31 Mar 2001, 8:00 am - 9:30 am

Earthquake Assessment of Critical Structures for Route US 60 Missouri

Ronaldo Luna
Missouri University of Science and Technology, rluna@mst.edu

Genda Chen
Missouri University of Science and Technology, gchen@mst.edu

Yulman Munaf
University of Missouri--Rolla

Huimin Mu
University of Missouri--Rolla

Shamsher Prakash
Missouri University of Science and Technology, prakash@mst.edu

See next page for additional authors

Ronaldo Luna, Genda Chen, Yulman Munaf, Huimin Mu, Shamsher Prakash, Paul Michael Santi, Tom Fennessey, and Dave Hoffman

Earthquake Assessment of Critical Structures for Route US 60 Missouri

Ronaldo Luna, Genda Chen, Yulman Munaf, Huimin Mu, Shamsher Prakash and Paul Santi
University of Missouri-Rolla
Rolla, Missouri, 65409 - USA

Tom Fennessey
Missouri Dept. of Transportation
Jefferson City, MO 65102 - USA

Dave Hoffman
Missouri Dept. of Natural Resources
Rolla, Missouri 65401- USA

Abstract

The Missouri Department of Transportation initiated a study of that segment of Route US 60 that has been officially designated as "emergency vehicle priority access". The objectives were to establish a current subsurface and earthquake design geographic information systems (GIS) database for the designated US 60 corridor, and to conduct detailed earthquake assessments at two critical bridge sites along US 60. Databases have been established for current subsurface and earthquake data for the US Route 60 corridor in Butler, Stoddard and New Madrid Counties. These databases serve as the beginning of a larger regional or statewide database for future development and usage by MoDOT. Detailed earthquake site assessments have been conducted for two critical US 60 roadway bridge sites (Wahite Ditch and St. Francis River Bridge). Liquefaction potential, slope stability, abutment stability, and structure stability analysis were performed at both sites for selected "worst case scenario synthetic bedrock ground motions" based on New Madrid source zone earthquakes with 2% and 10% probabilities of exceedance in fifty years. Site assessments indicate that both the Wahite Ditch and St. Francis River bridges could be rendered unusable by strong ground motion with a 2% probability of exceedance in the next fifty years. Studies indicate that the bridge themselves would not fail – rather they would probably be rendered unusable because of damage to their abutments and the failure of their approaches (as a result of slope instability and liquefaction). Problems could be exacerbated by the localized flooding as a result of levee failure and/or damage to the Wappapello Dam. A scheme of retrofit of these structures will be developed later.

INTRODUCTION

Southeast Missouri experiences relatively small magnitude earthquakes on a regular basis, and is the site of several of the largest magnitude earthquake events to strike North America in recorded history. Experts agree that similar (or greater magnitude) earthquakes will strike this region again. Geologic conditions in southeast Missouri are such as to make this region one of the most seismically susceptible in the country, based on its damage potential from intrinsically susceptible soil, high water levels and vast expanses of flood sensitive ground. If a high magnitude earthquake struck southeastern Missouri today, infrastructure could be devastated. Levees and dams could be breached, bridges across the Mississippi and Meramec rivers could collapse or be otherwise rendered unusable, extended sections of highway would be closed by landslides, floods, soil liquefaction, and the failure of roadway bridges and overpasses. The network of facilities and services required for commerce and public health in south St. Louis, Sikeston, Cape Girardeau and surrounding communities could be devastated. Utilities, including electrical power, communications, oil and gas distribution, sewage, waste disposal and water, could be disabled until emergency repair crews were able to access these communities. SE Missouri could be effectively cut-off from the rest of the world.

Because of the compelling need to reopen emergency vehicle access routes into St. Louis, Sikeston and Cape Girardeau following a devastating earthquake, the Missouri Department of Transportation (MoDOT) in conjunction with other state agencies have designated specific routes for vehicular access of emergency personnel, equipment and supplies in the event of a major earthquake in southeast Missouri. These routes include portions of US 60 and US 100 (see Figure 1).

Preliminary site-specific earthquake assessment of two critical bridge sites along US 60 and the development of an initial geotechnical database were conducted as part of Phase I of this multi-agency (MoDOT, MoDNR and UMR) initiative. The methodologies developed in this study will be used to establish an assessment protocol. The interpreted geotechnical data will be used for future prioritization and retrofit of deficiencies noted at the bridge sites studied.

