

## ASSESSMENT AND DESIGN OF SEISMICALLY ISOLATED BRIDGES IN PROXIMITY OF MAJOR FAULTS AND LOCATED ON LIQUEFIABLE SOILS

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**Abstract.** *In this paper, seismic isolation design of one highway viaduct and one high-speed railway viaduct located within few hundred meters of North Anatolian fault in Turkey is studied. Strong near-field characteristics such as directivity and liquefaction are expected due to soft soil conditions under the scenario earthquake. Seismic assessments of viaducts are performed considering these unfavorable effects.*

*For the highway viaduct, a hybrid seismic isolation system is chosen for the bridge consisting of four natural and two lead-rubber bearings supporting the six steel-girders at each substructure. Lead cores in the lead-rubber bearings are designed to have an overall characteristic strength equal to 10% of the superstructure weight. This large characteristic strength is intentionally chosen to limit the isolator displacements under near-field effects. Furthermore, the supplementary four natural rubber bearings over each substructure provided additional stiffness to achieve a small post elastic period.*

*For the railway viaduct, the same design philosophy was followed with considering the serviceability issues related to high-speed railway bridges. Since yield displacement of lead-rubber bearings are about 2-3 cm, which is unsatisfactory for stringent displacement limits specified in international railway codes, in this case curved surface slider bearings having a dynamic friction coefficient of 12% with 2.54 mm yield displacement are selected.*

## 1 INTRODUCTION

Bozuyuk-Mekece state highway is connecting the cities in the Mediterranean region and internal Anatolia to Istanbul, Turkey. Upgrading the highway was planned due to heavy traffic and accidents. Total length of the project is about 85 km and the project includes many tunnels, bridges and overpasses. Sakarya-II viaduct is the most important structure of this project since half of the project is at one side of Sakarya river and other half at the other side. Sakarya-II viaduct is the key element connecting the halves. Highspeed railway line between Ankara and Istanbul follows almost the same route with Bozuyuk-Mekece highway.

Turkey is an earthquake prone country and about 95% of the population is susceptible to earthquakes. Most of the earthquakes in Turkey take place on North Anatolian fault. North Anatolian fault is about 1500 km long and crosses the country from east to west. The fault is a strike-slip fault having characteristics similar to San Andreas fault. Destructive earthquakes up to  $M_w = 7.5$  occurred on the fault-line resulting in many life and economical losses. Recent earthquakes on the North Anatolian fault are 17 August 1999 Izmit ( $M_w = 7.4$ ) and 12 November 1999 Duzce ( $M_w = 7.2$ ) earthquakes.

The Sakarya-2 viaduct and highspeed railway viaduct VK-14A is very close to each other as shown in Figure 1.

