Research Report
KTC-00-01/SPR96-173

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RANKING AND ASSESSMENT OF SEISMIC STABILITY OF HIGHWAY EMBANKMENTS IN KENTUCKY

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Research Report
KTC-00-1

Ranking and Assessment of Seismic Stability of Highway Embankments in Kentucky (KYSPR 96-173)

by

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In cooperation with

Kentucky Transportation Cabinet
Commonwealth of Kentucky

And

Federal Highway Administration
U.S. Department of Transportation

July 2000

Mr. Jose M. Sepulveda **July 2000**
Division Administrator
Federal Highway Administration
330 West Broadway
Frankfort, Kentucky 40602

Subject:
- Implementation Statement for Final Report entitled "Ranking and Assessment of Seismic Stability of Highway Embankments in Kentucky"
- Study number: KYSPR 96-173
- Study title: "Seismic Rating and Evaluation of Highway Structures"

Dear Mr. Sepulveda:

The objectives of this study were to (a) evaluate the seismic stability of over 400 highway embankments along priority routes in western Kentucky and rank them in terms of their risk of failure, (b) provide a preliminary assessment of the seismic stability of the approach embankment of the U.S. 51 bridge across the Ohio River near Wickliffe, Kentucky, and (c) provide a stability assessment of the approach embankments for the U.S. 41 twin spans across the Ohio River near Henderson, Kentucky. These objectives have been completed in accordance with the state of the art.

The ranking of highway embankments for western Kentucky designated 6 embankments that may be at serious risk of failure for the 50 year event, and 145 embankments that may be at serious risk of failure for the 500 year event. The ranking also provided a rating of all embankments in terms of potential displacements or psuedo-static factors of safety for the 50 and 500 year events.

The preliminary study of the seismic stability of the U.S. 51 approach embankment near Wickliffe indicated a low to moderate risk of localized pockets of liquefied soils causing some embankment damage for a 50 year event, and a likelihood of liquefaction and possible loss of portions of the embankment for a 500 year event.

The stability assessment of the U.S. 41 approach embankments near Henderson indicated low risk of liquefaction and a factor of safety of approximately 1 for the embankments in the event of either a 50 or 500 year event. The displacements that would be expected in the event of a 50 or 500 year earthquake were estimated to be low enough that damage to spans would not occur and road surface damage would be minor to none. It was suggested that some minor damage to road surfaces could be incurred, but that it would be minor and within the scope of repair by normal maintenance personnel.

Sincerely,

J. M. Yowell, P.E.
State Highway Engineer

Cc: John Carr

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
KTC 00-1		

4. Title and Subtitle	5. Report Date
Ranking and Assessment of Seismic Stability of Highway Embankments in Kentucky (KYSPR 96-173)	July 2000
	6. Performing Organization Code
7. Author(s) K.G. Sutterer, I.E. Harik, D. Allen, and R. Street	8. Performing Organization Report No.
9. Performing Organization Name and Address Kentucky Transportation Center College of Engineering University of Kentucky Lexington, Kentucky 40506-0281	10. Work Unit No. (TRAIS)
	11. Contract or Grant No. KYSPR 96-173
12. Sponsoring Agency Name and Address Kentucky Transportation Cabinet State Office Building Frankfort, Kentucky 40622	13. Type of Report and Period Covered Final
	14. Sponsoring Agency Code

15. Supplementary Notes

Prepared in cooperation with the Kentucky Transportation Cabinet and the U.S. Department of Transportation, Federal Highway Administration

16. Abstract

This report presents the findings of three independent studies under the same grant. These were (a) assess and rank highway embankments along priority routes in western Kentucky according to seismic stability, (b) assess the seismic stability of the approach embankment for the U.S. 51 Ohio River crossing near Wickliffe, Kentucky, and (c) evaluate the seismic stability of the U.S. 41 Ohio River twin spans north of Henderson, Kentucky. Seismic stability of bridge foundations was not a part of this study.

The highway ranking assessed over 400 embankments and delineated each as having high, moderate, or little risk of significant failure for both the 50 and 500 year events. For the 50 year event, 6 were designated as high risk, while 145 were designated as high risk for the 500 year event. The report recommends evaluation of all high risk embankments and any moderate risk embankments along particularly critical sections of roadway.

The US 51 bridge embankment assessment indicated a high risk of embankment failure for the 500 year event, and a moderate risk of some deformation requiring repair for the 50 year event. The findings suggested the need for an emergency repair plan in the event of either earthquake, and suggested a more detailed evaluation if a higher confidence about the risk of failure was required.

The US 41 bridge embankment evaluation indicated little to no risk of major liquefaction for the 50 and 500 year events. A factor of safety of about 1.0 was estimated for seismic slope stability of the embankment at the bridge abutments, but a detailed stratigraphic section was not possible due to limited project budget. There thus remains some uncertainty about the overall seismic slope stability. The factor of safety is not likely to be significantly less than 1.0. This is acceptable for seismic loading, as a factor of safety of 1.0 implies some deformation of the embankment may occur, but the extent of damage should be repairable on relatively short notice.

