DESIGN AND CONSTRUCTION OF LRT VIADUCT ESENLER

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ABSTRACT

Esenler Viaduct is a single track LRT (Light Rapid Transit) viaduct located in Istanbul, Turkey. Total bridge length is 291 meters for Part-1 bridge and 128 meters for Part-2 bridge. Part-2 bridge has 5 spans with a maximum span length of 40 meters, whereas Part-1 bridge has 10 spans with smaller span lengths. Platform width of the viaduct is 5.30 meters. Part of the viaduct is horizontally curved with a minimum radius of curvature R=85 meters. A composite superstructure consisting of two steel U girders and cast in place R/C slab is selected to provide high torsional stiffness. To eliminate lateral torsional buckling risk of narrow top flanges during slab concrete pouring, horizontal bracing is also utilized. Previously constructed substructure of Part-1 bridge is retrofitted and upgraded due to its inadequate seismic performance. Since Istanbul is an earthquake prone city, which is likely to be affected by a major earthquake in near future, a seismic isolation system consisting of lead rubber bearings (LRB) is selected to reduce the seismic demands. Full scale dynamic prototype tests of the bearings are performed at EUCentre laboratory, with a maximum test velocity of 750 mm/sec. Construction of the viaduct has been completed in April 2013.

Key words: LRT, horizontally curved, seismic isolation, steel, composite

1. GENERAL

Esenler Viaduct is designed and constructed in scope of Istanbul Light Rapid Transit Project and is composed of two bridges of 291 meters (Part 1) and 128 meters (Part 2) total length. Superstructure is 5.30 meters wide and includes single directly fixed track. Substructure of Part-1 bridge was constructed about 25 years ago and it was never used. Seismic performance of previously constructed part of the viaduct does not satisfy the requirements of current or modern codes such as AASHTO 2011. Therefore, the project involves both new design and seismic upgrading of Part-1 bridge substructure. There are 10 spans in Part-1 bridge with a maximum span length of 33 meters, whereas Part-2 bridge has 5 spans with a maximum span length of 40 meters. The superstructure is continuous for both bridges and expansion joints are only present at abutments. By selecting a continuous superstructure, unseating risk of girders during an earthquake is eliminated and the service performance of the viaduct is improved by reducing the vertical deflections and accelerations under live load.
2. SUPERSTRUCTURE

The superstructure is composed of two steel U girders with variable depth and a 20 cm thick cast in place R/C slab. Cross-section of the superstructure is presented in Figure 1. Since superstructures of Part-1 and Part-2 bridges are similar, only superstructure details of Part-2 bridge are discussed in detail. In design of the superstructure and substructure AASHTO 2007&2011 specifications are used.

![Figure 1: Cross-Section of the Superstructure (Dimensions in mm)](image1)

Steel girder depth of Part-2 bridge varies due to different span lengths (28+20+20+40+20 meters), as presented in Figure 2. Maximum steel girder depth is 1500 mm and minimum girder depth is 950 mm. Sharp horizontal curvature of the structure (curvature radius as small as 85 meters) necessitated selection of a torsionally stiff superstructure. Due to easiness of fabrication, transportation and construction such a composite superstructure is preferred.

![Figure 2: View of Part-2 Bridge](image2)
Train to be utilized in the line is ABB train, whose properties are presented in Figure 3 and Figure 4. Details about the ABB train are provided by owner of the bridge.

**Figure 3: Properties of ABB Train**

<table>
<thead>
<tr>
<th>Mass and Passen. Capac.</th>
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<tbody>
<tr>
<td>6 passenger per m²</td>
<td>257(226+32)</td>
</tr>
<tr>
<td>tare(Empty vehicle)</td>
<td>30,000 kg</td>
</tr>
<tr>
<td>tare/m</td>
<td>1,293 kg/m</td>
</tr>
<tr>
<td>Full limit mass (pers)</td>
<td>49,000 kg</td>
</tr>
<tr>
<td>Full limit mass/m</td>
<td>2,112 kg/m</td>
</tr>
</tbody>
</table>

