Seismic Retrofit of the Poplar Street Bridge

Mark R. Capron[1]

Abstract

This paper summarizes the seismic evaluation, retrofit strategy, design, and construction of the seismic retrofit for the Poplar Street Bridge over the Mississippi River at St. Louis. The 660 meter (2,165 foot) structure consists of two parallel five span continuous roadways with orthotropic steel deck and steel box girders. The seismic evaluation considered three levels of design earthquakes and identified deficiencies in the bearings, reinforcement splices in the columns and piers, and one foundation. The retrofit included adding longitudinal shock transmission units, transverse shear blocks, column splice confinement, shear walls, and rock anchors.

Introduction

Jacobs Civil Inc. (formerly Sverdrup Civil) has supported the Missouri Department of Transportation (MoDOT) in evaluating and designing retrofits for several major bridges and bridge complexes throughout the state. Evaluation of these structures using current guidelines (FHWA 1983, 1995) has identified significant seismic deficiencies in critical bearing, column, and foundation elements, in spite of design acceleration coefficients as low as 0.12g. This paper presents the application of the seismic retrofitting guidelines to the Poplar Street Bridge over the Mississippi River at St. Louis. Design of seismic retrofits for this structure was developed in two phases: Phase 1, consisted of seismic evaluation and development of a retrofit strategy, and Phase 2, consisted of additional studies to refine the strategy and final design of selected alternatives. Construction of the seismic retrofit was completed in late 2002 at a cost of 6.2 million dollars. The following paragraphs present a description of the bridge, the approaches and results of the seismic evaluation, the retrofit strategy, construction issues, and conclusions from this project.

Description of the Bridge

The Poplar Street Bridge carries more than 130,000 vehicles per day across the Mississippi River at St. Louis, Missouri via U.S. 40/I-64 and I-55/70, see Figure 1. The 660 meter (2,165 foot) structure consists of two parallel five span continuous roadways, each providing four lanes of traffic. Span lengths from west to east are 91, 152, 183, 152, and 81 meters (300, 500, 600, 500, and 265 feet). Two variable depth steel box girders support each roadway. The deck is of orthotropic steel plate construction consisting of a deck plate, trapezoidal longitudinal ribs and transverse floor beams. The deck plate acts as the top flange of the transverse floor beams, the trapezoidal ribs, and the main longitudinal box girders. The reinforced concrete substructure from west to east consists of a hollow shear wall type structure at Pier 1, solid shafts with rectangular columns and continuous cap beams at Piers 2, 3, 4, and 5, and a hollow shear wall structure at Pier 6. The

[1] P.E., Jacobs Civil Fellow, Project Manager, Jacobs Civil, Inc.

substructure is founded on 1.8 meter (6 foot) diameter caissons to rock at Piers 1, 3, 4, 5, and 6, and a spread footing on rock at Pier 2. The superstructure is supported on the substructure by expansion bearings that allow rotation using spherical bearings and longitudinal translation through guided rollers at Piers 1, 2, 4, 5, and 6, and fixed bearings that allow only rotation with spherical bearings at Pier 3. Sverdrup and Parcel and Associates originally designed the bridge in 1963. The pier reinforcing steel is spliced near the base of the columns with lap splices of 24 bar diameters in length and confined with #5 ties spaced at 30 cm (12 inches), as typical of bridges designed in this timeframe. There are also similar splices in the reinforcing steel where the base of the pier shafts connects to the footings.

Figure 1. Poplar Street Bridge in St. Louis, Missouri

<u>Seismic Evaluation</u>

The analytical model used in the seismic evaluation consisted of over 15,000 degrees of freedom and included representation of the deck, box girders, and Piers 1 and 6 with shell elements, while the bearings, cross frames, and Piers 2, 3, 4, and 5 were modeled with beam elements. Linear spring elements were included in the model to represent the stiffness of the soil/foundation system, and mass was applied to account for unmodeled components such as parapets, major sign structures, and one lane of traffic live load per roadway.

Dynamic analysis was performed using the linear elastic response spectrum method and spectra based on the parameters and return periods shown in Table 1. In addition to the three design spectra, the analysis considered cases with completely rigid soil springs, with linear soil springs, with existing bearings, and with expansion bearings supplemented by “rigid” longitudinal restrainers.

Table 1. Design Response Spectra Parameters

Acceleration Coef. (A)	Soil Type	Return Period (yr.)	Reference
0.12	2	475	(AASHTO 1996)
0.23	S_2	2,500	(FEMA 1991)
0.50	2	---	Maximum Credible Earthquake (MCE) - estimated from unpublished literature review

The results of the dynamic analyses were evaluated using the Capacity /Demand (C/D) ratio method presented in the guidelines (FHWA 1983, 1995). In the context of the guidelines, C/D ratios approximate the percentage of the design earthquake at which a particular component can be expected to fail; therefore, ratios less than 1.0 indicate insufficient capacity. The significance of individual C/D ratios depends on the consequences of failure of each particular component, or groups of components. Table 2 summarizes the C/D ratios computed for the existing structure at the various design levels.

