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Assessment of Performance of Bolu Viaduct in the 1999 Duzce Earthquake

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Summary

The Bolu viaduct is a 2.3 km long seismically isolated structure whose construction was nearly complete when it was struck by the 1999 Duzce earthquake in Turkey. It suffered complete failure of the seismic isolation system and narrowly avoided total collapse due to excessive superstructure movement. This paper presents an evaluation of the design of the viaduct's seismic isolation system and an assessment of its performance in the Duzce earthquake. The evaluation revealed that the seismic isolation system did not meet the requirements of the AASHTO Guide Specifications for Seismic Isolation Design. Analysis of the viaduct with motions scaled in accordance to the AASHTO Guide Specifications resulted in a displacement demand equal to 820 mm, which is far more than the 210 mm displacement capacity of the existing isolation system. Analysis of the viaduct for a simulated near-fault ground motion with characteristics consistent with the site conditions resulted in an isolation system displacement demand equal to 1400 mm. This indicates that even if the isolation system had been designed in compliance with the AASHTO Guide Specifications it would have still suffered damage in the earthquake.

Introduction

The Bolu viaduct (Figure 1) forms a small segment of the Trans European Motorway (TEM), which runs from Turkey's capital city (Ankara) to Europe parallel to the North Anatolian Fault (NAF). The viaduct is a seismically isolated structure whose construction was nearly complete at the time of the 1999 Duzce earthquake. It suffered complete failure of the seismic isolation system and narrowly avoided total collapse due to excessive superstructure movement.

This paper presents an evaluation of the design of the viaduct's seismic isolation system and an assessment of its performance in the Duzce earthquake. For more details, the reader is referred to the report by Roussis et al., (2002). The outcome of this study becomes important in validating the AASHTO Guide Specifications (AASHTO 1991, 1999) and in developing experience in the seismic behavior of this type of structures.

Description of the Viaduct

The viaduct is a typical 10-span segment (Figure 2) and it incorporates an energy dissipation system in the form of yielding steel devices (YSD) installed on each pier cap to form, together with multi-directional sliding bearings, a seismic isolation system (Marioni 1997, 2000). Details of the YSD are displayed in Figure 3. The YSD is composed of an inner and outer ring interconnected by sixteen steel C-elements in a radially symmetric configuration. The inner and outer rings are connected to the substructure and the superstructure, respectively. As the superstructure moves relative to the substructure, the C-elements deform, yield and dissipate energy. Moreover, shock transmission devices are incorporated between the YSD and the substructure in the longitudinal viaduct direction. This system allows free longitudinal movement of the superstructure relative to the substructure due to creep, shrinkage, and temperature change, and locks up under high-speed movement to engage the YSD (Ciampi and Marioni 1991, Tsopelas and Constantinou 1997).