**Bogies**

- Empty axle load: 5,000 kg
- Full axle load: 7,560 kg
- Max. Design axle load: 8,000 kg
- Design axle load: 8,200 kg
- Wheel diameter (new-worn): 600-602 mm
- Wheel width: 125 mm
- Wheel type: Bandaji Khi Tip
- Distance between wheel center: 1,600 mm
- Weight of motor bogie: 4,500 kg
- Weight of bogie without motors: 3,100 kg
- Gear ratio: 6.92

**Figure 4: ABB Train Loads**

![Graph showing point load vs. distance with a peak at 175.5 kN]
Structural analyses are performed using Larsa 4D analysis software (Larsa Inc. 2007). Two different models are used in analyses and design of superstructure. In the first model, girders, braces and cross-frames are modeled using frame elements, whereas R/C slab is modeled using 4-node shell elements. Overview of this analysis model is presented in Figure 5. This model is used for analyzing overall behaviour of the structure and designing U girders and R/C slab. Bearings are modeled using linear springs.

![Figure 5: Overview of the Computer Model-1-Part 2 Bridge](image)

In the second model, shell elements are used for modeling U girders and diaphragm beams as well as R/C slab. Cross-frames and horizontal brace elements are modeled using frame elements. This model is used for analyzing stress concentrations and designing diaphragm beams, cross frames and bracing elements.

![Figure 6: Overview of the Computer Model-2-Part 2 Bridge](image)

Construction stages representing real construction scenario is taken into account by performing construction stage analysis. Time dependent behavior of slab concrete is included in the calculations by reducing its stiffness to $1/3$ of the original value. In negative moment regions, contribution of slab is almost ignored.
Analysis results indicate that each of the steel U girders has enough capacity to resist factored flexural moments and shear forces within the stress limits, without sharing load between each other. Cross-frames and diaphragm beams are only functional under unsymmetrical loading, which results in torsional action.

During slab concreting, in order to control negative moments, first concrete of positive moment regions are poured followed by negative moment regions, as presented by Figure 7. Similar concrete pour order can be found in SETRA Guidance Book: "Eurocodes 3&4-Application to Steel-Concrete Composite Road Bridges" (SETRA. 2007).

In design, details having fatigue class higher than C₀(AASHTO.2007) are preferred since fatigue is known to be critical in railway bridges due to repetitive nature of railway loading. Infinite fatigue life of the bridge is provided by keeping design stress range less than one-half of the constant amplitude stress threshold. In fatigue checks, ABB train with a load factor of 0.75×1.15 is used.

![Figure 7: Slab Concrete Pouring Sequence (Dimensions in cm)](image)

Slab concrete has 28 day compressive strength of 35 MPa and total weight of structural steel is 1168 tons (Part 1 and Part 2 bridges).

Due to sharp horizontal curvature and to increase load sharing capability between girders, spacing of K type cross frames (L80.80.8 angles) is kept around 1.5-2.5×girder depth. At negative moment regions, stability of wide bottom flange in compression is increased by using longitudinal stiffeners in the mid of the cross-section. Longitudinal stiffeners have T cross-section with D=500 mm, t_{web}=10.2 mm, t_{flange}=16 mm and b_{flange}=200 mm.

View of girders during fabrication is presented in Figure 8. Main girders are manufactured from S355 JR grade steel (f_{y}=355 N/mm²), while angles and U sections are manufactured from S235 JR grade steel (f_{y}=235 N/mm²). Web thickness of the girders is in between 15-20 mm and flange thickness is in between 30 to 80 mm.

To increase stability of relatively narrow top flanges (b=400 mm) during slab concrete pouring, which is known to be the critical activity in horizontally curved bridges, horizontal bracing is utilized in order to decrease out of plane deformations of the top flange, as presented in Figure 9 (Tawil et al. 2002). L100.100.10 angle profiles are used as horizontal brace elements.

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At piers and abutments, as presented in Figure 10 and Figure 11, very stiff diaphragm beams are designed in order to increase total stiffness of the superstructure under live and seismic loading. As presented in Figure 10, there are four bearings at each pier and abutment. The diaphragm beam is designed as an I girder and it also continues inside U girders.