Table 2. C/D Ratios for Existing Structure

Component	C/D Ratios			Remarks
	Design Level			
	AASHTO 475 yr.	FEMA 2500 yr.	MCE	
Bearing Displacement	8.9	5.2	1.7	Unrestrained bearings at Piers 1 and 6, Method 2, Method 1 0.59 all levels
Bearing Force (longitudinal)	0.3	0.3	0.1	Pier 3 only point of longitudinal seismic resistance
Bearing Force (transverse)	0.8	0.5	0.2	All Piers similar
Cap Beam Yielding	0.5 - 0.7	0.4 – 1.2	*	No collapse mechanisms at AASHTO Pier 3 near collapse mechanism at FEMA * Collapse mechanisms Piers 3, 4, and 5 at MCE
Splice Details at base of Columns	0.2 – 0.6	0.1 – 0.4	*	
Splice Details at base of Piers	0.2 – 0.9	---	---	Longitudinal collapse mechanism at AASHTO, ratios not computed for FEMA 2500 or MCE, results < AASHTO
Foundation Rotation	0.9 - 10.8	0.7 – 8.8	---	Ratios not computed for MCE
Cross Frames	1.2 – 3.7	0.6 – 2.0	0.3 – 0.9	

The C/D ratios for the existing structure identified significant deficiencies in the bearings, the lap splices at the base of the columns, the lap splices at the base of the Piers at the AASHTO design level. Evaluation of the structure with engaged restrainers at expansion Piers 1, 2, 4, and 6 showed notable improvement in the critical C/D ratios of Pier 3 at the AASHTO design level, but indicated the need to increase the overturning capacity of Pier 1. Comparison of the AASHTO 475 year and FEMA 2500 year design levels showed similar types of retrofits would be required for both levels, while the MCE design level would require major changes in the structural response, which could conceptually be provided by isolation bearings, and/or major increases in capacity.

Seismic Retrofit Strategy

Based on the findings and recommendations of the evaluation, MoDOT selected the AASHTO 475 year spectrum as the design level for the seismic retrofit, and a retrofit strategy that involved adding shock transmission unit type longitudinal restrainers to expansion Piers 1, 2, 4, 5, and 6; increased transverse force capacity of the bearings at Piers 1, 4, 5, and 6 using reinforced concrete shear blocks and steel bumpers; strengthening the longitudinal and transverse capacity of the Pier 3 bearings; adding rock anchors to the Pier 1 foundations; confinement of the lap splices in the reinforcement at the base of the columns; and reinforcement of the lap splices at the base of Piers 2, 3, 4, and 5. The construction cost of these retrofits was estimated to be on the order of 7.2 million dollars.

In the context of this retrofit, shock transmission units refer to devices that are designed to allow slow thermal expansion movements of the existing bearings and to lock-up and resist relatively fast movements, such as caused by earthquake loading. These devices are also referred to as force transmitters or lock-up devices. Also, the concept of adding shock transmission units to the existing expansion piers was adopted over replacing the bearings with seismic isolation bearings to minimize disruption of traffic on the bridge, to maintain the functionality of the existing bearing system, and to avoid the construction issues and costs associated with changing bearings on this major structure.

During Phase 2, the retrofit strategy was further refined by evaluating damper versus shock transmission unit longitudinal restrainers at all piers; investigation of using longitudinal restrainers at Piers 2, 4, and 5 with bearing seat extensions at Piers 1 and 6; investigation of the longitudinal shear capacity of Piers 1 and 6; investigation of alternatives for confining the lap splices in the column reinforcement; and investigation of alternatives for reinforcing the lap splices at the base of the piers.

The study of retrofitting with dampers, which are designed to dissipate significant amounts of energy as the structure moves longitudinally under earthquake loading, included time history dynamic analysis of the structure. These analyses used were based on AASHTO spectra compatible synthetic ground motions and damper characteristics developed from recommendations from damper manufacturers. Based on the results of this study, shock transmission units were selected for the retrofit, primarily because obtaining force reductions with the dampers was dependent upon the ability of the existing bearings to displace through multiple cycles under seismic loading. Because the actual displacement characteristics of the existing bearings could not be readily quantified, the shock transmission units, which did not require significant bearing displacement, were believed to provide a more reliable alternative to dampers for this project.

The analysis of retrofitting with longitudinal shock transmission units at all piers versus only the interior piers, showed that the former alternative provided greater longitudinal displacement control, and greater capacity for resisting seismic loading, as expected. The final longitudinal restrainer configuration provided four 1.56 mega-Newton (350 kip) shock transmission units at each of the expansion bearings at Piers 2, 4, and 5, and two 2.59 mega-Newton (500 kip) shock transmission units at each of the expansion bearings at Piers 1 and 6.