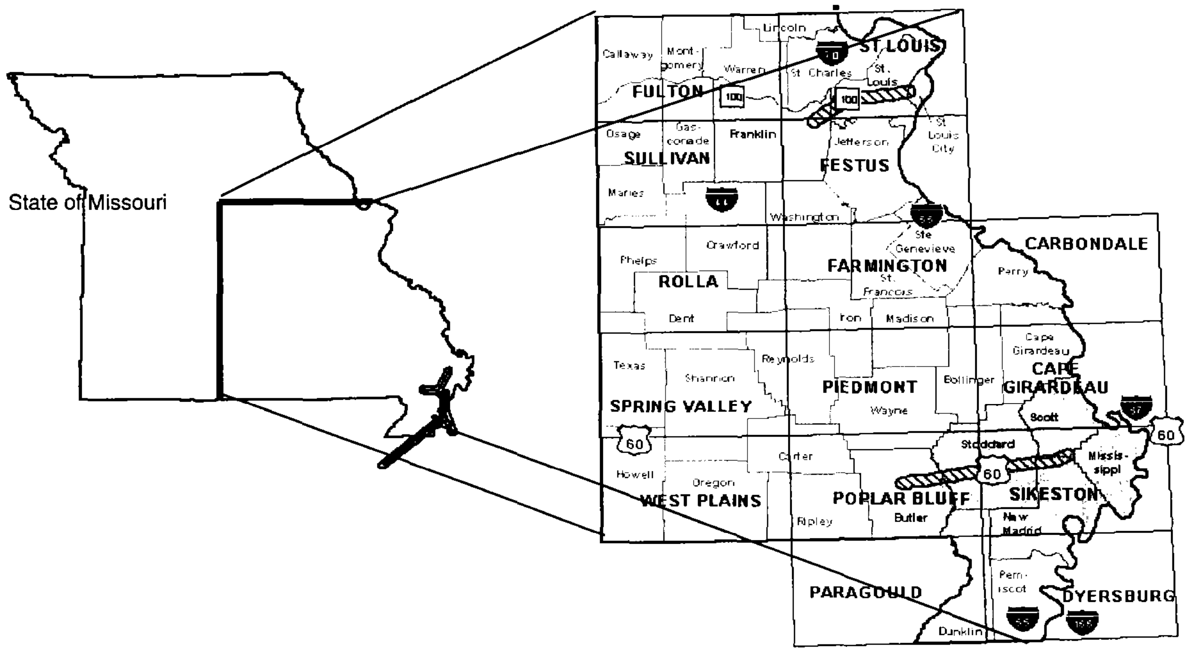

Figure 1 – Vicinity Map and Emergency Vehicle Access Routes

The designated US 60 corridor crosses the Butler, Stoddard and New Madrid Counties and was visited by the members of the MoDOT/MoDNR/UMR research team. Bridge sites with critical roadway features were ranked based upon geologic factors, structural factors and perceived criticality/risk factors. The top two sites with differing geologic settings were selected for detailed site-specific earthquake assessment (Wahite Ditch Number 1 bridge and the Saint Francis River bridge).

Detailed earthquake site assessments were conducted for both critical US 60 roadway sites. Site assessments included: subsurface exploration, and laboratory testing to identify subsurface materials and their engineering properties; evaluation of available seismic records and procedures to characterize the ground motions associated with various design earthquake events; and evaluation of the response of the subsurface materials and the existing bridge structures to the estimated ground motions.

The goals of the site assessments at these two locations were to:

i) Estimate peak magnitude and duration of ground surface motion (including amplification/damping) associated with various events at each site.

ii) Evaluate the susceptibility of each site to quake-induced slope instability and liquefaction.

iii) Estimate shaking effects on the various types of existing bridge structures at each site.

iv) Compare ground motion and structural response parameters from site specific earthquake analysis method with those from AASHTO response spectrum analysis method and provide preliminary guidance regarding selection of the analysis method at future sites.

v) Determine if site conditions could be exacerbated by localized flooding as a result of canal and/or dam failure

EARTHQUAKE GROUND MOTION

Liquefaction potential, slope stability, abutment stability, and structure stability analysis were performed at both sites for selected "worst case scenario bedrock ground motions" with probability of exceedance (PE) of 2% and 10% in 50 years. Ground motion analysis utilized synthetic ground motions for a New Madrid source zone.

In traditional site-specific earthquake hazard assessment, an initial step is to select rock base ground motion(s) at the site. This usually requires a site-specific seismic hazard analysis taking into consideration site conditions and all known earthquake sources (fault zones, epicentral distances, geological conditions, etc.). However, in the central U.S. there is a paucity of recorded strong ground motion from the New Madrid area that can be used for such purposes. Therefore investigators in the research community have resorted to procedures that develop synthetic seismic ground motions at a site (rock base).

