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ABSTRACT

The proposed Great River Bridge is a 1400-foot long cable-stay structure that will be constructed over the Mississippi River between Desha County, Arkansas and Bolivar County, Mississippi. Including the bridge approach structures and approach embankments, the total structure length is approximately 23,500 feet. Seismic issues have controlled most of the structural design. Design ground motions for three typical subsurface profiles were developed and resulted in near-surface peak accelerations between 0.23 and 0.26g. Level ground liquefaction analyses indicated widespread liquefaction in an abandoned channel of the river and sporadic liquefaction elsewhere. Seismic slope stability and lateral spreading analyses indicated minor displacements at the approach embankments, the Arkansas levee, and the Mississippi riverbank; moderate displacements at the Mississippi levee; and major displacements at a 25-foot high natural terrace and the Arkansas riverbank. Conceptual liquefaction mitigation/soil improvement options were investigated.

INTRODUCTION

The proposed Great River Bridge will span the Mississippi River from Desha County, Arkansas to Bolivar County, Mississippi (see Fig. 1). HNTB Corporation is leading a team of consultants in the design of the bridge. URS Corporation, a member of the HNTB team, performed the geotechnical earthquake engineering for the bridge.

The approach structure on the Arkansas side is approximately 14,680 feet long from the beginning of the bridge to the west bank main span anchor pier. Significant crossings in Arkansas include the U.S. Corps of Engineers' (USCOE) Arkansas levee and a wing dam. The proposed main structure is a steel cable-stay structure with a navigation span of approximately 1400 feet (see Fig. 2). The approach structure on the Mississippi side is approximately 2,610 feet in length from the east bank main span anchor pier to the end of the bridge. The USCOE Mississippi levee is the only significant crossing in Mississippi. The total length of the project is approximately 23,500 feet, with approximately 22,548 feet of bridge.

The bridge site is located approximately 120 miles south of the New Madrid Seismic Zone. Bedrock motions (at an equivalent B-C boundary) from this source dominated the seismic analysis. We used an equivalent-linear ground response analysis to propagate B-C boundary ground motions through the near-surface soils. We then used the resulting near-surface motions to evaluate liquefaction, earthquake-induced settlements, and seismic slope stability. Our analyses indicated that liquefaction would occur across the site and would result in liquefaction-induced settlement, lateral spreading at the Arkansas riverbank, and seismic deformations of proposed and existing embankments. To reduce liquefaction-related design issues, we investigated several mitigation options. This paper describes: (1) the field and laboratory investigation; (2) the geologic and seismologic setting; (3) the evaluation of ground shaking, site response, liquefaction, and seismic slope stability; (4) earthquake-related foundation recommendations; and (5) conceptual liquefaction mitigation options considered for the Great River Bridge.

FIELD AND LABORATORY INVESTIGATION

The field and laboratory investigation was divided into a preliminary and final stage. For the preliminary stage, the HNTB team drilled 15 exploratory borings in Arkansas to depths of 150 to 220 feet and 5 borings in Mississippi to depths of 120 to 220 feet. Rotary wash methods were used to advance each of the borings. Split spoon and Shelby tube samples were obtained at various depths in each of the borings. The split spoon samples were obtained in conjunction with standard penetration tests (SPT; ASTM D1586) using

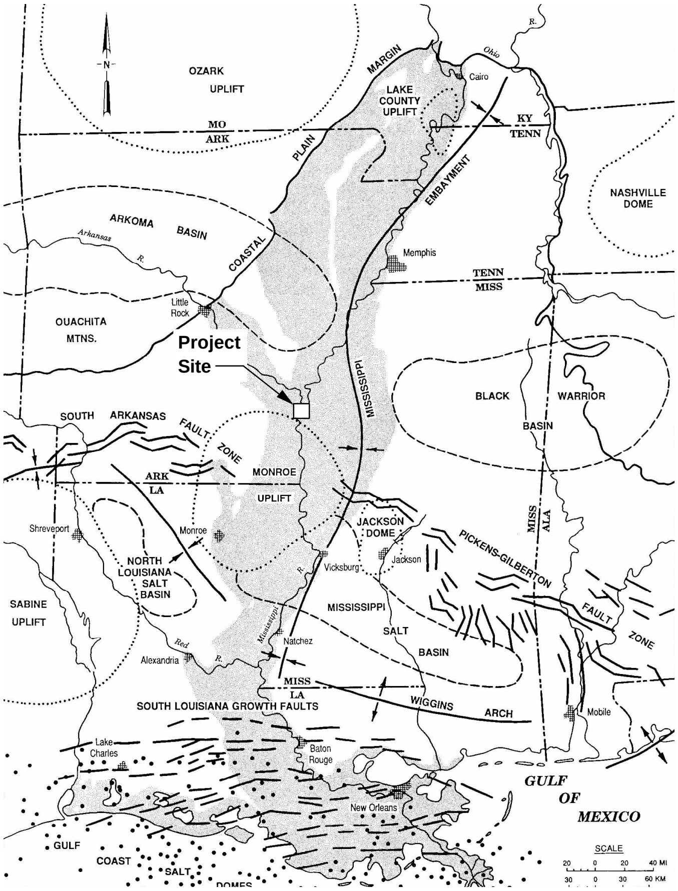

Fig. 1. Project Location and Geologic Structures of Mississippi Embayment (adapted from Saucier 1994).

automatic hammers. Energy measurements were made for each of the automatic hammers used at the site. In addition, we performed 36 piezocone penetration test (CPTu) soundings – 26 in Arkansas and 10 in Mississippi. Piezocone penetration tests were performed in accordance with ASTM D5778. In six of the soundings, we used a seismic piezocone penetrometer to obtain shear wave velocity measurements at one-meter (3.3-foot) intervals. Lastly, we performed downhole geophysical testing to depths of 150 feet in four boreholes to measure compressive and shear wave velocities.