Figure 1. General View of Bolu Viaduct

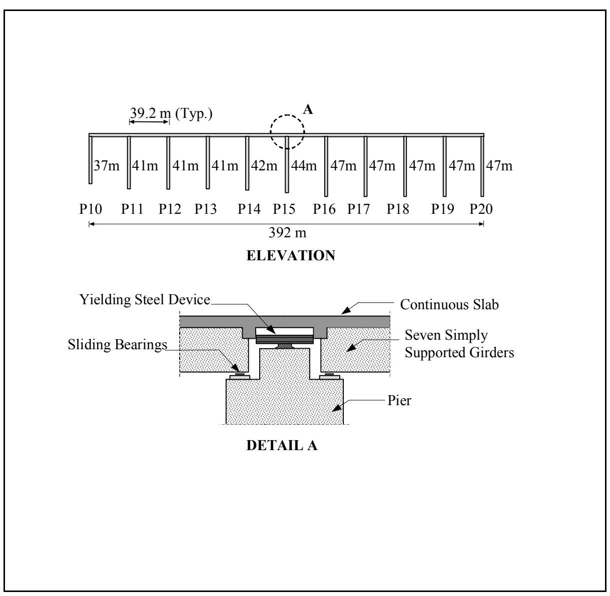

Figure 2. Elevation of viaduct at piers

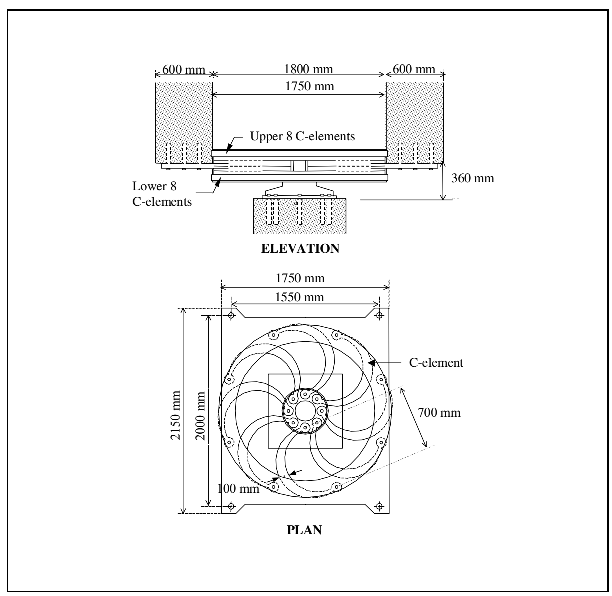

Figure 3. Details of yielding steel device

Design of the Seismic Isolation System

Turkey is an affiliated AASHTO State. However, the design concept for the viaduct involved the application of mixed criteria using the then applicable AASHTO Standard Specifications and seismic isolation guidelines developed by the designer. The design seismic forces and displacements were determined by means of nonlinear time history analyses performed using seven uni-directional artificial accelerograms. The structure was designed for the mean values of maximum forces and displacements obtained from the seven analyses (Marioni 1997, 2000).

Based on test results of the YSD, the seismic isolation system was modeled assuming a bilinear hysteretic behavior as shown in Figure 4 (Astaldi SpA 2000). This behavior is the combination of the inelastic behavior of the YSD and of the sliding bearings.

The design displacement of the seismic isolation system was calculated as equal to 320 mm. However, the bearings had a displacement capacity equal to only 210 mm and the YSD had an ultimate deformation capacity equal to 480 mm (Marioni 1997, 2000). Clearly, there is an apparent inconsistency in the design of the seismic isolation system as the displacement capacity of the bearings is much smaller than the design displacement, while the ultimate deformation capacity of YSD is larger.

It should also be noted that the assumed post-elastic stiffness does not meet the criteria for the lateral restoring force of either the 1991 or the 1999 AASHTO Guide Specifications. A direct application of the minimum requirements of the 1991 AASHTO Guide Specifications resulted in a required displacement capacity equal to 790 mm. This figure is greater than both the 210 mm displacement capacity of the installed system and the 320 mm calculated response. Moreover, the 1999 AASHTO Guide Specifications require that three-dimensional nonlinear dynamic analysis must be used if the seismic isolation system has insufficient restoring force capability. Such an analysis requires at least three pairs of horizontal ground motion histories selected from recorded events and scaled to represent the applicable response spectrum. Yet, the design of the structure was performed using only uni-directional ground motion inputs ignoring the three-dimensional effect of pairs of ground motion components simultaneously applied on the structure.