Construction of the superstructure took about 6 months and total construction time of the bridge is about 15 months (Part-1 and Part-2 bridges). Heavy highway and LRT traffic beneath the bridge sometimes necessitated night work, especially during placement of steel U girders. Construction of the bridge has been completed in April 2013 and it is in use at the moment.
3. SUBSTRUCTURE AND SEISMIC DESIGN

Istanbul is an earthquake prone city and in near future a maximum credible earthquake (MCE) is expected with magnitude of 7.5. The site is about 20 km away from North Anatolian Fault. Site specific earthquake hazard studies for the site are performed and seismic design spectrum presented in Figure 12 is obtained. Deep stiff soil layers consisting of sand, silt and clay are present at the site.

Seismic resistance and performance of the existing substructure of Part-1 bridge is insufficient according to modern codes (AASHTO, 2011), so seismic upgrading is necessary.

Seismic performance of the bridges (Part-1 and Part-2 bridges) is improved by using seismic isolation system composed of lead rubber bearings (LRB) so as to decrease seismic demand on the structure by increasing vibration period and energy dissipation provided by the lead core. Energy dissipation capacity provided by the lead core reduced spectral accelerations at the isolation period more than 40%. Moreover, lead core provide service load rigidity under braking and wind forces. Characteristic strength of the lead core is chosen accordingly so as to limit horizontal displacement of the superstructure under wind and braking loads.
Corrosion of the reinforcement of the existing substructure is also a major issue in addition to substructure's inadequate seismic performance. Corrosion of pier reinforcement is presented in Figure 13. The substructure that is already in place is retrofitted by increasing ductility of the piers with supplementary steel confining plates with a thickness of 15 mm, and increasing the flexural and shear capacity of the footing by increasing its thickness, as presented in Figure 14 and Figure 15 respectively. The structure is designed for 2475 year return period earthquake with R=1.5 for the piers and R=1 for the foundation. Hence, seismic inelastic demand from the piers is very limited.

Figure 13: Reinforcement Corrosion of the Existing Pier

Figure 14: Supplementary Confinement Provided By Additional Steel Plates

Figure 15: Footing Retrofitting (Dimensions in cm)
Linear dynamic analysis results indicate that design displacement of lead rubber bearings is ±200 mm. Bearings are 510 mm in diameter and have a total total height of 216 mm. Shear modulus of the rubber is selected intentionally low to decrease secondary stiffness of the bearing. Diameter of the lead core is 112 mm. Mechanical properties and dimensions of the isolation bearings are presented in Figure 16 and Figure 17, respectively. The tests of the bearings were performed at EUCentre laboratory Milano, Italy with a maximum dynamic test velocity of 750 mm/sec (EUCentre. 2012).

Figure 16: Mechanical Properties of Selected LRB

Figure 17: Dimensions of Selected LRB (Dimensions in mm)
4. CONCLUSION

In Esenler LRT Viaduct, a composite superstructure consisting of two steel U girders and cast in place R/C slab is selected to increase torsional stiffness of the superstructure. The stability of the top flanges during slab concrete pouring is increased by horizontal bracing. Closely spaced K type cross frames are utilized inside and in between U girders.

The viaduct is designed for MCE (maximum credible earthquake) with a return period of 2475 years. Previously constructed substructure which does not satisfy the requirements of modern seismic codes is retrofitted by supplementary steel plates so as to increase ductility of the columns and by increasing the thickness of the footing.

Seismic demand on the substructure is reduced by using seismic isolation system composed of lead rubber bearings. The design displacement of lead rubber bearings is ±200 mm. Lead rubber bearings also provide high lateral service load rigidity under braking and wind forces, which improves service load performance of the bridge.

5. REFERENCES

AASHTO. 2007. AASHTO LRFD Bridge Design Specifications, USA.
AASHTO. 2011. AASHTO LRFD Seismic Bridge Design, USA.
EUCentre. 2012. Characterization Tests on Alga LRB D 500-216, Test Report, Pavia-Italy
LARSA Inc. 2007. Larsa 4D. USA.
SETRA. 2007. Guidance Book: Eurocodes 3 and 4- Application to Steel-Concrete Composite Road Bridges, France.
Tawil, S.E. and Okeil A.M. 2002. Behavior and Design of Curved Composite Box Girder Bridges, Final Report, University of Central Florida USA.