INELASTIC SEISMIC RESPONSE OF A 59-SPAN BRIDGE WITH SOIL-STRUCTURE INTERACTION

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ABSTRACT :

This paper summarizes the results of a comprehensive study carried out to assess the seismic response of a 59-span bridge, considering Soil-Structure Interaction (SSI). The bridge was built in the early seventies at a distance of about 5 km from a major fault. The methodology adopted to idealize the bridge and its foundation system for elastic and inelastic analysis is briefly described. The SSI analysis is significant in this study due to the length of the bridge, the massive and stiff foundation and the relatively soft deep soil of the site. Detailed three-dimensional dynamic response simulations of the entire bridge are undertaken using several analysis tools to verify the analytical models for the extensive inelastic analysis. 144 input ground motions are employed to assess the inelastic response of the 2160-meter bridge. The strong-motion records used are the outcome of a site-specific seismic hazard assessment. The study identifies areas of vulnerability in the investigated bridge and assesses its response at three hazard levels, corresponding to 475, 975 and 2475 years return periods. It is concluded that the seismic response of the bridge at the 500 years ground motions is unacceptable, while the demands under the effect of the 1000 years ground motions almost exceed the capacity of most bridge components. The demands significantly increase under the effect of the 2500 years earthquake scenario and considerably exceed the collapse limit states. The results clearly reflect the pressing need to retrofit different bridge components to mitigate anticipated seismic risk. The presented assessment study exploits the most recent research outcomes to realistically predict the complex seismic response of this major bridge and thus contributes to improved public safety.

KEYWORDS: Major highway bridges, soil-structure interaction, inelastic analysis, high seismicity regions, bridges deficiencies.

1. INTRODUCTION

Research carried out during the past two decades has led to significant changes in seismic design provisions of bridges. The introduction of the AASHTO Load and Resistance Factor Design (LRFD) Specifications (2007) is aimed at providing more uniform safety for different types of bridge system. The newly released FHWA retrofitting manual for highway structures (2006) also provides comprehensive procedures for assessment and retrofitting highway bridges based on recent experiences in the US, Japan, and other countries. These revisions in design specifications and retrofitting guidelines draw attention to the need for seismic assessment of complex highway bridges designed to preceding provisions to determine the level of risk associated with loss of serviceability or possible damage. This is particularly significant in the light of the continuous updates in seismic hazard maps for several regions (e.g. USGS, 2008).

Structural analysis of multi-span bridges for earthquake design often employs simplifying assumptions such as the uncoupling between superstructure and piers provided with sliders. Little studies in the literature addressed the significance of different simplifying design assumptions, particularly for multi-span complex bridges (e.g. Mwafy et al., 2007). In-depth seismic assessment studies of existing highway bridges point the way towards improving the understanding of the seismic performance of similar structures and significance of design assumptions. The Caruthersville Bridge, which carries route I-155 over the Mississippi River, is an example of a major bridge that has a high priority for vulnerability assessment. Although the 59-span 2160-meter bridge is about 5 km from a presumed major fault, it was constructed in the early seventies with minimal seismic design requirements. The superstructure consists of eleven units separated by expansion joints and supported on a variety of elastomeric and steel bearings. The main channel crossing is composed of two-span asymmetrical cantilever steel truss and ten-span steel girders, while approach spans are precast prestressed concrete girders. The substructure includes piers on deep caissons and bents on steel friction piles driven into the near surface silty sands and clayey materials. Bedrock is located more than 800 meter below the sand, gravel, and hard clay strata. Figure 1 depicts a three-dimensional view of the bridge. This brief description of this major bridge highlights the pressing need to reliably assess its earthquake response under anticipated hazard levels.

The primary objectives of this comprehensive study is thus to:

- Realistically assess the seismic response of the bridge by comparing estimates of the capacities and demands at the structure and the member levels using verified analysis tools and state-of-the-art assessment methodologies.
- Examine the effect of the refined analytical modeling of the bridge and its foundation system on the earthquake response of this complex bridge under increasing hazard levels.

The modeling assumptions adopted to idealize the entire bridge and its foundation system for elastic and inelastic analyses are briefly described. The assessment methodology implemented to assess the seismic response of the Caruthersville bridge, considering SSI, is presented. Sample results of the detailed three-dimensional dynamic response simulations of the entire bridge with SSI effects under the effect of 144 site-specific input ground motions are discussed. Areas of vulnerability of different bridge components are finally presented.