A thorough search (literature and via professional contacts) revealed that acceptable, published synthetic ground motions are available at only three locations in proximity to the bridge sites studied (Saint Louis, MO; Memphis, TN and Carbondale, IL; Wen and Wu, 2000). (These three locations were originally selected due to their population density and level of importance.) These three locations and the bridge sites studied (St. Francis and Wahite) are effectively surrounded by the three locations for which synthetic ground motions are available. A "worst case scenario" (in terms of soil, slope and structure response) was developed for each bridge site (for both PE time periods) based on all available synthetic ground motions (from all three locations) using the one-dimensional wave propagation analysis program SHAKE. A profile of peak accelerations for each soil layer was generated for each bridge site and for each synthetic ground motion. The ground motion with the highest peak ground acceleration (maximum PGA) at the surface (for each of the two PE values) was used to develop a "worst case" scenario for that PE value. It is acknowledged that site-specific synthetic ground motions would probably be preferable to those generated through the "worst case scenario" described above.

SEISMIC RESPONSE OF SOILS

Programs SHAKE and SHAKEDIT were used to transfer the rock motion to the above soil layers. Liquefaction analysis was performed using the Seed and Idriss (1971) simplified method, as modified by Youd et al. (1997).

SHAKE Analysis

Program SHAKE computes the responses in a system of homogenous, viscoelastic layers of infinite horizontal extent subjected to vertically traveling shear waves. The adopted synthetic ground motion is described above. Soil profiles for the St. Francis River and Wahite Ditch bridge sites, with corresponding soil properties of layers of St. Francis and Wahite sites were developed for the analysis. The shear wave velocity (Vs) measured by the seismic cone penetrometer at the St. Francis site was consistently below 400 meters per second within the soil column.

The peak ground motion for each layer above the base rock is larger than the rock ground motion. This means that, the ground motion amplification has occurred for this site. The calculated peak ground motion for each soil layer was plotted against depth. At the ground surface a peak ground motion ranged from 0.22g to 0.4g for the PE 10% and 2% in 50 years, respectively. (Anderson, et al., 2000)

Liquefaction Analysis

Soil Profile

The soil profile at St. Francis Bridge is used in this paper to present the analysis procedures. Boreholes and cone penetrometer tests were located close to the bridge abutment. Soil at this site consists of clay with medium to stiff consistency up to 18 ft depth and about 30 ft thickness of dense to very dense sand layer. A brief description of the soil profile, which includes observed SPT (N) and corrected $(N_1)_{60}$ values are shown in Figure 2. The shear wave velocity profile

