13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 490

[1]THE DESIGN OF STEEL PLATE GIRDER BRIDGES USING SLIDING ISOLATION BEARINGS

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SUMMARY

In early 1999 the South Carolina Department of Transportation (SCDOT) upgraded their seismic design requirements by adopting revisions to the American Association of State Highway and Transportation Officials (AASHTO) Standard Specification for Highway Bridges, Seismic Design Provisions of Division I-A. Not long after this the SCDOT secured the services of Earth Tech, Inc. Consulting Engineers to design two new replacement overpasses over I-26 in Charleston, South Carolina. These bridges replace the existing structures carrying US Route 52 and Ashley Phosphate Road over I-26. Earth Tech decided on two-span continuous steel plate girder bridges for these overpasses and provided sliding isolation bearings to reduce the forces on the substructure during a seismic event.

INTRODUCTION

The sliding isolation bearings utilized on this project were first conceptualized in the late 1980's with research conducted at The National Center for Earthquake Engineering Research (now known as MCEER) located at the State University of New York at Buffalo, Kartoum [1]. Researchers utilized a sliding disk bearing and displacement recovery springs to achieve an efficient isolation system.

Disk bearings were developed back in 1970 by Fyfe [2] and have a successful track record on hundreds of bridges all over the world since that time. They are comprised of a polyether urethane load and rotational element which due to its inherent strength, does not need to be confined (Fig. 1). The urethane element is sandwiched between two steel load plates. The plates are interconnected through a pin and ring design which also contains the load element through a hole in the center. For displacement an upper slide plate faced with stainless steel is added and bears on a PTFE disk which is bonded and recessed into the upper bearing plate.

The sliding isolation design by Bradford [3], utilizes a disk bearing and polyether urethane displacement control spring design to restore the structure to its original pre-quake position (Fig. 2). One of the advantages of the orthogonal springs is that they can be custom designed to provide variable stiffness in

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different directions. One common scheme is to design stiffer springs in the transverse direction to keep the bridge from displacing too far laterally.

Fig. 1 – Sliding disk bearing.

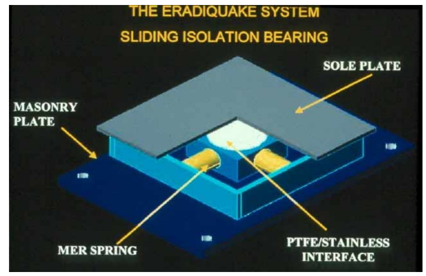

Fig. 2 - Sliding isolation bearing.

These sliding isolation bearings have been shake table tested extensively on the Seismic Simulator located at MCEER (Fig. 3). Tests run on a quarter scale steel plate girder bridge resulted in force reductions on the order of 3-5 times and a maximum permanent offset of 1.5mm.

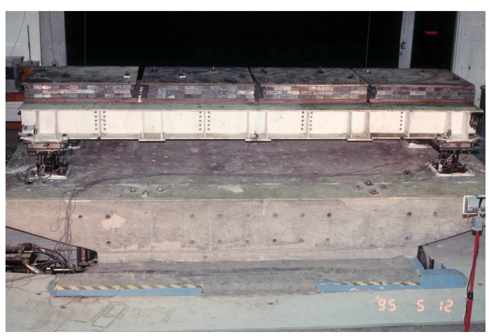

Fig. 3 – Shake table testing of sliding isolation bearing.

In addition to the shake table testing, numerous full scale prototype tests have been run on the sliding isolation bearings to demonstrate the efficacy of this system (Fig. 4). The result of this research indicates this bearing with polyether urethane displacement-control spring design is a low cost, high damping and practical isolation system. To date these sliding isolation bearings have been installed on over 50 bridges worldwide.

BRIDGE DESCRIPTIONS

Both new replacement overpasses on US Route 52 and Ashley Phosphate Road cross the major Interstate Highways I-26 in Charleston. SCDOT has classified these bridges are "essential bridges" because the closure of the highway or the bridges may create a major economic impact to the region. The I-26 and the new bridges are also designated as a critical path by the local emergency plan.

The US-52 Bridge is a two-span continuous composite welded steel plate girder bridge with a skew angle of approximately 45°. The bridge is oriented in south-north direction and has a 45m-42 m span arrangement. The 15.5 m wide superstructure consists of 6 steel plate girders with an 215 mm thick reinforced concrete deck slab. The superstructure rests on three reinforced concrete bents. The abutment seat consists of a 1.37m high by 1.5 m wide cap supported on five 1.07 m diameter columns with 1.2 m diameter drilled caissons. The back wall is approximately 1.67 m high, cast on the top of abutment seat. The abutments do not have wing walls. The center pier bent consists of a 1.5 m wide by 1.5 m deep cap beam that is supported on six 1.07 m diameter columns with 1.2 m diameter drilled caissons.