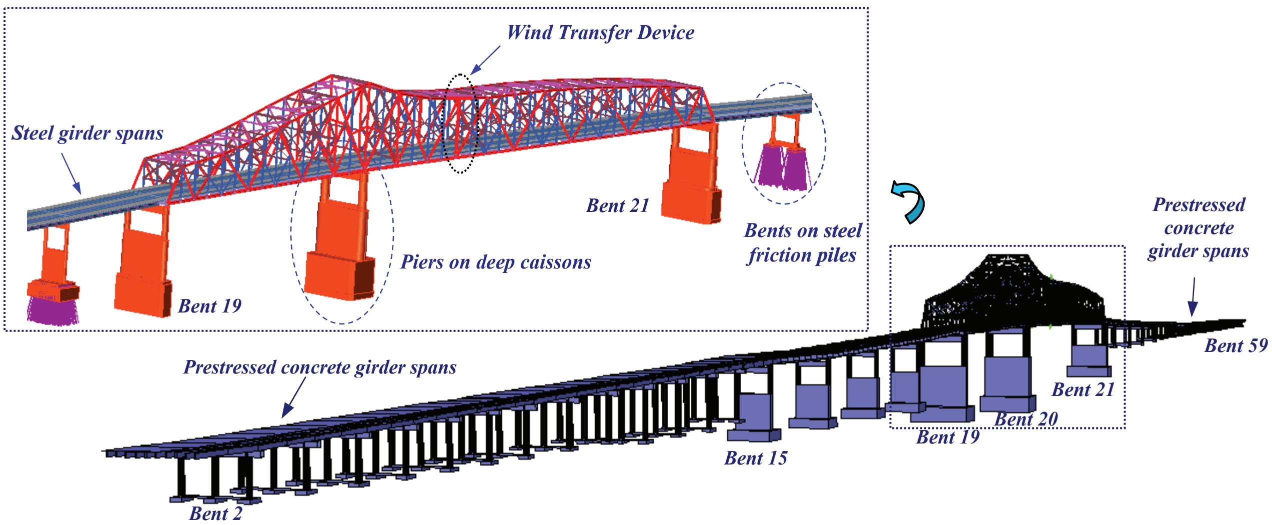


Figure 1. Three-dimensional view of the 59-span I-155 bridge.

2. MODELING METHODOLOGY FOR INELASTIC ANALYSIS

Detailed three-dimensional dynamic response simulations of the entire bridge including foundations and soil effects are undertaken using a number of analytical platforms. The finite element analysis programs SAP2000 (CSI, 2005) and ZEUS-NL (the Mid-America Earthquake Center analysis platform, Elnashai et al. 2007) are employed for elastic and inelastic analysis of the structure, respectively. The Pacific Earthquake Engineering Research (PEER) Center analysis platform OpenSees (McKenna et al., 2006) is used for an inelastic simulation of the foundation and the underlying sub-strata. The SAP2000 analytical models are mainly employed for verifications of the ZEUS-NL fiber model before executing the extensive inelastic analysis. ZEUS-NL is mainly employed to estimate the capacities and demands from inelastic pushover and response history analyses.

2.1. Super- and Sub-structure Modeling

Three different analytical models are developed for the bridge: (i) SAP2000 detailed model, (ii) SAP2000 simplified model and (iii) ZEUS-NL fiber model. The first model is developed to represent all sub- and super-structural components for elastic analysis. This modeling approach, particularly for the superstructure, is computationally demanding for inelastic response history analysis. Also, the design philosophy of bridges relies on bridge piers to dissipate energy rather than the superstructure, which remains elastic. The detailed SAP2000 analytical model is therefore modified to reduce the number of elements and nodes to a manageable limit for inelastic analysis. The superstructure is replaced by a number of cross sections with equivalent geometrical properties connected together using rigid arms. This simplification in the superstructure allowed reducing the number of elements and DOFs by about 50%. On the other hand, substructure members are refined by subdividing the columns to a number of elements to accurately monitor the inelastic response during time-history analysis. Moreover, the SAP2000 joint constraints, which are not available in ZEUS-NL, are replaced with strong arms. The rigid arms deformation and associated self-balancing loads are minimized by selecting large cross sections and high modulus of elasticity. The simplified SAP2000 model was transferred to ZEUS-NL for inelastic analysis. Due to the complex behavior of the truss, it was transfer to ZEUS-NL without any simplification. In the detailed ZEUS-NL model, each structural member is assembled using a number of cubic elasto-plastic elements capable of representing the spread of inelasticity within the member cross-section and along the member length via the fiber modeling approach.