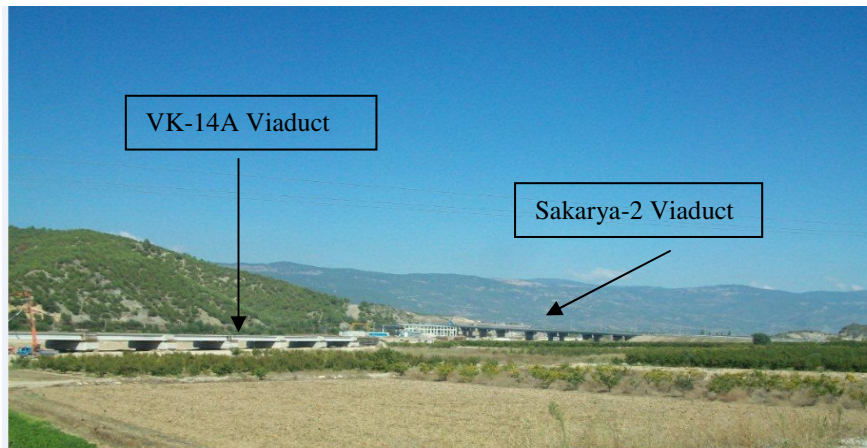


Figure 1: General View of the Highway and Railway Viaducts at Pamukova – Turkey

## 2 SEISMICITY AND GEOTECHNICAL PROPERTIES OF THE SITE

North Anatolian fault has two branches at Marmara region. The northern branch is more active compared to southern one and 17 August earthquake occurred on this branch. Northern branch is very close to TEM highway and Istanbul. On the other hand, part of the southern branch between Geyve-Iznik is not active for centuries. Therefore, this portion of the fault may be classified as seismic gap. Sakarya-II Viaduct is about 300 meters far from the southern branch of the North Anatolian fault. Historical seismicity of the region and bridge site is presented in Figure 2. In Figure 2, earthquakes with magnitudes larger than 4.0 are highlighted.

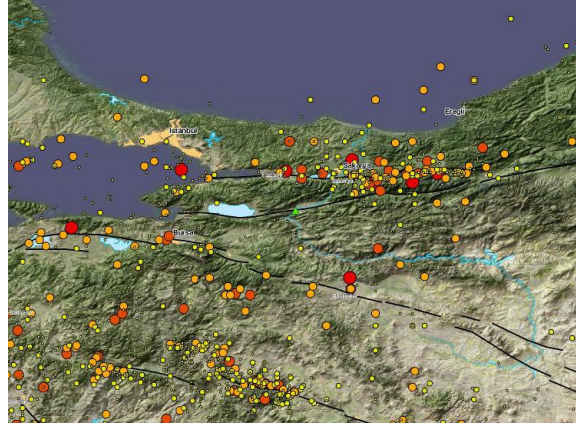


Figure 2: Seismicity of the Marmara Region and Bridge Site (1900-2006) [1]

Site specific studies have indicated that scenario earthquake having a magnitude of  $M_w=7.5$  may be expected at the site during the economic life of the bridge. Return period of the maximum credible earthquake was selected as 1000 years. By using the attenuation relationships, peak ground acceleration at the bedrock level is calculated as  $0.96g$  [1]. However, there is little seismic data for sites that are closer than 3 km to a major fault line.

Due to close proximity of the bridge to the fault, strong directivity effects are expected at the bridge site during a major earthquake. Near-fault earthquakes contain significant wave pulses. For strike-slip faults, they dominate the horizontal motion and may appear as single or double pulses with single or double-sided amplitudes. The period of the main pulse is in between 0.5sec-5 sec [2]. These pulses result in very high displacements and forces.

Elastic response spectrum analysis cannot capture strong nonlinear characteristics of near-field earthquakes. Therefore, nonlinear time history analyses were utilized in design of the bridge. Seven near-field earthquake records were selected and scaled according to AASHTO Guide Spec [3]. Records [1] are presented in Table-1.

Earthquake	Station Code	$M_w$	$PGA_{EW}$	$PGA_{NS}$	$PGV_{EW}$	$PGV_{NS}$
1976-Gazli-Uzbekistan	Karakyr	6.8	1.74	2.05	187	205
1994- Northridge-USA	Jensen	6.7	1.01	1.82	135	119
1994- Northridge-USA	La Sepulveda	6.7	2.74	3.41	285	277
1995 – Kobe, Japan	Takatori	6.9	0.49	0.50	103	98
1999- Duzce, Turkey	Bolu	7.2	2.05	2.31	158	175
Northridge -01	Sylmar	6.69	0.76	1.11	146	126
1999- Chi Chi, Taiwan	TCU 065	7.62	1.00	0.74	155	97

Table 1: Earthquake Record Utilized In Analysis [1]

Conformance of ensemble spectrum formed by taking the average of the SRSS spectra for the individual earthquakes and  $1.3 \times$  Code spectra is presented in Figure 3. It is observed from the figure that average of the earthquakes match the code spectrum at intermediate periods corresponding to isolation period. Scaling factors are selected for each record so as to match the code spectra in between 1.5sec-2 sec. On the other hand, as presented in Figure 4, at short periods, spectra of some records do not fit well to the code spectra. Since short periods represent vibration periods of the substructure, this unfit may result in overestimation or underestimation of substructure forces.