Evaluation of the longitudinal shear capacity of Piers 1 and 6 showed that these piers did not have sufficient longitudinal capacity to resist the design level restrainer forces. Consequently, the retrofit design included addition of reinforced concrete shear walls inside of Piers 1 and 6 to provide the required capacity. Also, because the depth to rock at Pier 1 was less than 8 meters (26 feet), the retrofit included adding rock anchors through the grade beams at the base of the pier, to increase overturning resistance.

Confinement of the lap splices near the base of the 4 meter (13 foot) by 3.7 meter (12 foot) rectangular columns with conventional grout filled elliptical steel jackets was considered undesirable for this project because the jackets would be exposed to stream flow during flood stages, and ice flow during the winter. Investigation of composite wrap alternatives showed that this approach would not provide the required confinement near the centers of the faces of these large columns. Because of the limitations of these alternatives, confinement of the lap splice regions by steel plates with closely spaced post-tensioned steel rods was selected for the retrofit design. This approach provided minimal horizontal exposure to stream and ice forces, and tension in the rods, which were extended into the core of the column, provided the desired confining pressure.

While the lap splices near the connection of the base of the pier shafts and the footings are poorly confined, like the columns, the location below the mud line and about 15 meters (50 feet) below the surface of the river, made retrofit of these details difficult. Retrofit alternatives that were investigated included drilling vertically through the pier walls and adding grouted post-tensioned steel rods that were continuous through the splice region and anchored into the footings, and external confinement that would be installed inside dewatered coffer dams, or installing precast concrete segments without dewatering. Based on evaluation of the cost of these alternatives, and the associated construction risks, along with evaluation of the ductility demands and potential failure modes associated with the existing details, a strategy of no retrofit was selected at the base of the piers.

Construction Issues

The following points summarize the major construction issues that were addressed on the project.

- Because the bridge is a major transportation link for the region, minimizing disruption of traffic on the bridge during construction was a major project objective. To address this issue, the Contractor elected to perform all construction activities from below the bridge, using barges for work in the river, and working from the ground, when possible during low water. This approach required coordination with barge operators on the river, but was accomplished with minimal disruption of construction work, and no disruption of the traffic on the bridge.

- Construction of the steel plate confinement retrofit for the column splices required installation of over 6,600 tensioned rods. To minimize conflicts between the rods and the existing column reinforcement, ground-penetrating radar was used to locate the reinforcing steel, and the rod layout was planned for each location, prior to fabrication of the plates or drilling the holes for the rods.

- Installation of the brackets for the shock transmission units required core drilling through the pier cap beams, which were approximately 3 meters (10 feet) thick, and installing high strength steel rods. In addition to control of the accuracy of the drilling operations, this modification required selection of a grouting material and installation method that would that would provide sufficient strength and durability, and allow sufficient construction time with sufficient flow characteristics under the temperature conditions that were expected at the time of construction.

- Installation of the shear walls inside Piers 1 and 6 required consideration of how the formwork, reinforcement, and concrete would be placed, and the formwork removed within the confined space of the piers. Construction access to these areas was gained by cutting 2.4 meter (8 foot) by 2.7 meter (9 foot) openings in the pier walls. These openings also were used to provide access for the necessary construction equipment for installing the rock anchors through the grade beams inside of Pier 1. These temporary openings were closed with reinforced concrete, after work inside the piers was completed.

- Construction of the various retrofit measures required field verification of dimensions to sufficient tolerance for fabrication of retrofit components. In some cases, templates were developed for existing bolt patterns to facilitate fit-up of fabricated steel components.

- Fabrication, testing, and delivery of the shock transmission units required a majority of the 17-month duration of the project, and were major scheduling considerations.

Conclusion

Seismic evaluation of this structure using current guidelines identified deficiencies in the bearings, piers, and foundations, in spite of a design acceleration of only 0.12g. A retrofit strategy was developed and implemented for the selected design level that involved adding shock transmission units to the existing bearings, and strengthening critical substructure components. Figures 2 and 3 show the completed retrofits of Pier 2 and the shock transmission units and transverse shear blocks at Pier 1. Construction of the retrofit was completed at a cost of 6.2 million dollars in late 2002.

Acknowledgment

The Author wishes to express appreciation to the Missouri Department of Transportation and the Federal Highway Administration for their support in the development of this project. The Author also wishes to acknowledge the Massman Construction Co. for constructing the project, and Taylor Devices, Inc. for supplying the shock transmission units for the project.

Figure 2. Pier 2 with retrofit installed

Figure 3. Pier 1 retrofits

References

American Association of State Highway Transportation Officials (AASHTO). 1996. “Standard Specifications for Highway Bridges.” Sixteenth Edition. Washington, D.C.

Federal Emergency Management Agency (FEMA). (1991). “NEHRP Recommended Provisions for Seismic Regulations for New Buildings.” No. 222. Washington, D.C.

Federal Highway Administration (FHWA). (1983). “Seismic Retrofitting Guidelines for Highway Bridges.” Report No. FHWA/RD-83/007, McLean, VA.

FHWA. (1995). “Seismic Retrofitting Manual for Highway Bridges.” Publication No. FHWA-RD-052, McLean, VA.