Equivalent gravity loads and mass are distributed in the ZEUS-NL fiber model on the superstructure and along the piers height. The employed distributed mass elements utilize cubic shape function and account for both the translational and rotational inertia. The total weigh of the bridge is 351,275 kip, which includes superstructure, substructure, non-structural members, pile caps and caissons. The superstructure weight is higher than the substructure in the approach spans, which is not the case in the steel girders and the truss spans. As a result of the several deficiencies observed in structural members in the latest available inspection report of the bridge and the lack of reliable information confirming the actual material characteristics, nominal material properties are used in analysis. A bilinear model and a uniaxial constant ‘active’ confinement concrete model were used to idealize steel and concrete, respectively. Bridge bearings and expansion joints are realistically modeled using ZEUS-NL joint elements. Figure 2 shows the models adopted for the expansion bearings, bronze self-lubricating bearings and structural gaps. The bearing idealizations follow the analytical models suggested by Mander at al. (1996), while a tri-linear asymmetric elasto-plastic idealization capable of representing the slippage and collision are employed to model bridge gaps (e.g. Mwafy et al., 2007).

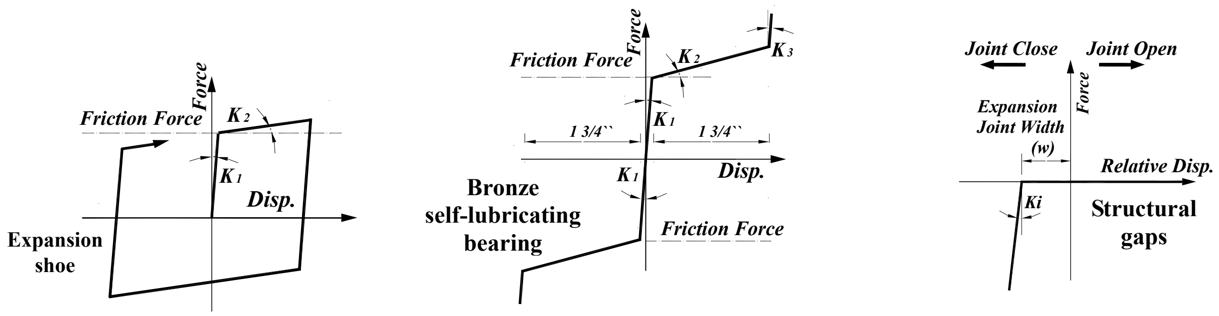


Figure 2. ZEUS-NL model of bridge bearings and expansion joints.

Damping is modeled in ZEUS-NL using Rayleigh damping elements, whereby damping is defined in proportion to the mass and stiffness of the structural member. 2.0% Rayleigh damping ratio is adopted after investigating several damping levels. This damping ratio is applied on substructure, which is the main source of energy dissipation. Higher level of damping is applied on superstructure, which is anticipated to remain in the elastic range. More information about the modeling approach and analysis tool can be found elsewhere (e.g. Elnashai et al., 2006; Elnashai et al., 2007; Mwafy et al., 2007).

2.2. Soil and Foundation Modeling

Based on the soil profile, number of piles and batter angle, thirteen soil-foundation profiles are idealized using OpenSees (McKenna et al., 2006). The number of piles of different footings varies from 9 to 112, depending on the supporting loads, while Bent 19, 20, and 21 are supported on massive caisson. The foundation and soil medium are all modeled with 8 node brick elements. The side boundary of the soil medium is restrained in the horizontal translation. Vertical DOFs of side boundary are released to allow settlement due to gravity loads. All DOFs of the bottom nodes of the soil medium are restrained. Figure 3 describes the OpenSees model and sample results of the Bent 2 foundation system under the effect of cyclic and monotonic loadings. The results confirm that the backbone of the hysteretic curve follows the monotonic pushover curve. It was decided based on this comparison to analyze other foundations profiles under a monotonic loading to estimate their load-deformation relationships. Tri-linear idealizations are adopted to simplify the monotonic pushover curves of different foundation classes to be used as nonlinear soil springs for inelastic analysis of the bridge.