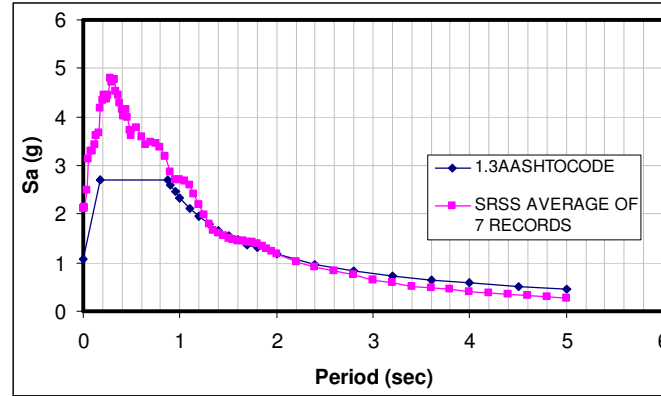


Figure 3: Code Spectrum and Average Spectrum of 7 Earthquakes

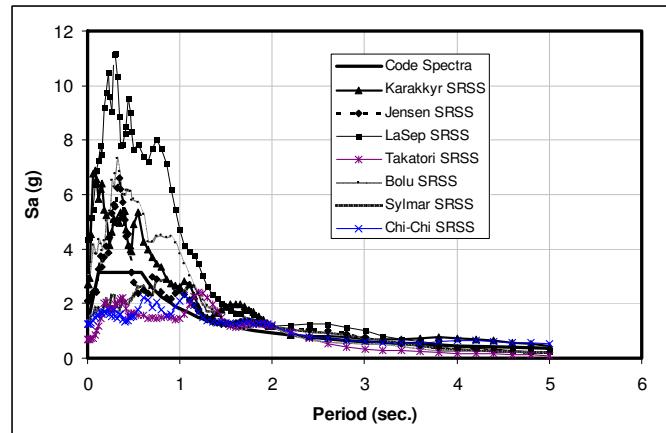


Figure 4: Code Spectrum and SRSS of Selected Earthquake Records

Upper 30 meters of the soil at the bridge site consists of sand, gravel and silt. SPT values increase as depth increases. SPT values ( $N_{1,60}$ ) in between 15-30 are common at the upper 15-20 meters. Moreover, level of water table is close to the free field since the viaducts are on Sakarya river. Liquefaction triggering is a function of ground motion intensity and characteristics as well as geotechnical parameters [4]. Procedure proposed by Seed et al. [4] was utilized for establishing the liquefaction susceptibility of the site. Results have revealed that the site is susceptible to liquefaction during a major earthquake due to high level of seismic input.

In order to verify susceptibility of the bridge site to liquefaction, equivalent linear dynamic analyses were performed by using software Proshake [5]. Scaled earthquake records and geotechnical parameters of the site were set as input. Analyses results have indicated that lateral stiffness of the soil strata between 0-10 meters from the top decreases to almost zero during a major earthquake as presented in Figure 5.

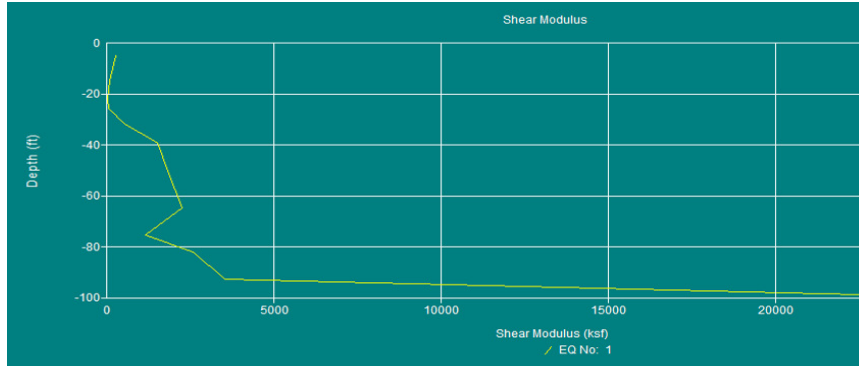


Figure 5: Equivalent Shear Modulus during Expected Earthquake

### 3 SAKARYA-2 VIADUCT

Sakarya-II viaduct consists of two twin bridges. Two bridges have very similar span lengths and substructures. The bridges have nine spans with a total length of 384 meters for the longer one and 365 meters for the shorter one. Longest span length is 52 meters and shortest span length is 32 meters. The viaduct has a horizontal curvature near one of the abutments as presented in Figure 6.