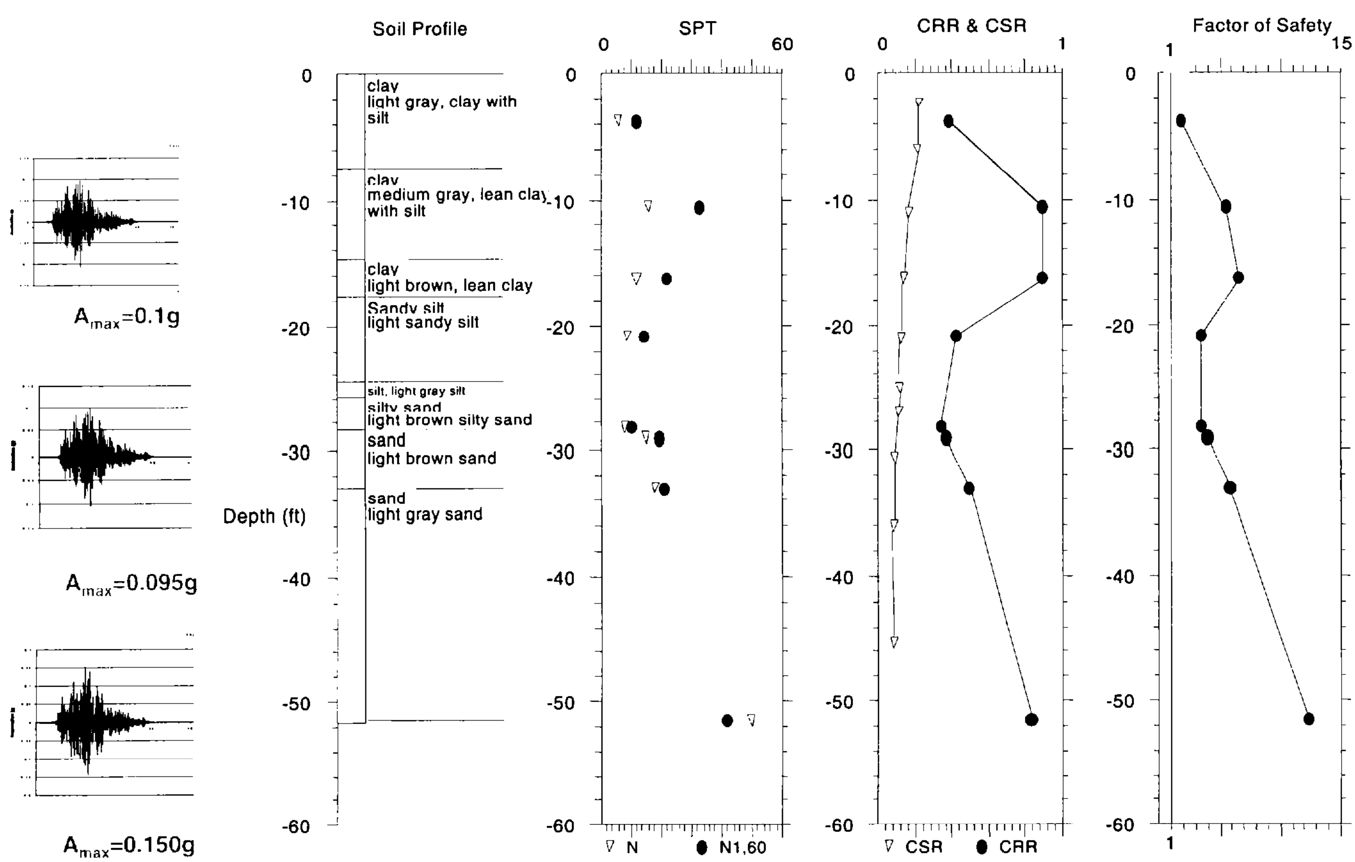

Figure 2 – Soil Profile, seismic ground response and liquefaction at St. Francis site

at this site was measured by CPT test up to about 40 ft of depth.

Wahite Ditch site followed similar analysis procedures. The subsurface soil at this site consists of clay of high plasticity up to 20 ft depth and about 170 ft thickness of medium sand, containing numerous thin gravel lenses.

Liquefaction Evaluation Potential

Liquefaction potential of that site is obtained by comparing the value of Cyclic Resistant Ratio (CRR) and the Cyclic Stress Ratio (CSR) to obtain the factor of safety against liquefaction. Figure 2 shows the plots of the CRR, CSR and factor of safety (FOS) against liquefaction with depth for PE 10% in 50 years of St. Francis site. For PE 10% in 50 years, the factor of safety is higher than that recommended value of 1.4 and those sites will be safe against damage due to liquefaction. However, for PE 2 % in 50 years, the factor of safety is less than 1.4 and the soil liquefied for PE 2% in 50 years at both sites.

SEISMIC RESPONSE OF BRIDGE ABUTMENTS

The older bridge (1978) at the St. Francis site (deck is sitting on the abutments) was analyzed, and a detailed analysis of abutment of this bridge was conducted. The new bridge (1992) has integral abutment with the deck. This requires a highly involved and sophisticated analysis that should be performed in recommended follow-up studies.

Displacements of bridge abutment were computed considering it as a two-degrees-of-freedom (2-DOF) model. Choudhry (1999) and Wu (1999) have proposed methods to calculate displacements of bridge abutment and retaining wall due to earthquake, based on permanent displacement concept. This method/procedure has been modified to predict seismic response of bridge abutment supported on piles.

Procedures of this method are presented as follow:

1. Seismic response of bridge abutment was calculated based on time history of acceleration acting on the base of foundation abutment.

2. The bridge abutment is supported on two rows of vertical and battered piles. The pile provided stiffness and damping and the abutment provides the mass.

3. Two degrees of freedom motion were used to obtain displacement of bridge abutment.

4. Mononobe-Okabe method was used to compute force acting in backfill. Vertical load acting on the bridge abutment was obtained based on reaction force of bridge structure from output analysis of bridge super structure.

5. Non-linear soil properties were used to obtain stiffness and damping parameter of base soil layer.

6. Spring and damping constants were calculated using recommendation of Novak's (1974) and Novak and El-Sharnouby (1983).

7. Point of rotation was assumed at the heel of bridge abutment. (Wu, 1999, Choudhry, 1999).

8. Displacements were calculated based on active state condition. This means that, permanent displacement occurred if acceleration acts towards the fill and the wall move away from the fill.

9. Total displacements at top of bridge abutment are calculated by cumulative of sliding and overturning displacement.

Load Acting on Bridge Abutment

Loads acting on bridge abutment are:

i) Self weight of abutment and time dependent inertia force.

ii) Vertical load of the deck and time dependent inertia force.

iii) Lateral static and time dependent load from backfill of soil .

Vertical load acting on bridge abutment is obtained from reaction force of dead and live load. The seismic motion at subsoil layer 1 (Figure 2) is used in typically this analysis.