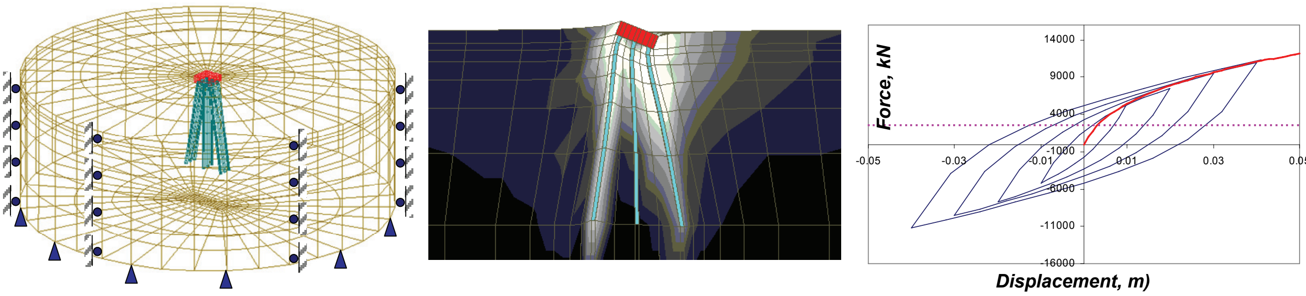


Figure 3. OpenSees model and sample results of Bent 2 foundation system

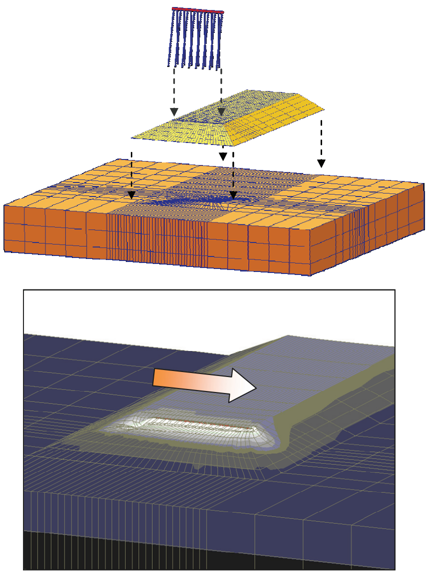


Figure 4. OpenSees abutment model and sample pushover analysis results

Table 1. Response of the bridge in the transverse direction using different abutment idealization

Bent	Top Displacement (mm)			Base Shear (kN)		
	Caltrans	OpenSees	Diff. (%)	Caltrans	OpenSees	Diff. (%)
2	5.2	3.0	74	422	272	55
15	11.7	9.5	23	5072	4869	4
20	38.1	22.7	68	42638	31153	37
21	23.2	10.4	123	30666	18530	65
25	8.1	6.7	22	988	857	15
59	5.9	2.0	195	671	231	191

Response history analysis results from Rec1T-500Y scaled to a PGA = 0.05g

The analytical model of the bridge abutment developed using OpenSees along with sample results are shown in Figure 4. Results of inelastic pushover analyses show a degrading stiffness but at a very high level of force. The response in the longitudinal and transverse directions of the bridge is almost linear. The abutment modeling approach obtained using OpenSees is compared in Table 1 with a more simplified approach suggested by Caltrans (2004). It is clear that the refined OpenSees model results in a significantly different response at the investigated low level of input ground motion (PGA = 0.05g). This confirms the significance of the refined modeling approach adopted in the present study for different components of the Caruthersville Bridge.

3. COMPARISON OF SEISMIC DEMAND VERSUS CAPACITY

Various types of analyses procedures are carried out using the analytical models developed for the bridge and its foundation system. Eigenvalue analyses are first conducted to determine the dynamic characteristics of the bridge. This simple analysis is used as an initial validation tool of the analytical models. Inelastic static pushover analyses are performed in both the longitudinal and transverse directions of bridge bents to evaluate their lateral capacities. This analysis is also conducted for all foundation classes to estimate their capacities, as explained above. Elastic and inelastic response history analysis is initially performed to verify the analytical models and select a realistic damping level. Extensive inelastic time history analyses are finally executed using site-specific input ground motions to examine the response of the bridge under various seismic scenarios with increasing severity. Capacities of the foundations, bents, bearings and expansion joints are compared with seismic demands at various hazard levels. Sample results from these comprehensive analyses are discussed below.