Figure 6: View of the Sakarya-2 Viaduct

#### 3.1 Superstructure

The superstructure is slab on steel girder with six continuous steel girders. Steel girders are selected in order to provide a lighter superstructure compared to reinforced concrete superstructure. A view of the superstructure cross-section is presented in Figure 7.

The depths of the girders are variable with minimum depth of 1.75 meters and maximum depth of 2.75 meters. Center-to-center spacing between steel girders is 2.30 meters. Expan-

sion joints are only present at abutments. By selecting a continuous superstructure, risk of unseating due to rotation of the substructure and unexpected displacements is eliminated to an important extent.

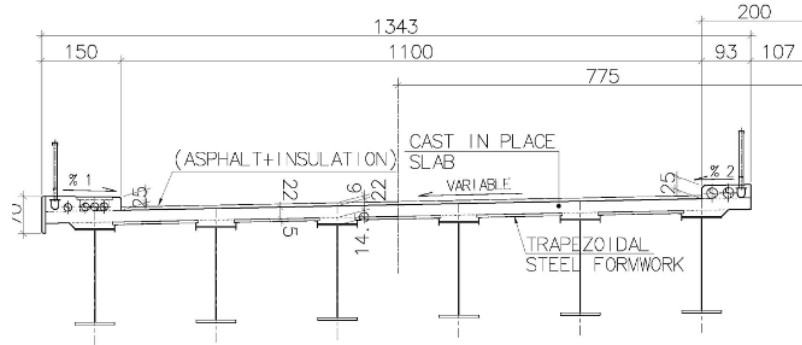


Figure 7: Cross-Section of Superstructure

### 3.2 Substructure

Piers are hammerhead type with an oblong shaped column. Past experience has indicated that seismic performance of bridges with piers having multiple columns is better compared to those having only one column. In case of piers with single column, stability problems in the transverse direction and large vertical accelerations at the cap beam level due to transverse displacement may be the reasons of such an advantage. Unfortunately, a pier with double column could not be selected for this bridge due to space problems since the bridge crosses the present railway line, highway line and the highspeed railway line that is recently in construction. The bridge has  $20^\circ$  skew angle. The dimensions of the oblong shaped column are 6.00 meters $\times$ 2.00 meters.

Cast in place reinforced concrete piles with a diameter of 120 cm were utilized in the bridge foundations. There are 20 piles at a pier and 25 at an abutment. The piles are embedded to the bedrock. One pier at each bridge has a spread footing since a very hard rock (conglomerate) was encountered at the surface during construction.

### 3.3 Seismic Isolation Bearings

A hybrid seismic isolation system consisting of natural rubber bearings and lead rubber bearings were selected for this viaduct. At each substructure, four natural rubber bearings and two lead rubber bearings support six steel I girders.

The four natural rubber bearings over each substructure was intended to provide additional stiffness to achieve a small post elastic period to deflect the earthquake input energy transferred through the underlying soft soil and to limit the isolator displacements under near-field effects.

Lead core in lead rubber bearings increases damping of the system by its elasto-plastic behavior under lateral loads. Structural deformations and forces diminish with energy dissipation capacity. Overall characteristic strength of lead rubber bearings is selected as 10% of the superstructure weight in order to limit the isolator displacements. On the other hand, higher characteristic strength may result in high heat generation at lead core. Therefore, heating up of

the lead core may result in reduced energy dissipation capacities under repeating loading cycles.