Bridge Abutment and Pile Parameters

Bridge abutments and piles are cast in-place concrete with the following properties;

Diameter of pile section	= 0.508 m
Length	= 13.4 m
Unit weight of concrete	= 23.58 kN/m^3
Elastic modulus concrete	= $2.15x10^7$ kN/m^2
Poisson ratio (υ)	= 0.3
Moment inertia of pile	= 0.00316 m^4

Table 1 - Soil Properties Used for Abutment Analysis

Backfill soil	Foundation soil around the pile
Unit weight = 19.54 kN/m^3	Unit weight = 21.56 kN/m^3
Internal friction angle (ϕ) = 33°	Internal friction angle (ϕ) = 35°
Friction angle between soil and wall (δ) = 33°	Friction angle between soil and wall (δ) = 23.3°

Calculated Time Dependent Displacement of Abutment

Using the selected synthetic ground motions referenced earlier and soil properties in Table 1, Figure 3 shows the time histories of sliding, rocking and total permanent displacement of bridge abutment. The sliding displacement of bridge abutment is 0.2 to 1.0 ft.

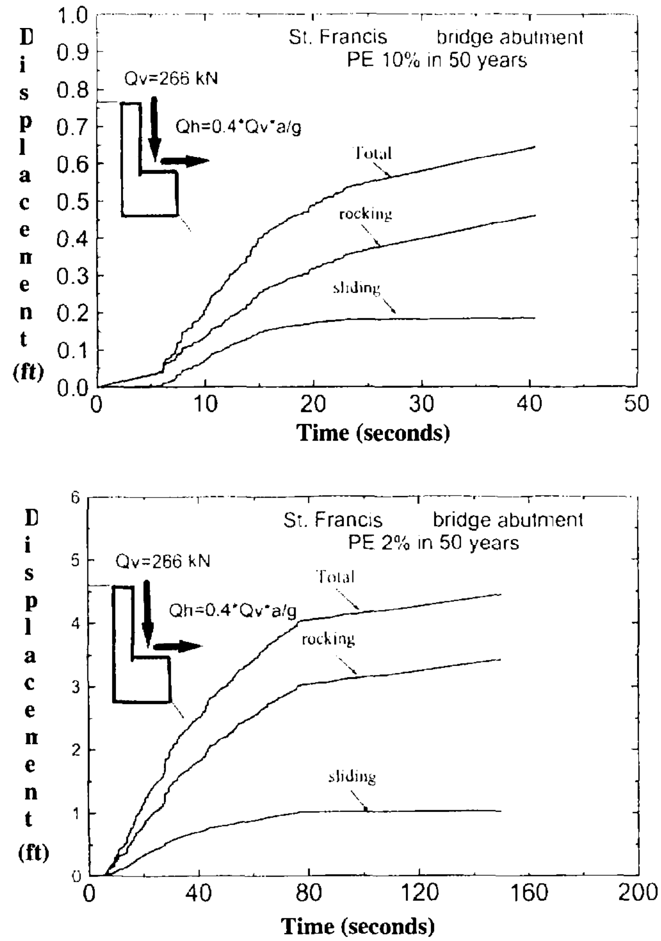

Figure 3 – Time histories of displacement at the abutments (St. Francis River Bridge).

SEISMIC SLOPE STABILITY

For the St. Francis Bridge site, slope stability analyses were completed for seven cross-sections. Each cross-section was analyzed for both low and high ground-water conditions under static analysis and under two pseudo-static earthquake accelerations. Cross-section locations are shown on Figure 4, St. Francis Bridge Site Topography. The cross-section data was then entered into the slope stability program *PCSTABL5* using the pre and post processor *STEDwin*. The slopes were analyzed under static and dynamic conditions using the Modified Bishop Method.

A summary of the St. Francis site analyses is included in Table 2. In general, the site slopes appear to be stable under static conditions, with both low and high ground-water tables, with factors of safety ranging from 1.93 to 3.96. When subjected to an earthquake with a 10% exceedance probability in 50 years (PE) (which would generate horizontal accelerations of 21%g), slopes continue to show stability, with factors of safety dropping to a range of 1.23 to 2.20. When subjected to an earthquake with a 2% PE (38%g), factors of safety less than or approximately equal to one are calculated for section F-F' under low water conditions and all sections under high water conditions. Expected failure planes pass through both the roadway and bridge piers. An example analysis output for cross-section C-C' is shown on Figure 5.