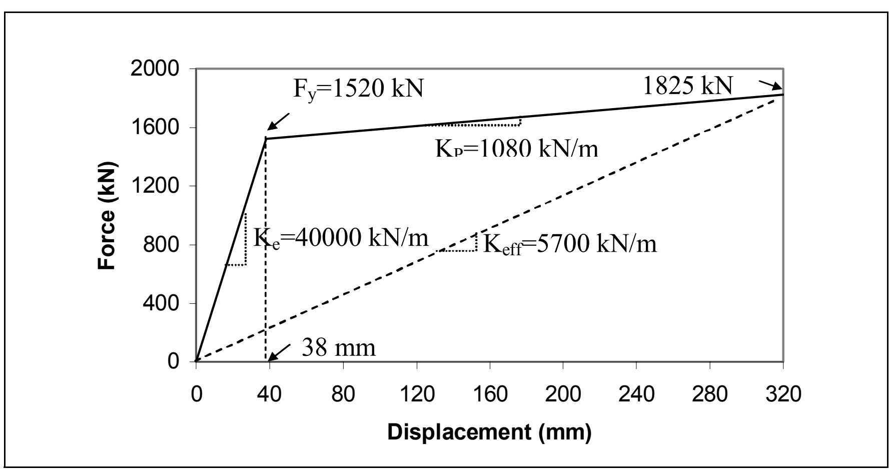

Figure 4. Assumed bilinear hysteretic behavior of seismic isolation system

Damage to Seismic Isolation System in the 1999 Duzce Earthquake

The viaduct was subjected to a near-fault, pulse-type ground motion as the ruptured fault crossed the viaduct at 25-degree angle with respect to the viaduct's longitudinal direction. Thus, the structure might have been exposed to significant directivity and fling effects, with associated large velocity pulses and permanent tectonic deformations. Consequently, the viaduct superstructure experienced a westward permanent displacement relative to the piers, leaving all ends of the girders offset from their supports in the order of 1000 mm longitudinally and 500 mm transversely. At nearly all locations, the sliding bearings suffered complete failure with their parts dislocated and ejected from the bearing pedestals. The observed scratch signs on the surface of the bearing's stainless steel plates resembled the number six, indicating that the bearings slid off probably at a very early stage before any significant cyclic movement.

Analysis of the Viaduct

A typical 10-span segment of the viaduct (Figure 2) was analyzed using the finite element program ANSYS (SAS 1996). The analysis was based on a three-dimensional finite element model. Nonlinear dynamic analyses were carried out to estimate the response of the viaduct in the 1999 Duzce earthquake. The two records at Bolu and Duzce were first used to assess the performance of the viaduct during the Duzce earthquake. However, it was recognized that neither of the two records truly represent the conditions experienced by the viaduct in the vicinity of the fault. Accordingly, the ground motion at the viaduct's site was simulated based on stipulated earthquake source parameters and it was then used for the assessment of the performance of the viaduct.

In addition, nonlinear time history analyses were performed to calculate the design displacements for the isolation system in compliance with the AASHTO Guide Specifications. The calculated displacements were used to determine whether the structure would have survived the earthquake had it been designed in compliance with the AASHTO Guide Specifications. The analyses were conducted using three pairs of scaled horizontal ground motions selected from different recorded events. These ground motions are: (i) 360 and 270 components of 1992 Landers earthquake recorded at Yermo station; (ii) 90 and 0 components of 1989 Loma Prieta earthquake recorded at Hollister station; and (iii) S00W and S90W components of 1971 San Fernando earthquake recorded at station 458.

Analysis of Results

Calculated maximum resultant displacements of the isolation system at each pier are summarized in Table 1. Note that the calculated peak displacement response of the isolation system in the simulated near-fault motion is on the order of 1400 mm. While this figure may be a conservative estimate, it demonstrates that the demand in the Duzce earthquake far exceeded the capacity of the isolation system. Furthermore, results for ground motions scaled in accordance to the AASHTO Guide Specifications demonstrate that the displacement capacity of the isolation system should have been equal to at least 820 mm. This value is substantially larger than the 210 mm capacity provided to the isolated viaduct.

Figure 5 presents graphs of the calculated displacement paths of the isolation bearings at Pier 15 during the initial portion of movement for the simulated, Bolu station, and Duzce station ground motions. The calculated paths of the isolation system displacement for the Bolu station and the simulated near-fault ground motions consist of a half cycle of displacement whose amplitude is equal to about 100-150 mm, followed by movement in the opposite direction that exceeds the displacement capacity of the bearings. Observations of the scoring on the stainless steel plates of the sliding bearings of the viaduct are consistent with the calculated displacement paths. In contrast, the calculated paths of the isolation system displacement for the Duzce station ground motion show no resemblance to the observed traces on the stainless steel plates of the bearings.