### 3.4 Structural Model

A 3D model of the viaduct is built and analyzed using the software Larsa 4D [6]. The structural model is capable of simulating the non-linear behavior of piers, seismic isolation bearings and soil-structure interaction effects.

Lead rubber bearings are modeled using hysteretic translational springs. An equivalent pile diameter was found by performing a non-linear static analysis on the foundation system. Equivalent single pile has nearly identical lateral response with that of the foundation system. Utilizing a single pile in analyses drastically reduced computation time. Soil-structure interaction is modeled using linear viscous dampers representing effects of soil damping and radiation damping. P-y curves are utilized in analyses. Dimensions of the soil column should be selected sufficiently large in order to be able to change vibration characteristic of pile group. Shear moduli of the soil layers are taken from dynamic analyses results of the soil site.

### 3.5 Analysis Results

Non-linear analyses with near-field earthquake records resulted in very high substructure forces and isolator displacements. Effective isolation period is found out to be around 1.5 sec. This relatively short isolation period was intentionally selected in order to reduce the sensitivity of the bridge to long-period velocity pulses and high dominant site period due to soft soil and liquefaction. Lateral design displacement for the lead rubber bearings and elastomeric bearings are  $\pm 850$  mm which resulted in very large isolation bearings. Maximum isolator displacements for each record are presented in Table 2 (in cm).

Earthquake	Long Disp	Trans Disp
Karakyr	50	100
Jensen	40	58
La Sepul- veda	117	89
Takatori	24	60
Bolu	41	80
Sylmar	32	80

Table-2: Isolator displacements (in cm)



### 3.6 Tests of Seismic Isolation Bearings

Natural rubber bearings and lead rubber bearings at 9 of 10 axes have a diameter of 1100 mm. Overall heights of the bearings are 580 mm. Diameter of the bearings utilized in the pier with spread footing is 1400 mm with an overall height of 912 mm. Shear moduli of the rubber bearings is in between 0.42 MPa-0.915 MPa. Bearings with 1400 mm diameter were manufactured in two parts and then connected by using pins and welding.

Test velocity of the lead rubber bearings was calculated as 2.2 m/sec. Combined with lateral design displacement of  $\pm 850$  mm, testing became the critical issue since there are few laboratories in the world capable of performing such tests. Bearing manufacturer Alga Spa selected EUCenter Laboratory for prototype tests. However, lateral stroke capacity of the testing equipment in EUCenter is limited to  $\pm 580$  mm. Therefore, prototype tests were divided into two parts as static prototype tests and dynamic prototype tests.

Static prototype tests were performed at Alga spa laboratory with a velocity of 1.6 mm/s and maximum lateral displacement of approximately 1060 mm. Lateral displacement capacity and stability of the bearings were verified during static prototype tests. Nevertheless, no heating in the lead core was observed due to very low testing velocity.

Dynamic prototype tests started with a velocity of 2.2 m/s. However, lateral accelerations in the movable base plate increased to 4g at tests having low displacement amplitudes [7]. Lateral inertial force at the base plate accounted for approximately 50% of the total force. Moreover, stability of the equipment was endangered due to very low axial load levels. For these reasons, tests with 1.4 m/sec, 1.6 m/sec and 1.8 m/sec were performed as confirmation tests. Unfortunately, heating-up of the hydraulic oil did not allow successive cycles without interruption. On the other hand, tested bearings were not damaged during tests as presented in Figure 8.

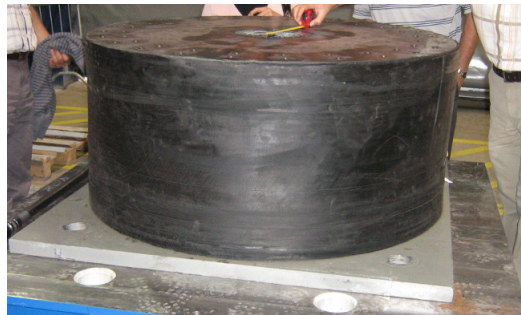


Figure 8: Lead Rubber Bearing After Dynamic Prototype Test

## 4 VK14A VIADUCT

VK14A viaduct is a 1020 meter long structure with 33 meter spans. There is a gentle horizontal curvature on the viaduct. The viaduct is a part of Ankara-Istanbul highspeed railway line. A general view of the viaduct is presented in Figure 9.