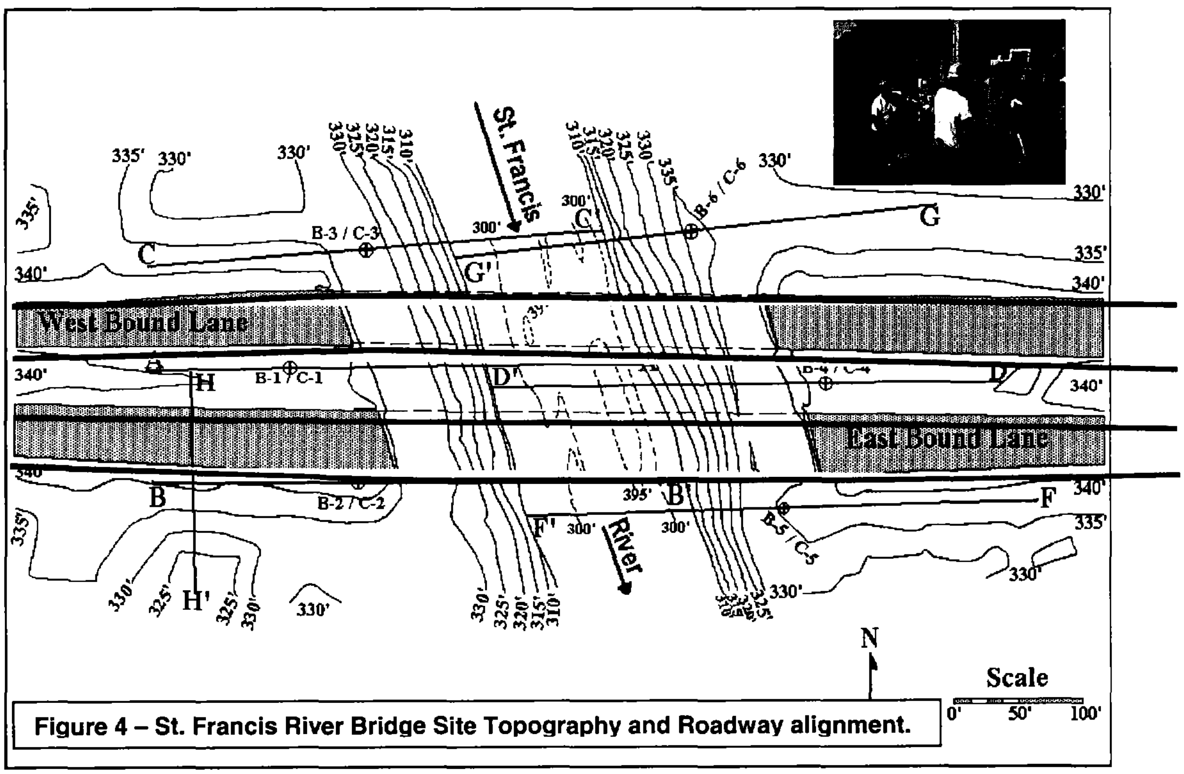

Figure 4 – St. Francis River Bridge Site Topography and Roadway alignment.

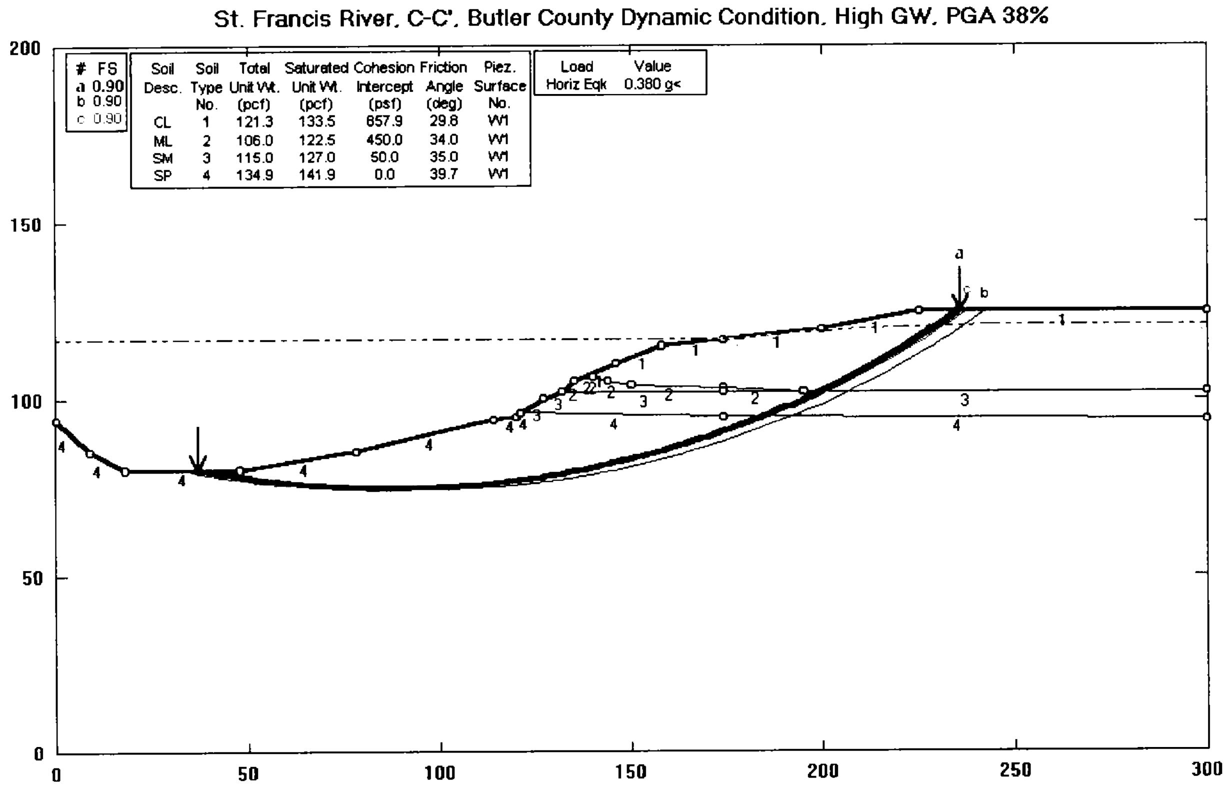

Figure 5 – Seismic Slope Stability Analysis – example.