3.1. Input Ground Motions

Probabilistic Seismic Hazard Analyses (PSHA) for hard rock site conditions were performed for the site (Fernandez and Rix, 2006). Three hazard levels corresponding to return periods of 500, 1000 and 2500 years were considered. The developed Uniform Hazard Spectra UHS are shown in Figure 5(a). Three records (Record 1, 2 and 3) were selected from the PSHA records for propagation through the thick embayment deposits (Hashash, 2006). Given the length of the structure, ground motion incoherency, including wave passage, was included in the propagated ground motion. In total 144 input ground motions are used in the assessment study. Figure 5(b) compares between the spectra of synchronous input ground motions generated in the longitudinal direction of the bridge for bedrock and those propagated to the surface. The site response analysis concluded that the surface motions are attenuated in the low period range, but significantly amplified in the mid period range (0.1-1 sec) for the 500 years return period records and the period range above 1 sec for the 1000 and the 2500 years return period records (Hashash, 2006).

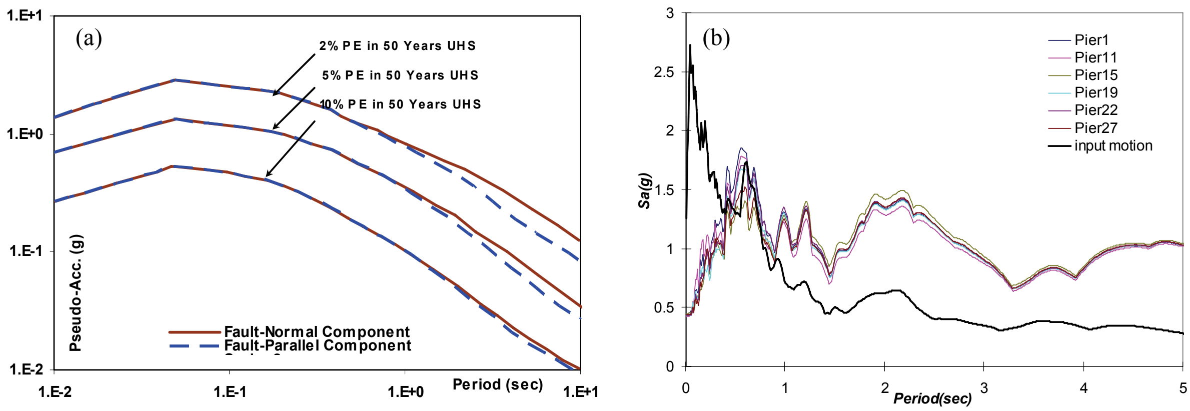
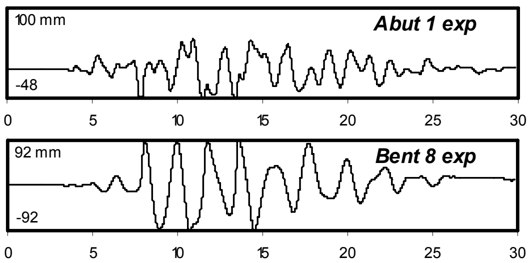


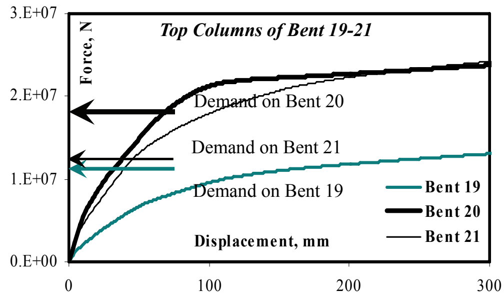
Figure 5. (a) Bedrock Uniform Hazard Spectra (UHS) for return periods of 500, 1000 and 2500 years; (b) Input ground motions generated for bedrock and those propagated to the surface for the 2500 years seismic scenario.