Figure 9:View of VK14A Viaduct

#### 4.1 Superstructure

The superstructure consists of 10 precast prestressed I girders with a 30 cm cast-in-place R/C slab. The view of the superstructure is presented in Figure 10. Superstructure is made continuous by cast in place diaphragm beams connected with I girders by means of U bars emerging from I girders, as presented in Figure 11.

The rails are continuous UIC 60 type rails. In the superstructure, expansion joints are present at every 3 or 4 spans in order to limit additional rail stresses due to train acceleration, deceleration and temperature changes.

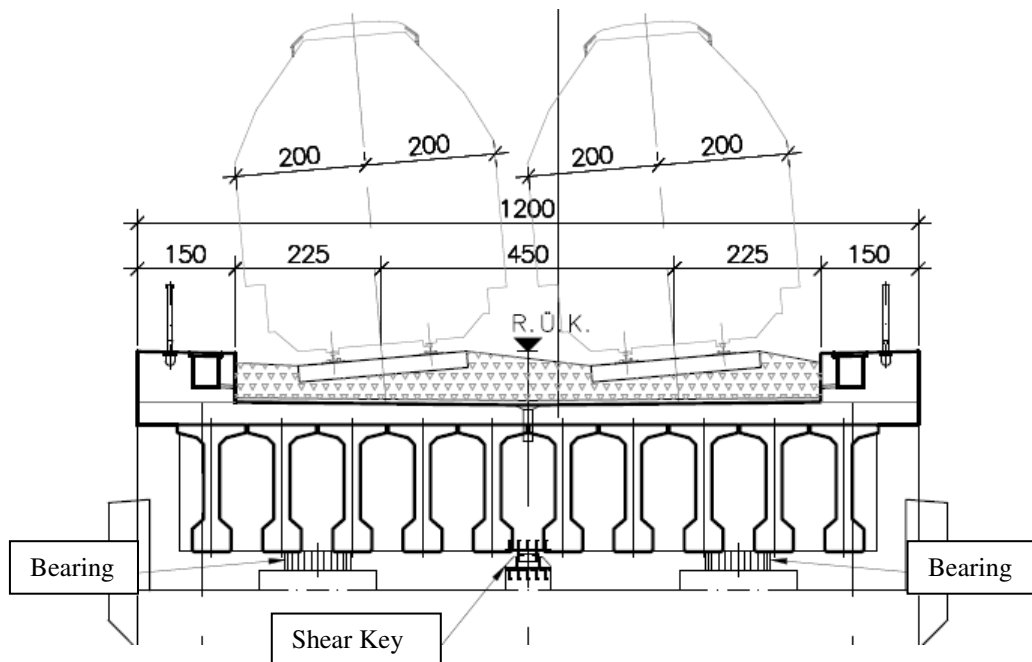


Figure 10:Cross-Section of VK14A Viaduct



Figure 11: Reinforcement Cage of I Girders

#### 4.2 Substructure and Seismic Isolation System

The selected bearing system consists of curved surface sliding bearings with 12% dynamic friction coefficient. There are two bearings at each pier located on diaphragm beams. View of a curved surface sliding bearing during installation is presented in Figure 12. Relatively high friction coefficient is selected in order to fulfill stringent service load requirements of railway bridges. Radius of curvature of the bearings is selected as 2000 mm in order to limit post-elastic period of the structure, which is critical for near-field structures. The bearing supplier is Alga Spa.



Figure 12: Curved Surface Sliding Bearing Installation

In the longitudinal direction, viscous lock-up devices were utilized in order to connect modules to provide continuity. The devices are force-limiting type, meaning that after a force threshold the control valves open and the device acts as a viscous damper. Total stroke of the device is  $\pm 200$  mm.

