tr>
<td>Strength</td>
<td>-</td>
<td>47.5 kN</td>
<td>28.9 kN</td>
</tr>
<tr>
<td>Yield Strength</td>
<td>-</td>
<td>47.5 kN</td>
<td>28.9 kN</td>
</tr>
<tr>
<td>Vertical Stiffness</td>
<td>550000 kN/m</td>
<td>425000 kN/m</td>
<td>3,865,000 kN/m</td>
</tr>
<tr>
<td>Yield Displacement</td>
<td>-</td>
<td>0.0065m</td>
<td>0.0025m</td>
</tr>
<tr>
<td>Ultimate Elongation</td>
<td>400%</td>
<td>500%</td>
<td>-</td>
</tr>
<tr>
<td>Diameter of Lead Core</td>
<td>-</td>
<td>80mm</td>
<td>-</td>
</tr>
<tr>
<td>Horizontal Force, d=20cm</td>
<td>350 kN</td>
<td>190 kN</td>
<td>161 kN</td>
</tr>
<tr>
<td>Radius of Curvature</td>
<td>-</td>
<td>-</td>
<td>50.8 cm</td>
</tr>
<tr>
<td>Friction Coefficient</td>
<td>-</td>
<td>-</td>
<td>0.08</td>
</tr>
<tr>
<td>Diameter of the Ball</td>
<td>-</td>
<td>-</td>
<td>100 mm</td>
</tr>
</tbody>
</table>
In nonlinear analysis, gap-compression only springs are used to simulate pounding between superstructure and cap beams in longitudinal direction; between superstructure and shear keys in transverse direction; between superstructure and backwall at abutments; and pounding at expansion joints. In addition, passive resistance of the soil behind the abutment is taken into account by using gap-compression only spring having properties as defined in Caltrans\(^2\). In modelling gap-compression only springs simulating pounding, crushing of concrete cover in both colliding surfaces are expected at ultimate state. Maximum pounding force is taken equal to 45% of the axial compressive strength of the pounding concrete surfaces. An exception is the shear keys, whose capacities are based on shear friction behaviour. Capacities of the backwalls are calculated according to flexural yielding at the bottom of the backwall at the ultimate state. It is well known that seismic pounding is a highly nonlinear and uncertain phenomenon. Therefore, in literature other methods and definitions may be found\(^3,4\). Force displacement relationship of a typical gap-compression only element is presented in Figure 4.

In response spectrum analysis, design spectrum given in the AASHTO Guide Spec.\(^1\) is used. Effective stiffnesses are used in order to model bilinear horizontal force displacement relationship of Lead Rubber Bearings (LRB) and Friction Pendulum Bearings (FPS). Hysteretic energy dissipation of these bearings are converted into equivalent viscous damping by using the formulas available in AASHTO Guide Spec.\(^1\). Then spectral acceleration values are reduced in accordance with the equivalent viscous damping for the isolated periods. Reduction in spectral accelerations should be made after 80% of the dominant period. The solution procedure is iterative for systems having bilinear or nonlinear horizontal-force displacement characteristics. First a displacement is assumed, effective period and damping are calculated according to this displacement and then the displacement corresponding to calculated period and damping is obtained. Iteration continues until assumed and calculated displacements come out to be close to each other. In this study, iteration is carried out until average of the differences between the effective stiffnesses of bearings is less than 10% in a step.

![Fig. 4 Force-Displacement Relation of a Typical Pounding Element](image)

In AASHTO Guide Spec.\(^1\), when three earthquake records are selected, maximum forces or displacements should be used in design. Selected earthquake records should be made response spectrum compatible per AASHTO Guide Spec.\(^1\). In this study, records obtained at Yarimca, Izmit and Gebze during 12 August 1999 and 17 November 1999 earthquakes in Turkey are used. Matching the average of the spectral accelerations of three
earthquakes and 1.30 times the design spectrum is shown in Figure 5. Basic properties of the earthquake records used in analysis are presented in Table 3.