The final field investigation consisted of 67 borings in Arkansas, 19 borings in Mississippi, and 12 borings in the river performed from a barge. In addition, we performed 18 more CPTu soundings to better delineate subsurface conditions in the approach embankment areas and near the Arkansas levee.

The HNTB team conducted laboratory testing on selected samples obtained during the preliminary and final geotechnical investigations. The tests included index, consolidation, and strength tests.

Fig. 2. Proposed Great River Bridge.

GEOLOGIC SETTING

The project site lies within the Mississippi Embayment, part of the Atlantic and Gulf Coastal Plains (see Fig. 1). The Embayment fills a southward-plunging syncline that extends from the southern tip of Illinois to the Gulf of Mexico and is located between the Ozark Uplift and the Nashville Dome (Ervin and McGinnis 1975). The axis of the syncline roughly follows the present course of the Mississippi River (Cushing et al. 1964). The Embayment dates to the Paleozoic period when crustal movement and compaction of deposits caused subsidence and created a basin for the deposition of alluvial sediments (Cushing et al. 1964; Romero and Rix 2001).

The surface geology at the site consists of Quaternary (Pleistocene and Holocene) alluvium overlying a thick column of sedimentary deposits of varying induration ranging in age from Jurassic to Quaternary. Paleozoic basement rock is encountered at about El. -4700 (feet, mean sea level datum), or about 4800 feet below grade. The Jackson Group is a hard, Tertiary-age clay encountered approximately 100 to 140 feet below grade at the project site. The Jackson Group was deposited during the last widespread inundation by the sea. Since the Tertiary period, most of the Mississippi Embayment has remained above sea level (Cushing et al. 1964; Saucier 1994). During the Quaternary, the alluvial fill of the Mississippi River was deposited and extensive terrace systems were formed as a result of changing sea levels (Saucier 1994).

SEISMIC SETTING

Seismic hazard near the project site results both from potential local earthquakes and from larger events occurring farther away. Of particular significance is the New Madrid Seismic Zone (NMSZ) to the north. Damaging earthquakes have also originated in Arkansas and Mississippi outside of the NMSZ. A number of earthquakes with magnitudes between 4 and 5 are documented in southern Arkansas and central Mississippi.

The New Madrid sequence of 1811-12 contained at least three primary earthquakes. The exact locations of the earthquakes are uncertain because of the lack of instrumental data and the sparse local population during the period. Nuttli (1973), on the basis of variations in the spatial extent of the most serious damage and felt effects, suggested that the first event occurred in northeastern Arkansas, the second occurred between New

Madrid, Missouri and the location of the first event, and the third occurred near New Madrid (see Fig. 3). Numerous aftershocks occurred with diminishing frequency for more than 5 years after the three main earthquakes (Nuttli 1982). Recent estimates place the moment magnitude of these earthquakes on the order of 7.6 or 7.7±0.5 (Wheeler and Perkins 2000; Atkinson et al. 2000). Current monitoring shows this zone is still active today, as shown in Fig. 3.

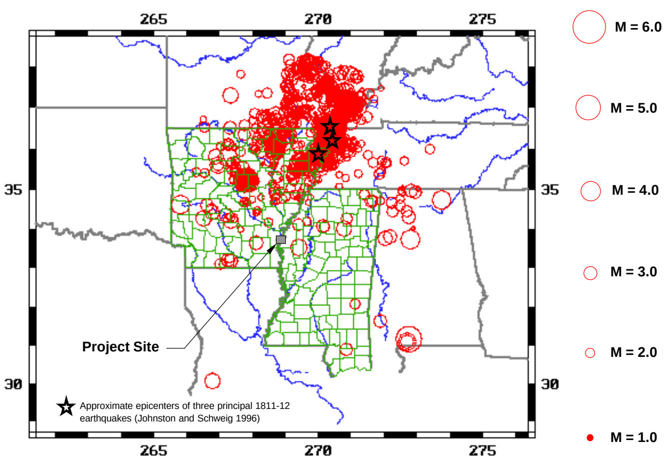

Fig. 3. Earthquakes from 1974 through May 2002 within 500 km of Site (adapted from www.ceri.memphis.edu).

SITE AND SUBSURFACE CONDITIONS

On both sides of the Mississippi River, the site generally is flat and consists of agricultural fields, backswamps, and wooded areas. In Arkansas, existing grade varies from El. 130 to El. 144, except where the current and former Arkansas levees reach El. 166 and 150, respectively. A natural terrace is present about 1500 feet west of the river, east of which the grade drops to about El. 122 until the alignment reaches the riverbank. In Mississippi, grade varies from El. 128 to El. 139, except where the Mississippi levee reaches El. 168.

As discussed above, the upper strata at the site consist of Holocene and Pleistocene alluvial deposits to depths of about 100 to 140 feet overlying Tertiary-age clays. The alluvial deposits can be separated into four distinct geologic categories: overbank silts and clays; abandoned channel (i.e., cutoff meander) deposits; point bar clean sands; and braided stream sand deposits. The abandoned channel can be subdivided into alluvial and lacustrine clays and silty/sandy channel deposits. The braided stream deposits can be subdivided into medium dense alluvial sands and dense alluvial sands and gravelly sands. Table 1 provides in situ test results for the various strata.