3.2. Capacity versus Demand under the 500 Years Earthquake Scenario

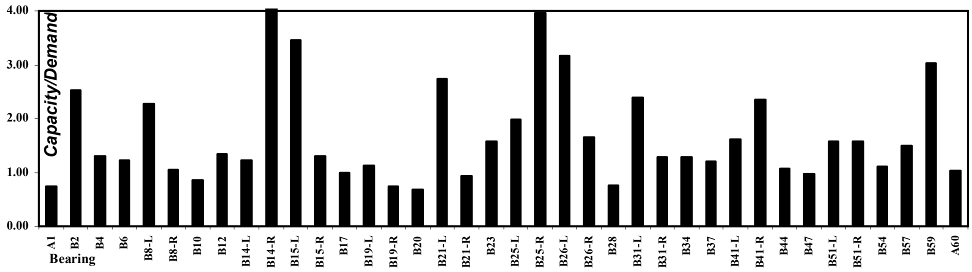
Due to the length of the bridge and the non-uniform distribution of stiffness and mass, higher modes of vibrations notably contribute to the seismic response. Inelastic response history analyses carried out in the transverse directions of the bridge indicate that the drift demands are acceptable (less than 0.63%), with the exception of the high displacement at the truss intermediate hinge. Record 2 significantly amplifies the truss deformation demands compared with other records. The relative displacements between piers along the length of the bridge are inconsistent as a result of the difference in stiffness, which causes high demands on the superstructure. Higher drift demands are observed in the longitudinal direction of the bridge (1.1%) compared with the transverse direction due to its lower stiffness. High deformations are observed in the truss portal frames, which are also observed from free vibration analysis. The high relative displacement demands in the longitudinal direction cause pounding at the two abutments and the expansion joint of Bent 8, as shown in Figure 6(a). The response of the foundation system are acceptable in both directions, while high demands are observed at the top columns of Bents 15-21, as shown in Figure 6(b). The demands of other bents are moderate to high. Several plastic hinges are observed when applying the load in the transverse direction, particularly at the top columns of Bents 15-21. The non-uniform distribution of stiffness along the height of these piers causes high stress concentrations at the top columns. A number of bridge bearings are vulnerability under the effect of this earthquake scenario, particularly the bearings at the expansion joints, as shown in Figure 6(c).



(a) Capacity of expansion joints vs displacement demand (Record 2 in longitudinal direction)



(b) Capacity vs average demand at the top columns of Bents 19-21 (EQ in transverse direction)



(c) Bearing force capacity versus demand (Record 2 in transverse direction)

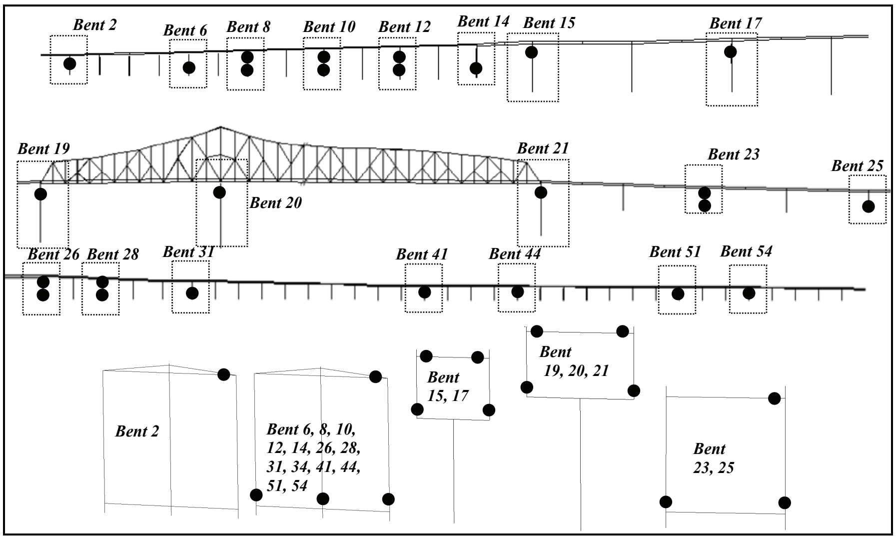
Figure 6. Capacity vs demand of different bridge components under the 500 years earthquake scenario