Table 2 - Factor of Safety for Select Cross-sections

Cross-Section	A - A'	C – C'	F – F'	G – G'
Static				
Low GW	2.63	2.88	1.93	3.96
High GW	3.06	3.48	2.02	2.67
Dynamic* **(Low GW)**				
10% PE, PGA 21%	1.46	1.52	1.23	2.20
2% PE, PGA 38%	1.03	1.06	0.90 §ψ	1.56
Dynamic* **(High GW)**				
10% PE, PGA 21%	1.28	1.41	1.01	1.41
2% PE, PGA 38%	0.83 §ψ	0.90 §ψ	0.66 §ψ	0.99 §

These results indicate that slopes at the St. Francis Bridge site are expected to be stable under small earthquake shaking (10% PE), and unstable at higher levels of shaking (2% PE), regardless of the ground-water level.

A similar set of analyzes were performed for the Wahite Ditch site. The anticipated behavior is similar to that described for the St. Francis Bridge site. The site slopes are expected to be stable under static conditions (F.S. range from 3.48 to 7.76) and under 10% PE (27%g) loads (F.S. range from 1.28 to 2.60) for both low and high ground-water conditions. Under 2% PE (39%g) loads, factors of safety are greater than one for all analyzed sections for low ground-water conditions. Under high water conditions' factors of safety are less than or approximately equal to one for sections A-A', C-C', D-D', E-E', and F-F' (not shown).

Both sites are expected to be stable under small earthquake conditions. The results at the St. Francis Bridge site indicate slightly higher sensitivity to ambient ground-water levels (which are affected by water levels in the river) than at Wahite Ditch. Stability analysis under large earthquake conditions indicates instability at the St. Francis Bridge site, regardless of the ground-water level and instability at Wahite Ditch when ground-water levels are high.

ANALYSIS OF ST. FRANCIS RIVER BRIDGE (1978) SUPERSTRUCTURE

For this preliminary analysis of the older St. Francis River Bridge, soil-structure interaction was not included. All columns were fixed at the centroid of pile caps and, abutments and their supporting soil strata were assumed rigid. The seismic acceleration time history (maximum acceleration: 0.1g) at the elevation of one pile cap of Bent 2 was used as longitudinal input at all boundaries of the bridge model. The maximum responses from such time history analyses were compared with those due to the design earthquake specified in AASHTO.

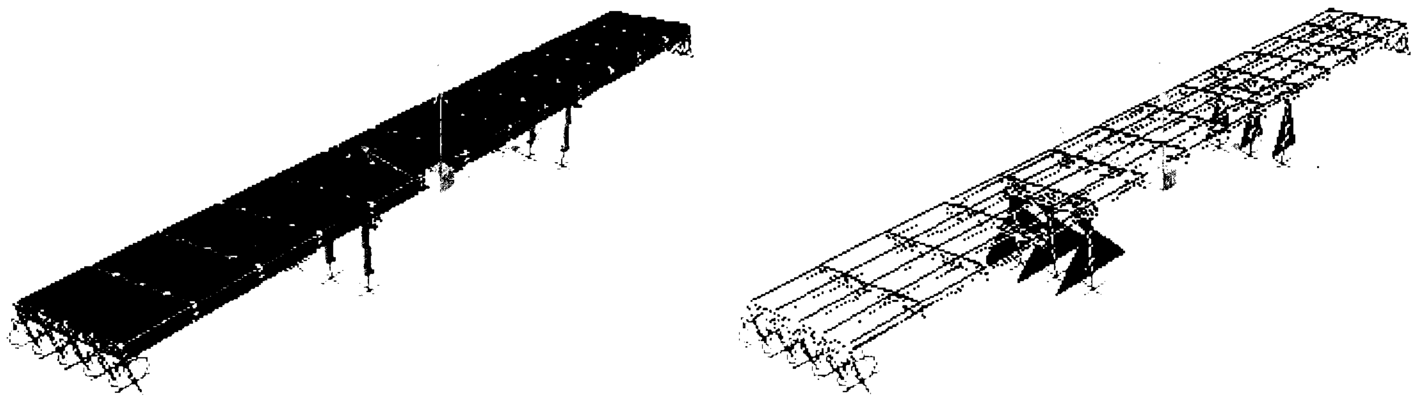

Figure 6 – St. Francis Bridge Structural Dynamic Analysis

Under the site-specific seismic load, the bridge deck experiences about 0.14 in movement, which is less than the existing joint width (2.5 in or 1.875 in). Pounding will not occur in this case. It is observed that the bridge mainly moves in the longitudinal (traffic) direction.