Underlying the alluvial deposits is a Tertiary-age, hard, high plasticity clay with occasional layers of low plasticity silty clay, sandy clay and silt. Most likely, the hard clays present at the site belong to the Yazoo Formation of the Jackson Group. Table 1 includes in situ test results for this stratum.

Table 1. In situ test results for subsurface materials

Stratum	SPT blowcount (bpf)	CPT tip resistance (tsf)	Shear wave velocity (ft/sec)
Overbank clay	1 - 12	10 - 70	600
Channel silt/sand	2 - 28	20 - 200	600
Channel clay	0 - 12	5 - 40	450
Point bar sand	3 - 17	40 - 300	600
Med. dense sand	5 - 47	100 - 300	850
Dense sand	10 - 100+	200 - 300+	1250
Tertiary clay	19 - 100+	n/a	1400

Groundwater readings were taken in each of the completed boreholes and piezometers were installed in seven boreholes at various depths to monitor groundwater conditions away from the river. Measurements indicated that the groundwater level ranged from 10 to 20 feet below grade and dropped toward the river. Groundwater levels near the river closely mirrored river level changes.

For the geotechnical earthquake engineering analyses, we selected a design groundwater level at El. 125 along the entire alignment. This elevation is lower than the groundwater level corresponding to the 5-year flood stage, El. 138. For reference, the low water reference plane is El. 99.5. We selected a design groundwater level of El. 125 to provide a reasonable groundwater level considering the severity of the earthquake loading and the low probability that the design earthquake would occur during high river stage of a 5-year flood.

GROUND SHAKING

Based on project design criteria established by the state of Arkansas, we evaluated bedrock shaking at the project site based on seismic design parameters recommended by the 1996 USGS Seismic Hazard Maps (Frankel et al. 1996). We used these seismic parameters to select and modify site-specific bedrock acceleration time histories. We then used these bedrock time histories to perform ground response analyses based on soil conditions encountered at the site.

<u>Bedrock Spectrum and Time Histories</u>

The bedrock response spectrum from the USGS National Seismic Hazard Maps for 2% probability of exceedence in 50-years is shown in Fig. 5. Somerville et al. (2001) suggested that horizontal and vertical bedrock spectra for Eastern North America (ENA) are nearly identical. On this basis, the bedrock response spectrum shown in Fig. 5 corresponds to both horizontal and vertical bedrock ground motions. For later analyses, we de-aggregated the probabilistic response spectrum and concluded that large distant earthquakes occurring in the NMSZ dominated the ground motions at the site. The deaggregated moment magnitude (M_W)–distance pair correspond to approximately 8 and 200 km, respectively.

Table 2 lists the bedrock acceleration time histories selected for the Great River Bridge. The first time history set is from the 1988 Saguenay, Quebec earthquake (M_W 5.8; Somerville et al. 1990). The Saguenay earthquake is the largest earthquake in eastern North America for which strong motion recordings have been obtained. The other two time history sets are from broadband strong motion simulations performed by Somerville et al. (2001). These time histories are representative of M_W 7.5 earthquakes occurring on various segments of the New Madrid fault system at a distance of 75 miles. The selected time histories provide comparable peak accelerations, velocities, and displacements to those anticipated for the deaggregated magnitude–distance pair applicable to the Great River Bridge.

Table 2. Time Histories Selected for Analysis

Source	M_W	Distance (miles)	Components
1988 Saguenay earthquake[(1)]	5.8	27	124, 243, vertical
New Madrid A[(2)]	7.5	75	East, north, vertical
New Madrid B[(3)]	7.5	75	East, north, vertical

[(1)]Recorded at Chicoutimi Nord Station
[(2)]Synthetic records using deep hypocenter
[(3)]Synthetic records using shallow hypocenter

The acceleration time histories in Table 2 do not correspond precisely to the bedrock response spectrum. Therefore prior to conducting ground response analyses, we spectrally-matched these time histories to the bedrock (target) spectrum using the method proposed by Lilhanand and Tseng (1988) as modified by Abrahamson (1993). The 5% damped, spectrally-matched bedrock motions agreed closely with the USGS probabilistic bedrock response spectrum.

Ground Response Analysis

To simplify the ground response analysis and provide a means to differentiate various reaches along the 4.3-mile long project alignment, we calculated the average shear wave velocity to the top of the Jackson Group at discrete stations along the alignment as shown in Fig. 4. We performed ground response analyses for 3 representative soil columns that had average V_s values that fell within the ranges shown in Fig. 4. These representative soil columns were labeled as: Profile 1 – General Conditions; Profile 2 – Abandoned Channel; and Profile 3 – Riverbanks & Point Bar.

The bedrock motions developed for the site correspond to a NEHRP (1997) B-C boundary, i.e., a moderately hard rock

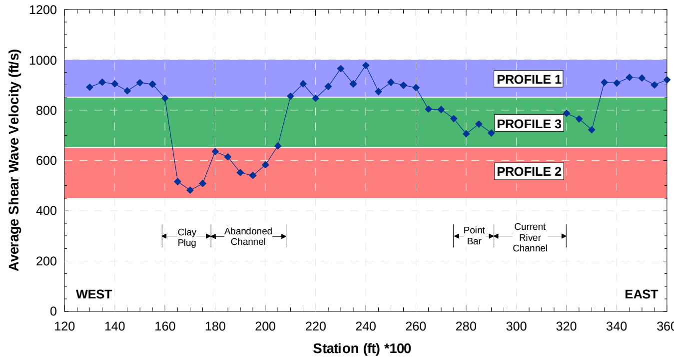

Fig. 4. Average V_s to Top of Jackson Group

site with V_s = 2500 ft/sec. Because of the limited depth of the geophysical investigation, the actual depth of the B-C boundary at the site is not known. Therefore, we selected depths of 200 (i.e., "shallow" column) and 500 feet (i.e., "deep" column) as potential B-C boundary depths based on experience at other sites in the Mississippi Embayment. The shear wave velocity profile was extended to these depths using limited existing geophysical measurements available in the New Madrid Seismic Zone (as summarized in Romero and Rix 2001).