3.3. Capacity versus Demand under the 1000 Years Earthquake Scenario

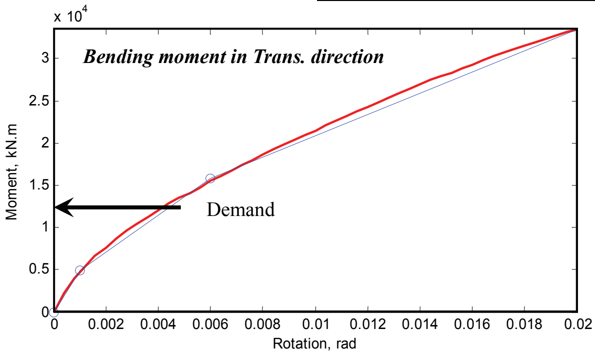
The displacement and force demands increase under this seismic scenario by up to 200% and 100%, respectively, compared with the 500 years seismic scenario. The maximum drift demands in the longitudinal (3.8%) and the transverse (2%) directions are unacceptable. Exceeding the 3% drift limit is considered in the present study as an indication of extensive structural damage. The high relative displacement demands in the longitudinal direction cause pounding at all expansion joints. Unacceptable displacement demands (934 mm) are observed in the transverse direction at the truss intermediate hinge, particularly under the effect of Record 2. Extensive damage and yielding are also detected in several bents. The high ductility demands imposed on the top columns of Bents 15-21 cause severe damage. The spread of plastic hinges is extensive (refer to Figure 7-a), while a severe damage in several bearing is confirmed, as shown in Figure 7(c). Unacceptable yielding is observed in some footings under the effect of the 1000 years seismic scenario, as shown in Figure 7(b).

3.4. Capacity versus Demand under the 2500 Years Earthquake Scenario

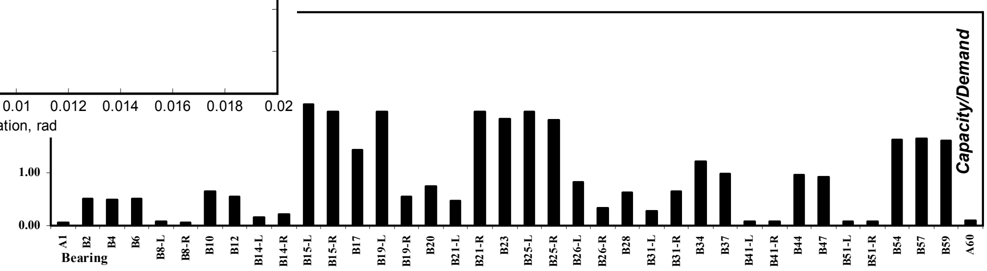
Displacement demands significantly increase in the longitudinal direction of the bridge by up to 90% compared with those from the 1000 years input ground motions. The results obtained from Record 2 confirme that the drift at the top of bents (7.25%) considerably exceeds the collapse limit state. The very high relative displacement demands cause collision between all bridge segments. Significantly higher force demands are also observed in the bents and the foundation system compared with those observed from the 1000 years earthquake scenario. Yielding in the foundation system is confirmed, while extensive damage and yielding at the base of major bents supporting the steel truss are detected. Most of the bridge bearings are vulnerable (capacity/demand ratio is less than unity) under the effect of the 2500 years earthquake scenario, as shown in Figure 8. The results confirm the pressing need to retrofit different bridge components to mitigate the anticipated seismic risk.



(a) Mapping of plastic hinge formation (Record 2 in transverse direction)



(b) Capacity vs bending moment demands of foundation class 3 (EQ in transverse direction)



(c) Bearing force capacity versus demand (Record 2 in longitudinal direction)

Figure 7. Observed response of different bridge components under the 1000 years earthquake scenario

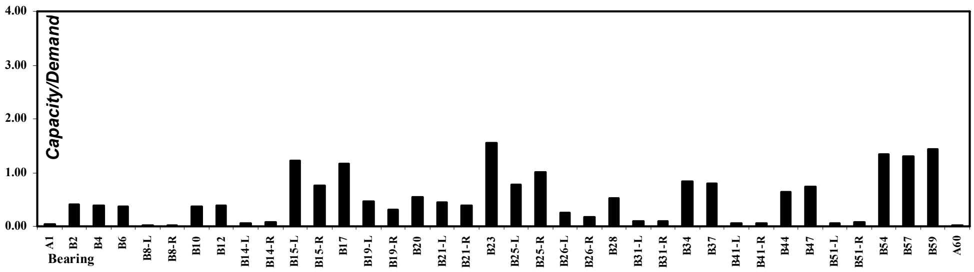


Figure 8. Bearing capacity versus demand under the 2500 years ground motions (Record 2 in the long. direction)