Although there are no seismic forces in the transverse direction, columns are subject to bending in the transverse plane due to the skew effect of the bridge. The maximum moment of column in the transverse plane is only 36% of that in the longitudinal plane. Since its longitudinal movement is restrained at the top of cap beam by fixed bearings, the columns at this bent carry the most seismic load from the superstructure and are thus subject to a significantly larger moment than that of Bent 2. It is also interesting to note that several girders are subject to bending due to the skew effect. For the same reason, those girders carry little axial forces.

Figure 6 shows the computer model performed using SAP 2000 for the structural dynamic analysis and the moments develop in the columns at both bridge bents.

The maximum ground acceleration is about 0.288g according to the AASHTO spectrum, which is significantly higher than the maximum acceleration of the site-specific time history (0.1g). Therefore, the displacement and force of the bridge are much higher under a design earthquake specified in AASHTO than the site-specific earthquake used in analysis. Pounding will not occur under the AASHTO design earthquake.

FLOODING POTENTIAL

Evaluation of the effects of flooding due to failure of levees was based on a series of topographic maps covering the entire study section of US 60. This evaluation was field checked by visual observation of the elevation of the roadway compared to surrounding land. Some of the maps were as old as 1962 vintage without photo-revision, so the estimate of the limits of potential flooding should be considered tentative. Furthermore, the roadway elevation was shown only to 5-foot accuracy, and slight elevations or depressions in the roadway could significantly change the degree of anticipated flooding. In general, the following hydrologic features are expected to be affected during an earthquake, presented in order from west to east: Blue Spring Slough, St. Francis River, Mingo, Cypress Creek Lateral, and Prairie Creek Ditches, Unnamed Creek 1 mile West of Essex, Bess Slough, Six Unnamed Ditches Between Bess Slough and the Castor River, Wahite Ditch. The remaining sections of US 60 to the east of the Wahite Ditch appear to be elevated and are not anticipated to experience flooding due to levee failure.

CLOSING

Overall, the seismic assessment of the critical structures along US60 in the state of Missouri performed satisfactorily for an earthquake event with a PE 10% in 50 years. However, for an event PE 2% in 50 years the structures evaluated, bridge foundations, abutments and embankment fills will be significantly damaged to a level that may render the access routes unusable.

The dynamic structural analysis is preliminary in nature and it does not include the effect of local soil conditions or soil-structure interaction. The bridge structure selected was considered the weakest link or oldest (built in 1978) among the bridges over these crossings, therefore, it was the initial focus of the study. Future analysis will be considering the more modern bridges, which include an integral bridge deck and abutment.

REFERENCES

Anderson, N. et al. (2000), "Earthquake Hazard Assessment Along Designated Emergency Vehicle Access Routes," Report No. R198-043, Missouri Department of Transportation, Jefferson City, Missouri, 65102.

Choudhry, M.S., 1999, Seismic Analysis of Pile Supported Rigid-Bridge Abutments Using Material Geometric Non-linearity: Ph.D. Dissertation, University of Missouri-Rolla.

Monobo, N. and H. Matsuo, (1929), "On the Determination of Earth Pressure During Earthquakes, Proc. World Earthquake Engineering Congress, Tokyo.

NISEE (National Information Service for Earthquake Engineering), University of California, Barkeley. Http://www.eerc.berkeley.edu/

Novak, M., 1974, Dynamics Stiffness and Damping of Piles: Canadian Geotechnical; Journal, Vol 11, 574-598.

Novak, M. and El-Sharnouby, B., 1983, Stiffness and Damping Constants of Single Piles: Journal Geotechnical Engineering, Div., Am. Soc. Civ. Eng. 109(GT-7), 961-974.

Seed, H.B. and Idriss, I.M., 1971, Simplified Procedure for Evaluating Soil Liquefaction Potential: Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 97, No. SM9, pp. 1249-1273.

Youd, T.L. and Idriss, I.M. (editors.), 1997, Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils: National Center for Earthquake Engineering Research, Technical Report No. NCEER-97-0022, December 31, 1997.

Wen, Y.K. and Wu, C.L., 2000, Generation of Ground Motions for Mid-America Cities: (MAE report currently under review).

Wu, Y., 1999, Displacement-Based Analysis and Design of Rigid Retaining Walls During Earthquake: Ph.D. Dissertation, University of Missouri-Rolla.