We performed both horizontal and vertical ground response analyses, but only the horizontal analyses are described herein. Cooling et al. (2003) describe the results of the horizontal and vertical ground response analyses in detail. We performed one-dimensional, equivalent-linear ground response analyses using the computer software SHAKE (Schnabel et al. 1972), as most recently updated by URS in 1996. This analysis assumes that horizontal motions can be approximated using vertically propagating shear waves (S-waves).

Figure 5 presents the horizontal ground response results for Profile 1 – General Conditions. Our analyses at each soil profile included runs for each horizontal component of each earthquake (i.e., total of 6 time histories) for the "shallow" soil column, the "deep" soil column, and parametric runs using the "shallow" soil column with ±20% of the average V_s.

Using the ground response analyses for the individual soil profiles, we developed smoothed, design horizontal response spectra for each profile as summarized in Fig. 6. As anticipated, the lower V_s profile yielded a softer response (i.e., lower accelerations at short periods and higher accelerations at long periods), while the higher V_s profile yielded a stiffer response (i.e., higher accelerations at short periods and lower accelerations at long periods). The ground motions correspond to the top of the pile caps for each foundation group (i.e., approximately 10 feet below grade) and do not include the effects of liquefaction. For comparison, Figs. 5 and 6 include NEHRP guideline response spectra for Soil Classes D and E. Using these smoothed design response spectra, we calculated 27 spectrally matched acceleration time histories at the top of the pile caps. The 27 time histories consist of 3 orthogonal components (x, y, and z) x 3 soil profiles x 3 earthquakes.

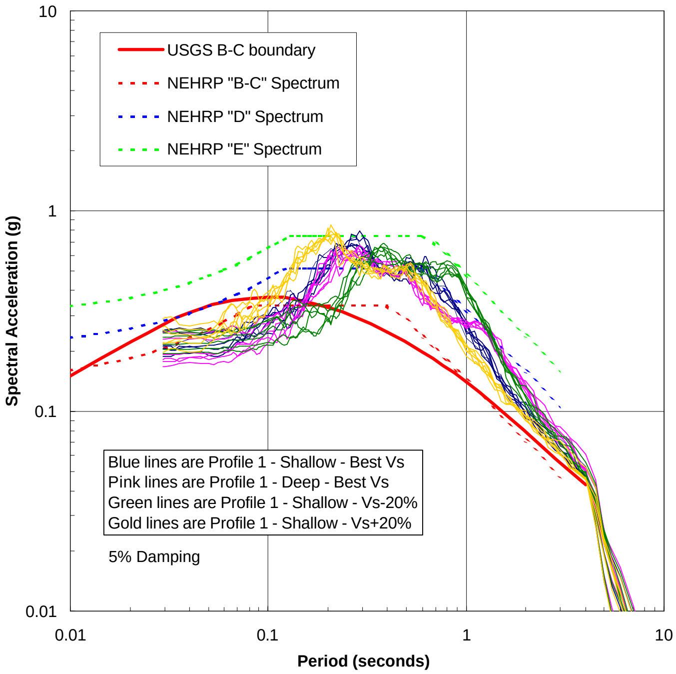

Fig. 5. Response Spectra for Profile 1 - General Conditions.

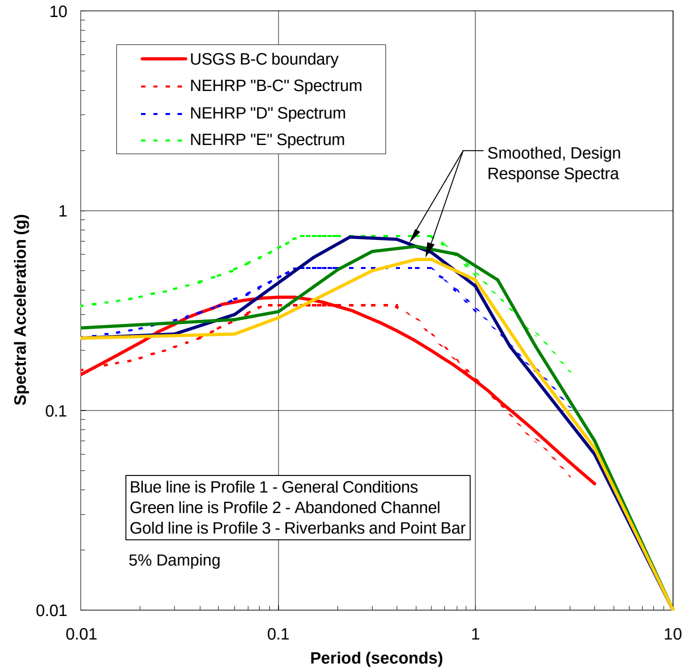

Fig. 6. Summary of Horizontal Design Response Spectra.

LIQUEFACTION

Liquefaction is the process of strength loss in saturated, cohesionless soils as a result of an increase in porewater pressure, often due to earthquake shaking. Liquefaction can have numerous detrimental effects on natural and man-made structures, including settlement, bearing capacity failure, downdrag on deep foundation elements, lateral spreading, and large-scale slope instability (or flow failure). This section describes the analysis of liquefaction under level or mildly sloping (i.e., slopes < 6%) ground conditions. The effects of liquefaction on slope stability are discussed subsequently.