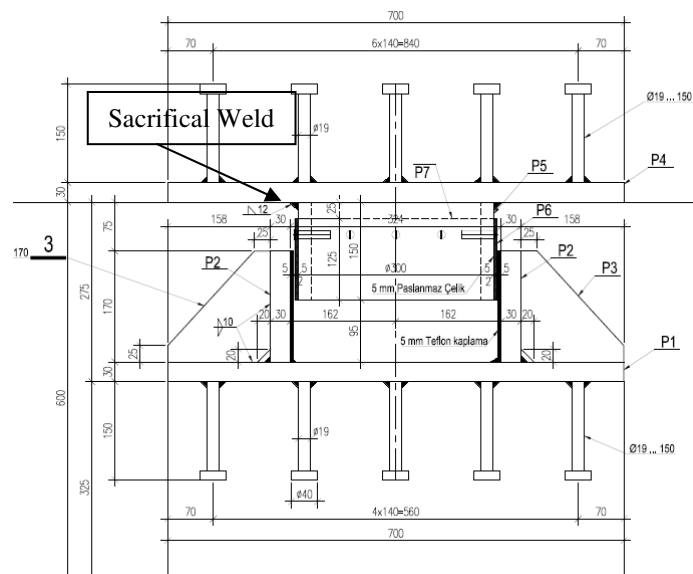


Figure 13: Shear Key Detail

### 4.3 Structural Model and Analysis Results

The structural model of this viaduct is similar to that of Sakarya-2 viaduct. Design isolator displacement is selected as 500 mm. According to Eurocode 15129 [8], restraining rings should not be utilized at sliding bearings so as to prevent seismic pounding, which may result in bearing damage and transmission of very high pounding forces to substructure. Sectional view of the utilized bearing is presented in Figure 14.

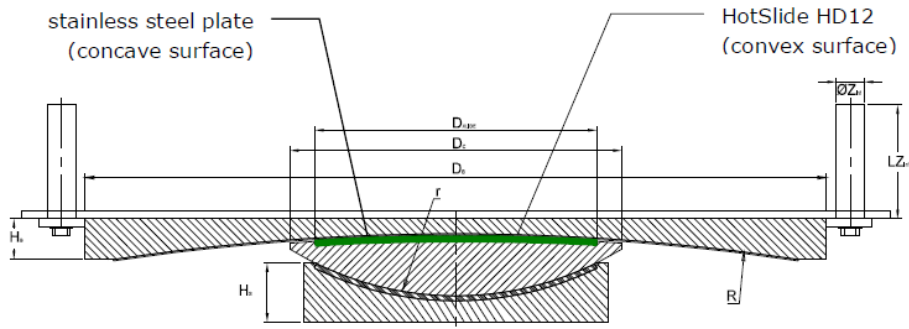


Figure 14: Sectional View of the Selected Curved Surface Sliding Bearings

#### 4.4 Tests of Seismic Isolation Bearings

Tests of seismic isolation systems for railway viaducts is more severe compared to other structures due to stringent service load requirements for these structures in addition to seismic load requirements. Wearing is an important issue for sliding bearings and as expected the effect of wearing increases as the friction coefficient increases.

Wearing resistance of sliding material utilized for this viaduct should be very high since selected friction coefficient is 12% and the superstructure is located directly on bearings. Wearing tests are performed for a total travel path of 1.5 miles. Results of wearing tests are presented in Figure 15.