On the other hand, the Ashley Phosphate Bridge is a two-span structure with a total span length of 66 m and a total deck width of 43 m. The bridge is oriented in west-east over the Interstate I-26 with 28 m - 38 m span arrangement. The bridge superstructure consists of 17 lines of continuous steel plate girders approximately 1.4 m deep with a 215 mm thick cast-in-situ composite concrete slab. The two abutment bents consist of 1.2 m high x 1.2 m wide concrete cap beams supported on ten 0.9 m diameter concrete columns with 1.1 m diameter drilled concrete caissons. Mechanically Stabilized Earth walls approximately 5.5 m high were provided in front of the abutments to retain embankment fills. The center pier bent consists of a 1.67 m deep by 1.2 m wide concrete cap beam supported on eight 0.9 m diameter concrete columns on 1.1 m diameter drilled concrete shafts. All substructures are constructed parallel to each other with a skew angle of approximately 10° to a perpendicular to the longitudinal axis of the superstructure.

SEISMIC ANALYSIS AND RESULTS

According to SCDOT Specifications, these essential bridges are assigned to seismic performance category (SPC) D and a two level design method was implemented. The first earthquake event designed for was a Functional Evaluation Earthquake (FEE) with a 10% probability of exceedance of 50 years. The second was a Safety Evaluation Earthquake (SEE) with a 2% exceedance in 50 years. The expected peak ground accelerations for these two events are 0.42g and 1.05g respectively.

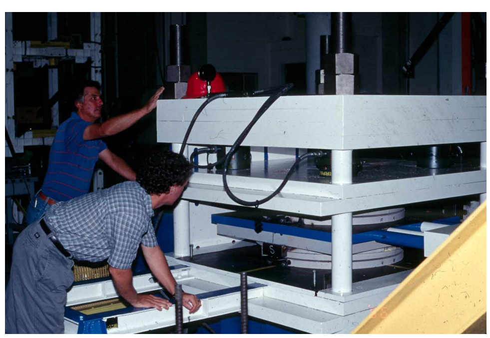

Fig. 4 – Prototype testing of sliding isolation bearings.

A static pushover analysis is used to determine the deformation capacity of the bridges during each earthquake event (FEE and SEE). The analysis uses a step-by-step force deformation response analysis that takes account of inelastic response of the structural details specified in the design, such as inelastic behavior and restraint of the bearings and the hinging in the concrete columns. The displacement is achieved by incrementally increasing lateral forces to the structure. After each stage of force application, the force in each structural element is assessed. If the force demand in the element exceeds its capacity, the structural model is modified to represent the yielding of that element. This procedure is repeated until the maximum displacement allowed by the confinement detailing is reached. This ultimate possible displacement is compared to the actual displacement demand in the form of a ductility ratio demand.

In the pushover analysis, the lateral forces are applied in the "weakest" direction of the bridge in order to obtain the collapse mechanism. In the longitudinal direction, when the 50 mm gap between the superstructure and the abutment is closed, the longitudinal seismic forces are transferred through the abutment and into the backfill. A collapse mechanism cannot form in the longitudinal direction until the backfill behind the abutment fails. Based on a maximum soil bearing pressure for the backfill behind the abutment ballast wall of 370 kN/m^2, as recommended by CALTRANS, the force required to fail the backfill was compared to the earthquake force of the superstructure. This comparison indicated that the soil capacity is greater than the force imposed by the structure, and the displacement required to mobilize the soil pressure does not impose significant forces in the abutment piles. Based on this analysis, a collapse mechanism will not develop in the longitudinal direction under the 2% in 50 years event, as the superstructure would oscillate between the two abutments in the longitudinal direction.

The "weakest" direction is therefore in a direction parallel to the skew of the bridge. When a lateral force is applied in this direction, the abutment backfill is not engaged and the entire seismic force has to be taken by the three bents.