Level Ground Liquefaction Analysis

Level ground liquefaction analysis compares the load imposed by earthquake shaking (i.e., seismic demand) to the resistance of the soil to porewater pressure increase (i.e., seismic capacity). Seed and Idriss (1971) developed the "cyclic stress" method to estimate seismic demand in terms of the approximate seismic stresses imposed by earthquake shaking. The cyclic stress ratio (CSR) is defined as:

$$CSR = \frac{\tau_{avg,cyclic}}{\sigma'_v} \cong 0.65 \frac{\tau_{\max,cyclic}}{\sigma'_v} = 0.65 \frac{a_{\max}}{g} \frac{\sigma_v}{\sigma'_v} r_d \qquad (1)$$

where τ_{cyclic} is the approximate seismic shear stress; a_{max} is the free-field peak ground acceleration (pga); σ_v and σ'_v are the vertical total and effective stresses, respectively; and r_d is a depth reduction factor to account for flexibility of the soil column. We used horizontal pga's of 0.26g for the abandoned channel and 0.23g for the remainder of the site based on the ground response analysis results. We used values of r_d as proposed by Youd and Idriss (1997).

The seismic capacity, or liquefaction resistance, was defined using the CPT procedures proposed by Robertson and Wride (1997) and Stark and Olson (1995) and the SPT procedure proposed by Seed et al. (1985) as modified by Youd and Idriss (1997). We did not use shear wave velocity to evaluate liquefaction resistance due to minor difficulties in the geophysical testing. Both the CPT and SPT procedures define liquefaction resistance using a cyclic resistance ratio for a magnitude 7.5 earthquake ($CRR_{7.5}$), such that the factor of safety against liquefaction can be written as:

$$FS_{Liq} = \left(\frac{CRR_{7.5}}{CSR}\right) MSF \qquad (2)$$

where MSF is a magnitude scaling factor that adjusts the value of $CRR_{7.5}$ to earthquake magnitudes other than 7.5. Since the de-aggregated earthquake magnitude is M_W 8.0, the value of MSF was taken as 0.88 (Idriss 1999). Using the CPT and SPT results, Fig. 7 shows zones of probable liquefaction ($FS_{Liq} \leq 1$) along the project alignment. Liquefaction is most prevalent within the abandoned channel and in the current point bar. Sporadic liquefaction is predicted elsewhere.

Liquefaction-induced Settlements

In reaches that experience liquefaction, post-earthquake settlements will occur as shaking-induced excess porewater pressure dissipates and the liquefied soil re-sediments and

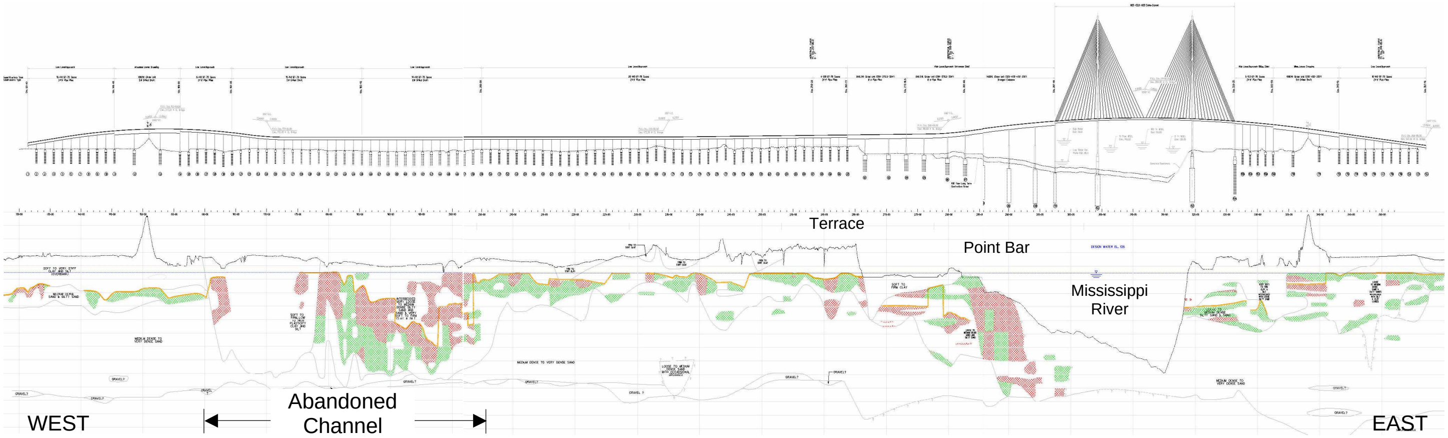

Fig. 7. Potential Zones of Liquefaction along Bridge Alignment (shown as shaded regions).

consolidates. We estimated post-earthquake settlement using the Tokimatsu and Seed (1987) SPT-based procedure.

The magnitude of liquefaction-induced settlements is primarily a function of the liquefied layer thickness and the severity of liquefaction (i.e., FS_{Liq}). As the liquefied layer thickness increases or the FS_{Liq} decreases, liquefaction-induced settlement increases. Figure 8 presents the settlement calculations graphically. These computed settlements are very approximate and could occur as either total or differential settlements. Liquefaction-induced settlements of overlying cohesive or nonliquefied soils typically will result in downdrag on deep foundation elements, as discussed subsequently. Furthermore, in areas of potential lateral spreading, e.g., the current point bar, vertical displacements could be much larger as the result of the lateral movement of liquefied materials.