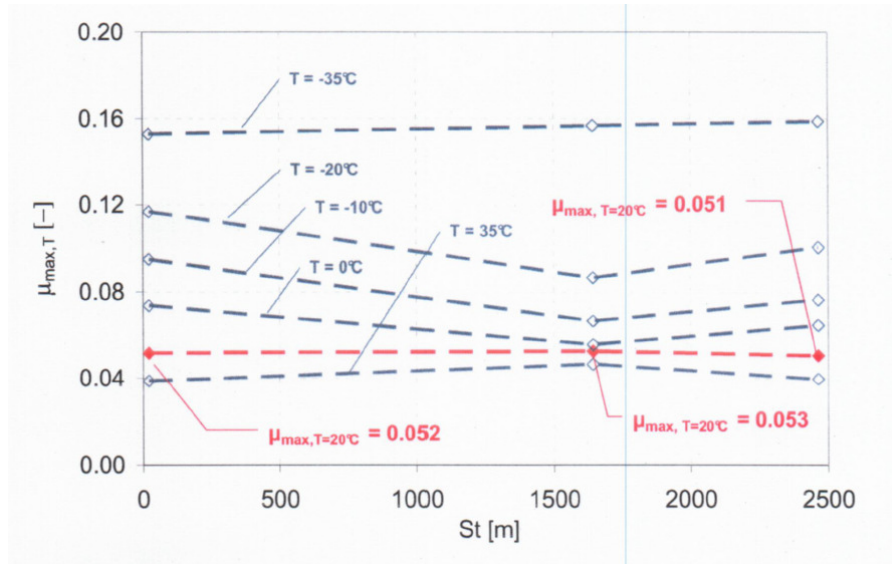


Figure 15: Results of Wearing Tests [9]

As seen in Figure 15, wearing resistance of the sliding material is quite high which has a favorable effect on rail-structure interaction. Temperature sensitiveness of friction coefficient implies that friction coefficient shall not be considered as a definite value but its variation with temperature and wearing shall be considered during structural design.

Behavior of the bearings during dynamic prototype tests was quite satisfactory and degradation in energy dissipation capacity was quite low under repeating load cycles, as presented in Figure 16.

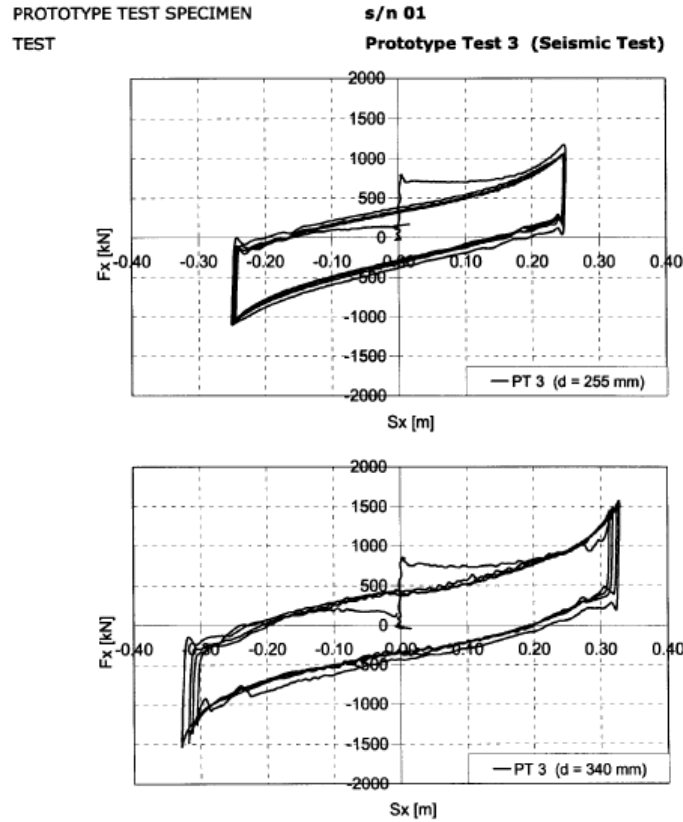


Figure 16: Results of Dynamic Prototype Tests [10]

## 5 CONCLUSIONS

- The two seismically isolated viaducts were designed to withstand an MCE  $M_w=7.5$  earthquake.
- Seismic isolation period in near-field structures can be kept relatively small in order to provide high restoring force and shift isolation period from that of high intensity velocity pulses.
- Soil structure interaction shall be considered for structures on soft soils.
- Testing of seismic isolation systems for near-field structures require special attention.
- For railway bridges, requirements for bearing systems during service load conditions are as important as seismic load conditions.

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