Table 1. Calculated maximum isolation system resultant displacements

Ground motion	Maximum Isolation System Resultant Displacement (mm)											
	P10	**P11**	**P12**	**P13**	**P14**	**P15**	**P16**	**P17**	**P18**	**P19**	**P20**	**Max**
Bolu	325	325	329	337	344	346	342	355	367	380	393	393
Duzce	530	525	521	516	515	514	504	511	513	511	509	530
Simulated	1297	1291	1302	1308	1322	1336	1354	1375	1391	1409	1425	1425
Scaled Yermo	487	489	496	506	514	522	531	543	557	569	581	581
Scaled Hollister	644	743	742	734	756	790	820	818	815	812	810	820
Scaled San Fernando	325	341	353	363	372	379	379	385	388	392	397	397

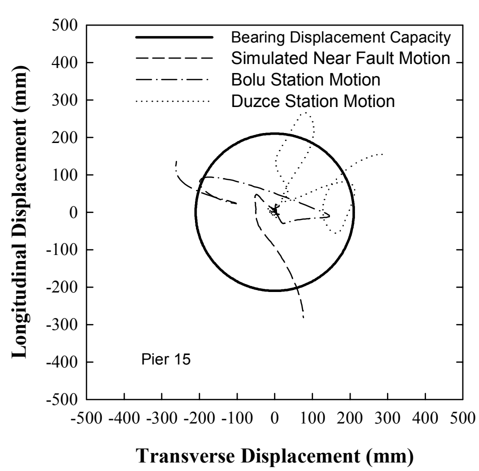

Figure 5. Isolation system displacement paths at Pier 15

Conclusions

Results of this study demonstrate the following: (1) the seismic isolation system of the Bolu viaduct did not meet the requirements of either the 1991 or the 1999 AASHTO Guide Specifications; (2)

analysis of the seismically isolated viaduct with motions scaled in accordance to the 1999 AASHTO Guide Specifications resulted in a required displacement capacity equal to 820 mm. Moreover, a direct application of the minimum requirements of the 1991 AASHTO Guide Specifications resulted in a displacement capacity equal to 790 mm. Either value of the displacement capacity is substantially larger than the 210 mm capacity provided; (3) analysis of the viaduct for the Duzce and Bolu records resulted in isolation system displacements that exceeded the displacement capacity by a factor of two or more. The Bolu record contained clear near-fault characteristics and the calculated displacement paths closely resembled the observed paths. However, realistic results are believed to have been obtained in the analysis for a simulated near-fault ground motion that resulted in isolation system displacement demands of up to 1400 mm; and (4) it appears that had the isolation system been designed in accordance with the AASHTO Guide Specifications to have a displacement capacity equal to 820 mm, it would have still suffered damage in the Duzce earthquake, given that the displacement demand was likely of the order of 1400 mm.

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References

AASHTO (1991, 1999): *Guide specifications for seismic isolation design*. American Association of State Highway and Transportation Officials, Washington, DC.

SAS (1996): *ANSYS User's Manual, Version 5.3*. Swanson Analysis Systems, Inc., Houston, PA.

Astaldi SpA (2000): Miscellaneous documents and diagrams related to the Bolu Viaduct in Duzce, Turkey.

Ciampi V, Marioni A (1991): New types of energy dissipating devices for seismic protection of bridges. *Proceedings of the Third World Congress on Joint Sealing and Bearing Systems for Concrete Structures*, National Center for Earthquake Engineering Research, **2**, 1225-1245.

Marioni A (1997): Development of a new type of hysteretic damper for the seismic protection of bridges. *Proceedings of the Fourth World Congress on Joint Sealing and Bearing Systems for Concrete Structures*, American Concrete Institute, **2**, 955-976.

Marioni A (2000): Behaviour of large base isolated prestressed concrete bridges during the recent exceptional earthquakes in Turkey. *Proceedings of the Seminar "Difendiamoci dai terremoti: le piu recenti applicazioni italiane delle nuove tecnologie antisismiche d'isolamento e dissipazione energetica"*, Bologna, Italy.

Roussis PC, Constantinou MC, Erdik M, Durukal E, Dicleli M (2002): Assessment of performance of Bolu viaduct in the 1999 Duzce earthquake in Turkey. *Technical Report MCEER-02-0001*, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, NY.

Tsopelas P, Constantinou MC (1997): Study of elastoplastic bridge seismic isolation system. *ASCE Journal of Structural Engineering*, **123** (4), 489-498.