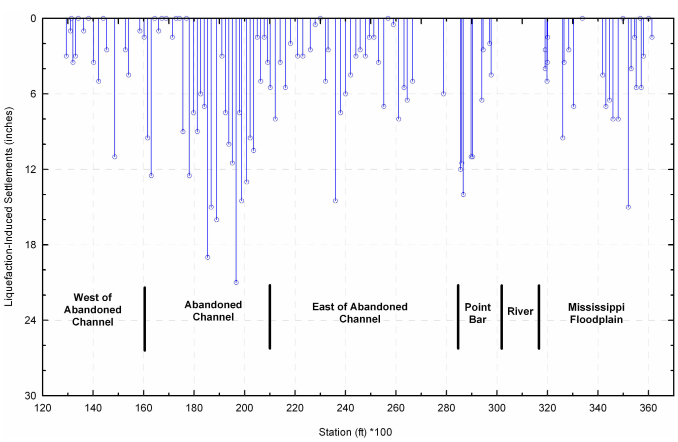

Fig. 8. Estimated Liquefaction-induced Settlements.

<u>Lateral Spreading</u>

Lateral spreading occurs in mildly-sloping ground when the combined downslope static and earthquake-induced inertial forces exceed the undrained resistance of the soil. Because the inertial forces are required to exceed the resistance of the soil, lateral movement ceases when shaking ends. Modeling of lateral spreading is complicated by the complex non-linear behavior of soils (particularly moderately dense soils) under seismic loading. As a result, the state-of-the-practice for lateral spreading analysis is based on empirical relationships to predict lateral displacements that account for various soil properties, seismic parameters, and geometric conditions.

We evaluated lateral spreading at the Arkansas and Mississippi riverbanks. The remainder of the site (with the exception of the embankments and slopes) generally is level, and lateral spreading was judged not to be a concern. We used empirical procedures developed by Bartlett and Youd (1995) and Rauch et al. (2000) to estimate magnitudes of lateral displacement. The Bartlett and Youd (1995) and Rauch et al. (2000) methods provide estimates of lateral displacement for both free-face and sloping ground conditions. We considered the sloping ground model for the Arkansas bank and the free-face model for the Mississippi bank.

Using an average slope of 4%, we estimated a lateral displacement ranging from 3 to 13 feet for the Arkansas riverbank. The zones of soil subject to lateral spread [i.e., $(N_1)_{60} < 15$] at the Mississippi riverbank are discontinuous. Therefore, we judged that lateral spreading of this bank was not likely. With a permanent concrete revetment placed on the bank, the bank geometry and potential for lateral spreading are unlikely to change significantly over the structure's lifetime.

SEISMIC SLOPE STABILITY

We conducted seismic slope stability analyses for the approach embankments, the Arkansas and Mississippi USCOE levees, and the natural terrace on the Arkansas side of the river. The seismic slope stability analysis first considered whether or not the foundation soils underlying the slope are contractive (i.e., susceptible to liquefaction flow failure) using the procedure proposed by Olson and Stark (2003). If the foundation soils are contractive, we used the Seed and Harder (1990) procedure (as updated by Harder and Boulanger 1997) to evaluate the triggering of liquefaction. If liquefaction was predicted to occur, we conducted a slope stability analysis considering only gravity forces and using the liquefied shear strength. If flow failure was unlikely (i.e., FS against flow failure > 1), we estimated maximum horizontal seismic displacements using the Makdisi and Seed (1978) method and the liquefied shear strength in the liquefiable soils. We capped the horizontal deformation to that required to mobilize the liquefied shear strength based on a limiting shear strain of 50% (Byrne 1991).

At the approach embankments, we predicted liquefaction of thin, but continuous, sandy foundation layers. We developed design charts relating embankment height, fill slope angle, and fill material (sand versus clay) to FS for slope stability. Figure 9 presents an example stability analysis for the Arkansas approach embankment. In addition, we computed potential seismic lateral displacements for each trial embankment. Based on a cost analysis comparing embankment height, bridge costs, predicted lateral displacements, and soil improvement costs (to limit lateral displacement), the design team elected to minimize embankment heights and to not use soil improvement at the approach embankments. The final embankment configurations are 2H:1V sideslopes and heights of 17.5 ft in Arkansas and 12 ft in Mississippi.

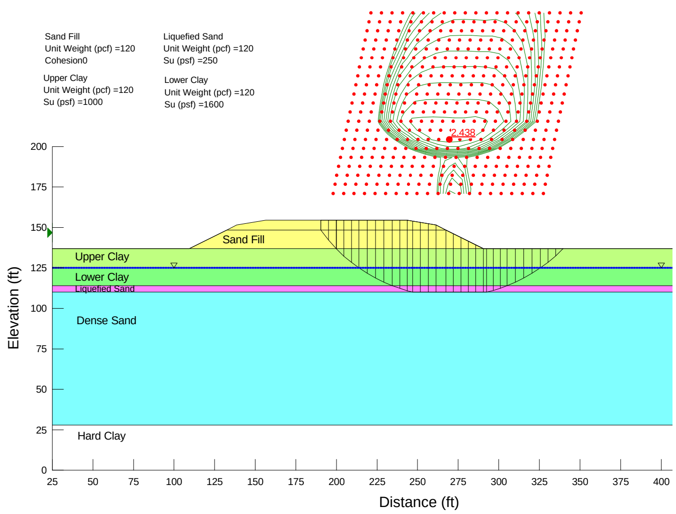

Fig. 9. Example Seismic Slope Stability Analysis

At the Arkansas USCOE levee, liquefaction is likely in a sandy layer about 25 to 30 feet below grade. Because of the depth of this layer, we predicted a seismic lateral displacement of less than 4 inches. Thus, no special precautions were taken at the Arkansas levee. At the Mississippi levee however, we predicted lateral displacements on the order of 3 feet due to sliding on a near-surface liquefiable layer. To account for potential liquefaction-induced displacements, we elected to move the bridge piers outside the zone of displacement.