Sliding isolation bearings were utilized to ensure that the structures would perform elastically and remain undamaged at the FEE event. The pushover analyses indicate that the total transverse movement of the deck over the center pier location is approximately 55 mm for the US 52 Overpass, of which 35 mm is accommodated by the bearings and 52mm for the Ashley Phosphate Bridge, of which 29 mm is accommodated by the bearings. The net transverse displacements on the pier bents are therefore approximately 20mm and 23 mm on the US 52 and Ashley Phosphate Overpasses respectively, resulting in a force demand on each pier column that is still within its elastic range.

On the other hand, Earth Tech engineers discovered that significantly large displacements would occur during the SEE event. This presented some damage issues at the end of bents, which were undesirable. As a result a unique feature was added to the isolation bearings. A force transfer mechanism (FTM) was added to the isolation bearings, which limits the displacement to 100mm. Once this displacement is reached, the FTM is engaged and the superstructure is no longer isolated from the substructures, resulting in the lateral forces being redistributed throughout the substructures. The pushover analyses indicate that the net transverse movement on center pier is approximately 121mm and 112 mm at the US 52 and Ashley Phosphate Overpasses respectively. As a result, all pier columns would form plastic hinges at the SEE event with a maximum plastic hinge rotation of about 0.009 to 0.012 radians. The reinforcing and confinement in all columns were subsequently reviewed and checked to ensure that they are within in the allowable limits as specified by SCDOT.

There are a total of 69 sliding isolation bearings on these two structures ranging from 760 kN to 2450 kN in vertical capacity.

The bearings were fabricated in 2003. Prior to shipment each bearing had to be tested in accordance with the SCDOT Specifications and the 1999 AASHTO Guide Specifications for seismic isolation design [4] and according to the recommendations of Watson [5].

The isolators were installed in September 2003 (Figs. 5 - 7). The installation of an isolation bearing is no different from any other conventional device. The bearing needs to be set to proper grade and aligned accurately with the superstructure. Connections can be bolted or welded depending on the contract requirements. In the case of the I-26 overpasses project, a welded connection was detailed between the sole plate of the isolation bearings and the bottom flange of the plate girders. Care needs to be taken to ensure that the heat generated from the welding process does not damage the PTFE and polyurethane materials contained within the isolators.

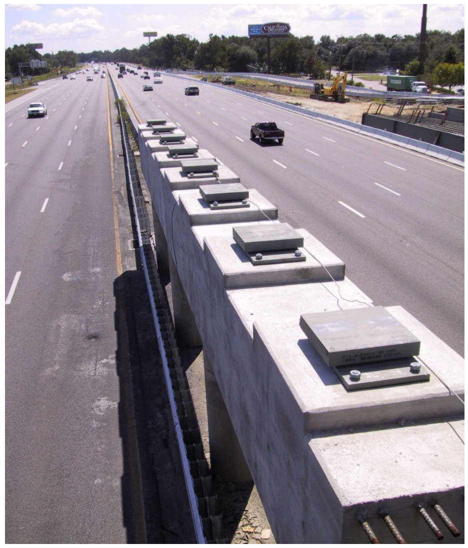

Fig. 5 – Sliding isolation bearings on pier of I-26 overpass.

Fig. 6 – Steel plate girders installed on sliding isolation bearings (I-26 overpass).

Fig. 7 – Sliding isolation bearing supporting steel plate girder (I-26) overpass.

CONCLUSIONS

Isolation bearings are now being used on a regular basis by engineers trying to reduce the impact of seismic forces on bridges. While many of the early isolators were the elastomeric type, bridge engineers are finding that sliding systems offer more versatility at a lower cost and have much higher damping capabilities. Steel plate girder bridges are excellent candidates for using sliding isolation bearings.

REFERENCES

1. Kartoum, A., Constantinou, M.C., Reinhorn, A.M., 1992, “Sliding Isolation System for Bridges.” Analytical Study, Earthquake Spectra; Vol. 8, No. 3.
2. Fyfe, E.R., “Development of a Shear-Restricted Disc Bearing.” American Concrete Institute, SP-70, 1981,
3. Bradford, P.F., Watson, R.J., “Retrofit of a Medium Span Structure Using a Sliding Isolation System.” 4^{th} International Conference on Short and Medium Span Bridge Conference Proceedings, Canadian Society for Civil Engineering, Halifax, Nova Scotia, 1994: pp. 839-847.
4. “Guide Specifications for Seismic Isolation Design, American Association of State Highway and Transportation Officials.” 1999.
5. Watson, R.J., “Isolation Bearing Specification Issues”, World Congress on Joint Sealing and Bearing Systems.” ACI SP 164-9, pp 165-176, 1996.