4. CONCLUSIONS

This paper presents highlights of a project carried out to comprehensively assess the seismic response of a 59-span bridge, considering Soil-Structure Interaction (SSI). The bridge was built in the vicinity of a major source of earthquakes; the New Madrid Seismic Zone, and includes typical deficiencies of bridges constructed without adequate seismic provisions. The refined three-dimensional simulations of the bridge and its foundation system carried out using verified analysis tools and a detailed site-specific seismic hazard study enabled the identification of areas of vulnerability of the investigated bridge and assessment of its response at three hazard levels, corresponding to 500, 1000 and 2500 years return periods. The study confirmed that simplifying modeling and design assumptions may have significant impact on seismic response of complex bridges. Higher modes of vibrations notably contributed to the seismic response of the I-155 bridge due to the length of the bridge and the non-uniform distribution of stiffness and mass. Under the 500 years ground motions, the response of the bridge was unacceptable due to the observed yielding and damage in a number of bents and bearings. The demands corresponding to the 1000 years ground motions almost exceeded the collapse limit state and the capacity of bridge components. Indications of yielding in foundations were also observed. The displacement and force demands under the effect of the 2500 years earthquake scenario significantly increased compared with the 1000 years input ground motions and exceeded by far the collapse limit states. The presented assessment study confirmed the urgent need to retrofit different bridge components to mitigate seismic risk and improve public safety. The tools and procedures used for this assessment are applicable to similar situations of complex bridges constructed in a mix of material and structural systems.

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REFERENCES

AASHTO LRFD Specifications. (2007). AASHTO LRFD bridge design specifications, 4th Ed., American Association of State Highway and Transportation Officials, Washington, D.C.

Caltrans. (2004). Caltrans Seismic Design Criteria, California Department of Transportation, Sacramento, CA.

CSI. (2005). SAP2000 – Structural analysis program, Computers and Structures Inc., Berkeley, California.

Elnashai, A.S., Papanikolaou, V., and Lee, D. (2007). Zeus-NL - a system for inelastic analysis of structures - User Manual, Mid-America Earthquake Center, Univ. of Illinois at Urbana-Champaign, Urbana, IL.

Elnashai, A.S., Mwafy, A.M. and Kwon, O-S. (2006). Analytical assessment of a major bridge in the New Madrid seismic zone, Proceedings of the 5th National Seismic Conference on Bridges & Highways, September 18-20, San Francisco, USA.

FHWA. (2006). Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges, Publication No. FHWA-HRT-06-032, Federal Highway Administration, Washington, DC.

Fernandez, J.A. and Rix, G.J. (2006). Seismic Hazard Analysis – Preliminary results, Seismic retrofit study of bridge A-1700 Route I-155, Pemiscot county, Georgia Institute of Technology, Atlanta, Georgia.

Hashash, Y. (2006). Seismic site response analysis, Seismic Retrofit Study of Bridge A-1700 Route I-155, Pemiscot County, Univ. of Illinois at Urbana-Champaign, Urbana, IL.

Mander, J.B., Kim, D.K., Chen, S.S. and Premus, G.J. (1996). Response of steel bridge bearings to the reversed cyclic loading, Report No. NCEER 96-0014, NCEER.

McKenna, F., Fenves, G.L. et al., 2006, "OpenSees - Open System for Earthquake Engineering Simulation", Pacific Earthquake Engineering Research Center, http://opensees.berkeley.edu.

Mwafy, A.M., Elnashai, A.S., and Yen, W-H. (2007). Implications of design assumptions on capacity estimates and demand predictions of multi-span curved bridges. *ASCE J. of Bridge Engineering* **12:6**, 710-726.

USGS (2008). 2008 United States National Seismic Hazard Maps. U.S. Geological Survey Earthquake Hazards Program, <http://earthquake.usgs.gov> (July 22, 2008).