At the natural terrace, a loose sandy silt deposit is present. We predicted that extensive liquefaction would occur in the sandy silt (see Fig. 10), resulting in a liquefaction flow failure. The toe of the terrace is located near a bridge pier; therefore, we considered soil improvement at this location as discussed subsequently.

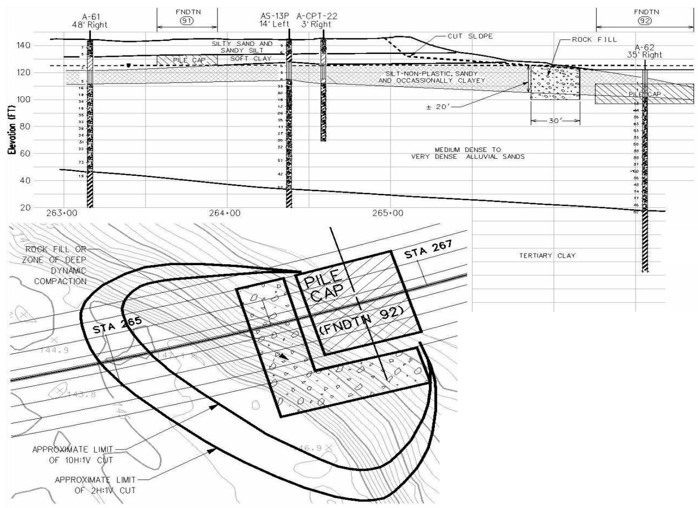

Fig. 10. Liquefaction and Ground Improvement at Terrace

FOUNDATION DESIGN RECOMMENDATIONS

We provided the structural engineers with foundation design recommendations for three conditions: (1) no liquefaction; (2) level ground liquefaction; and (3) liquefaction with lateral ground displacement.

Reaches with No Liquefaction

Ground shaking poses a significant design concern for the bridge foundations. Reaches with no liquefaction will experience significant ground motion amplification. For example in the abandoned channel, ground response analyses indicated that at a period of 1 second, horizontal motions will be amplified about 5 times that of the B-C boundary motions. For structural multi-modal analysis, we provided smoothed horizontal and vertical response spectra. In addition, we developed spectrally-matched horizontal and vertical acceleration time histories at the top of foundation level for structural time history analysis.

For lateral load analysis of driven pile and drilled shaft groups in reaches with no liquefaction, we recommended group efficiency (G_e) factors of 1.0 and 0.5, respectively, for typical spacings of 3 times the pile diameter. An average value for the group was recommended in lieu of individual row p-multipliers because seismic shaking occurs randomly rather than in one direction.

Reaches with Level Ground Liquefaction

Because liquefaction likely will occur during shaking, the foundation analysis must account for the reduced lateral stiffness of the liquefied soil during shaking and changes in side friction for axial loading.

To assess lateral resistance, we recommended the soft clay p-y model (Matlok 1970) and liquefied shear strengths that increase with depth based on a liquefied strength ratio approach (Olson and Stark 2002). This approach yielded results consistent with the approach recommended by Po Lam et al. (1998) of using 10% of the initial, static sand p-y values in liquefied soil. In areas of marginal liquefaction, installing displacement piles with spacing-to-diameter (s/d) ratios of about 3 likely will densify the soil sufficiently to preclude liquefaction. In these locations, interior pile p-multipliers of 1.0 can be used while perimeter piles should use reduced p-y values for liquefaction. In zones of more severe liquefaction, we anticipated that pile installation would not preclude liquefaction. Therefore, all piles (interior and perimeter) should use reduced p-y values. For post-earthquake analyses, all the soils will provide lateral resistance to pile movement. Materials above the bottoms of the pile caps generally were non-liquefiable and we recommended that passive resistance behind the cap be incorporated in design.

During liquefaction, we recommended that side friction from the liquefied soils and cohesive soils overlying the liquefied soils be discounted. Following the earthquake, shaking-induced excess porewater pressures will begin to dissipate in the liquefied soils leading to settlement of the overlying deposits. This settlement will cause downdrag on deep foundation elements. The actual zones of soil that liquefy will not cause downdrag on deep foundation elements.

Reaches with Lateral Soil Displacement

Liquefaction-induced lateral soil displacement is anticipated at the natural terrace, the Arkansas riverbank, and the Mississippi levee. Lateral soil displacement will induce lateral forces on foundation elements. There is large uncertainty in estimating liquefaction-induced lateral forces on foundations. Recent studies from Mageau and Stauffer (1998) based on case history back-analysis suggest that liquefied soil (with strengths similar to that at the project site) exerts a force of 1500 psf on large diameter (±20-ft) drilled shaft foundations. Arching effects between closely spaced piles are unknown.

We recommended using a lateral soil pressure of 1500 psf on the projected area of the foundation and piles or shafts. If present, we suggested that non-liquefied soils above the zone of liquefaction will apply a passive pressure on an area that is 1.5 times wider than the projected area of the foundation. Empirical evidence suggests that this passive pressure should be calculated using Rankine theory (not log-spiral). Due to the large lateral forces and uncertainties in estimating these forces, we recommended liquefaction mitigation be utilized where practical rather designing the foundations to resist liquefaction-induced forces.