![Fig. 5 Spectrums of Earthquake Records Made Response Spectrum Compatible](image)

Table 3  Basic Features of the Used Earthquake Records

<table>
<thead>
<tr>
<th>Record</th>
<th>Site</th>
<th>Condition</th>
<th>Distance to Fault (km)</th>
<th>PGA (g)</th>
<th>North-South</th>
<th>East-West</th>
<th>Vertical</th>
</tr>
</thead>
<tbody>
<tr>
<td>Izmit</td>
<td>Rock</td>
<td></td>
<td>4.26</td>
<td>0.167</td>
<td>0.227</td>
<td>0.149</td>
<td></td>
</tr>
<tr>
<td>Yarimca</td>
<td>Rock</td>
<td></td>
<td>3.28</td>
<td>0.322</td>
<td>0.230</td>
<td>0.291</td>
<td></td>
</tr>
<tr>
<td>Gebze</td>
<td>Rock</td>
<td></td>
<td>7.74</td>
<td>0.269</td>
<td>0.143</td>
<td>0.195</td>
<td></td>
</tr>
</tbody>
</table>

**ANALYSIS RESULTS**

In elastic dynamic analysis, models with LRB and FPS have less both superstructure displacements and substructure forces compared to the results of model with low damping elastomeric bearings. EQX is the longitudinal earthquake analysis and EQY is the transverse earthquake analysis. However, in some cases, for instance in 1.00EQX+0.3EQY combination, transverse forces in models with Lead Rubber Bearings (LRB) and Friction Pendulum Bearings (FPS) came out to be larger than results of models with low damping elastomeric bearings. The reason for this is the elastic behaviour of some isolators in the direction which design spectrum is multiplied by 0.3. High elastic stiffnesses of the isolators resulted in higher forces and moments. When seismic isolators (LRB,FPS) are used, reductions up to 50% in superstructure displacements and substructure forces were obtained compared to low damping elastomeric bearings. Column shear force diagrams are provided in Figure 6 and Figure 7.

Sometimes, designers may increase the stiffness of the bearings in longitudinal and transverse directions artificially in order to model restraining effects of inverted T cap beams and shear keys. In a case where horizontal stiffness of the elastomeric bearings are
increased up to 20 times of the original stiffness, results revealed that increasing the bearing stiffnesses artifically cause redistribution of the shear forces and moments in the piers. Hence, this method of modelling restraining actions of inverted T cap beams and shear keys become questionable.

In the following figures, some outputs of nonlinear time history analysis are presented.
The reason for the drastic difference between the shear forces in Figure 8 and Figure 9 is the pounding between superstructure and cap beam in Yarimca record. In Gebze record, substantial seismic pounding was not observed. It should be noted that during pounding, acceleration could be multiple times greater than gravitational acceleration. As obtained from analysis results, a dramatic increase in substructure forces and moments in longitudinal direction occur during seismic pounding between superstructure and inverted T cap beam. To a smaller scale, a similar effect was observed due to pounding between superstructure and shear keys in the transverse direction. Although energy dissipation due to seismic pounding is not modeled, it is not expected to change the results to an important extent. In case of response spectrum analysis and Gebze record, column capacities seems sufficient even with moderate longitudinal reinforcement. In Yarimca and Izmit records where extensive pounding was observed; column capacities seems almost insufficient even if they have 4% longitudinal reinforcement, being the upper limit at many codes. In case of pounding, although models with seismic isolators still have less forces and moments as compared to model with low damping elastomeric bearings, seismic isolation is not any more economical and efficient.

Fig. 9 FPS-Gebze Record Longitudinal Shear Force at Pier 5

A sample force displacement graph of a Lead Rubber Bearing (LRB) in Yarimca record is provided in Figure 10.

Fig. 10 Force-Displacement Graph of a LRB in Transverse Direction
CONCLUSIONS

Following results are obtained at the end of this study\(^5\):

- It may not be possible to model and include nonlinear effects such as pounding by using an elastic dynamic analysis, response spectrum.
- Seismic pounding between superstructure and cap beams, between superstructure and shear keys is the important parameter effecting the earthquake response of this bridge. In case of pounding, a possibility of transfer of large forces and moments to the substructure exists.
- In case of seismic pounding, seismic isolation may not be efficient and economical since superstructure is locked to the substructure at inverted T cap beam.
- In order to utilize seismic isolation efficiently in this kind of bridges, details reducing or eliminating pounding effect should be used. For example, gap between superstructure and cap beams or shear keys can be set greater than displacement capacity of seismic isolators.

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REFERENCES