LIQUEFACTION MITIGATION CONCEPTS

We explored options to mitigate liquefaction at the Arkansas terrace and the Arkansas riverbank. At the Mississippi levee, the design team moved the nearest bridge pier outside of the likely zone of lateral displacement.

Arkansas Terrace

As indicated above, we anticipate that liquefaction of the sandy silts underlying the terrace will trigger a flow failure of the terrace slope and threaten a bridge pier (see Fig. 10). We considered several options to mitigate liquefaction at this location, including: (1) flatten the slope and install a rock key trench; (2) deep dynamic compaction; and (3) deep soil mixing. In all cases, flattening the slope would reduce the likely failure mode from flow failure to lateral spreading and was highly recommended. We did not perform detailed cost comparisons of these options, but we developed preliminary costs for options (1) and (2).

We assumed that a zone of soil as shown in Fig. 10 should be improved for both options. Sheetpiling may be driven (and later extracted) to allow vertical excavation. Assuming typical excavation costs of $\$6/yd^3$ and shot rock replacement costs of $\$25/yd^3$, the overall cost of this improvement option was about \$550,000. The unit cost of deep dynamic compaction (DDC) is about $\$1.50/ft^2$, resulting in an improvement cost of about \$120,000. However because of the high silt content of the soils underlying the terrace, we recommended that a field test section be performed to verify the effectiveness of DDC. In addition, we recommended that a gravel pad be placed over the entire area to be densified prior to production DDC work to improve its effectiveness.

Arkansas Riverbank

We anticipated that the Arkansas riverbank would experience lateral spreading on the order of 3 to 13 feet. The depth of lateral spreading was expected to be between 15 and 50 feet (see Fig. 7). As a result, flattening the slope and providing scour protection would reduce the overall magnitude of displacement but it would not prevent lateral movements and the resulting forces on the foundations. Because of the thickness of liquefiable soils and the difficulty in accurately estimating lateral spreading forces, we strongly recommended soil improvement and subsequent bank protection in this area. Available mitigation options at this location include: (1) vibratory compaction; (2) explosive compaction; (3) deep soil mixing; and (4) jet or compaction grouting. In all cases, we recommended erosion protection for the riverbank so that the improved soil is not removed by scour. The following cost estimates do not include scour protection.

At four foundation units, we estimated that the depth of lateral spreading would likely approach 50 feet. In this reach, we anticipated that improvement by vibrocompaction would be most economical. We estimated that 3-foot diameter vibro-compaction columns on an 8-foot spacing to a depth of 55 feet (i.e., 5-foot penetration into dense sand) would be sufficient in this area to resist the lateral spreading forces and not transmit lateral spreading forces to the caisson. The required zone of improved soil is shown schematically in Fig. 11. The improvement cost for each caisson is about $1.1 million assuming a unit cost of \$10/yd^3. (This estimated price does not includes a premium for barge work.)

At two foundation units, we estimated that the depth of lateral spreading would decrease to about 20 feet. In this reach, making the same assumptions as above but decreasing the size of the improved zone, the improvement cost was about $200,000 per caisson.

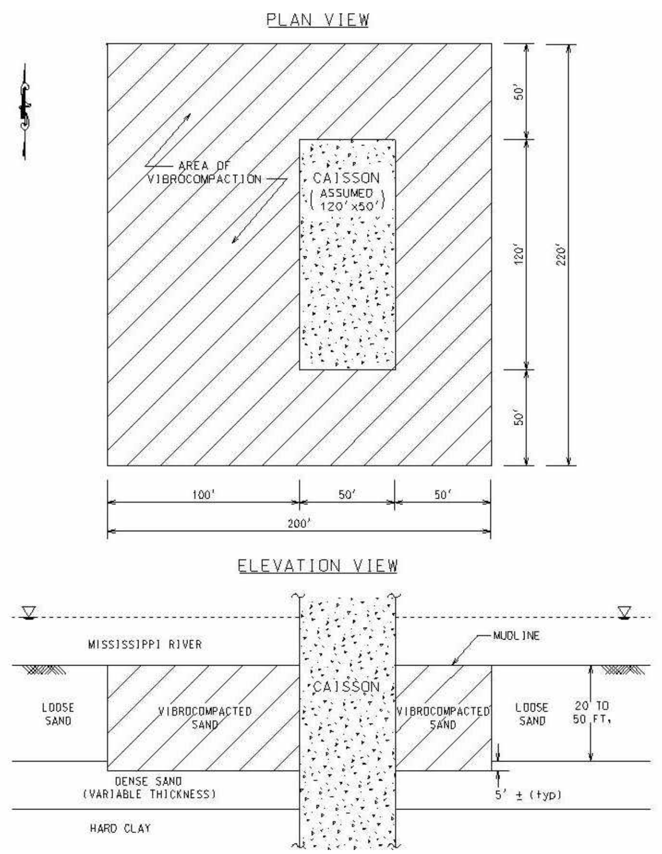

Fig. 11. Preliminary Liquefaction Mitigation Concept for the Arkansas Riverbank. (Wider Zone Faces Arkansas Bank.)

CONCLUSIONS

The Great River Bridge, still under design (in 2003), has proven to be a very challenging project due to the stringent seismic design criteria. Predicted magnitudes of ground shaking, liquefaction, and liquefaction-induced lateral soil displacements have forced the design team to evaluate more detailed analysis techniques and liquefaction mitigation concepts. As design progresses, these options will be refined and incorporated into the bridge design to achieve a constructible and cost-effective solution.

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