17. Key Words	18. Distribution Statement
seismic, slope stability, embankment, US41, US51, western Kentucky, liquefaction	Unlimited with approval of Kentucky Transportation Cabinet

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	97	

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EXECUTIVE SUMMARY

Research Objectives

The objectives of this study were to (a) evaluate the seismic stability of over 400 highway embankments along priority routes in Western Kentucky and rank them in terms of their risk of failure, (b) provide a preliminary assessment of the seismic stability of the approach embankment of the U.S. 51 bridge across the Ohio River near Wickliffe, Kentucky, and (c) provide a stability assessment of the approach embankments for the U.S. 41 twin spans across the Ohio River near Henderson, Kentucky. Completion of objective (a) required development of some new innovative methods for assessing a multiple of embankments simultaneously. This was done successfully. Objectives (b) and (c) have been completed in accordance with the state of the art as described herein.

Kentucky Embankment Stability Rating (KESR)

A 1988 report by Kentucky Transportation Center recommended inspection and ranking of critical embankments and slopes along priority routes in 15 counties in Western Kentucky. The project reported herein developed a ranking model to complete this task, titled Kentucky Embankment Stability Rating (KESR). This rating system ranks embankments in one of classes A, B, C, or Z. Class A embankments are considered most at risk of failure during a seismic event, while class C embankments were considered to not be at significant risk. Class Z embankments are not ranked due to unusual embankment geometry or insufficient data.

The study inspected and evaluated 408 slopes along those priority routes for both the 50 and 500 year earthquakes using KESR. Of these, 17% were in class Z. The study found 1% in class A and 60% in class C for the 50 year event, and 35% in class with 27% in class C for the 500 year event. The counties with the most class A embankments were Ballard, Fulton, Graves, and Marshall. Further evaluation of class A embankments, along with evaluation of the more critical class B embankments for critical bridges, is recommended. The scope of the evaluation can be determined by a qualified geotechnical engineer. The estimated factor of safety or displacement for each embankment is not an accurate assessment of predicted behavior, but merely a tool that permits ranking of the embankments to permit assessment of the more critical embankments first.

U.S. 51 Ohio River Approach Embankment, Wickliffe

This study also included preliminary assessment of the seismic stability of approach embankments for the U.S. 51 Ohio River crossing just north of Wickliffe, Kentucky. The study found the risk of widespread liquefaction and thus extensive embankment failure is high for the 500 year earthquake event, but there is only marginal risk of limited liquefaction and localized embankment displacement for the 50 year event. Delineation of specific zones to remediate to reduce the risk of liquefaction for the 50 year event would be very difficult for the approximately 3 miles of embankment from Wickliffe to the Ohio River bridge. Deformations due to localized liquefaction during the design 50 year earthquake likely would not be excessive, so emergency earth moving equipment could perform repairs quickly to keep at least one lane of traffic open.

It is recommended a plan be in place for repair of localized embankment failures in the event of damaging movement. A 500 year event is likely to cause widespread liquefaction and potential loss of the embankment. Although an analysis of liquefaction around the bridge foundations was not performed, it is likely a similar risk exists in those soils. Thus, extensive repair and mitigation would be necessary to design for a 500 year event.

U.S. 41 Twin Spans Ohio River Approach Embankments, Henderson

The study also performed an assessment of the seismic stability of the approach embankments of the twin spans of U.S. 41 across the Ohio River north of Henderson, Kentucky. A more detailed analysis including a drilling and sampling program, geophysical testing, one dimensional total stress analysis modeling, and careful examination of the site conditions was conducted for these spans. The extensive testing necessary to complete a detailed stratigraphic section along the four approach embankments was not within the work scope, but it was possible to develop a reasonable assessment of likely behavior for the slopes. The study found liquefaction was of low to moderate likelihood for both the 50 or 500 year events. The north abutment embankments are likely at greater risk, based on the study, but the estimated the psuedo-static slope stability of the embankments to be within the normal standard of practice.

ACKNOWLEDGEMENTS

The financial support for this project was provided by the Kentucky Transportation Cabinet and the Federal Highway Administration. The authors would like to acknowledge the cooperation, suggestions, and advice of the Study Advisory Committee: Donald Herd (committee chairperson), Glenn Givan, Ray Greer, David Moses, Ted Noe, N.B. Shah, and David Steele.

Many thanks are extended to the following undergraduate and graduate students in the Department of Civil Engineering who worked many hours in the completion of this project: Andrew Dodds, Bartolomej Grolewski, Chris Banbury, and Keith Hughes.

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1. INTRODUCTION

In the 1980's, Kentucky undertook a number of initiatives to consider earthquake hazards and their mitigation with respect to the state's infrastructure. Part of this effort included the transportation infrastructure. An assessment of seismic performance and risk to the highway infrastructure resulted in the 1988 report (Allen *et al.*, 1988) entitled "Earthquake Hazard Mitigation of Transportation Facilities." That report recommended assignment of priority routes for movement of goods and services in western Kentucky, where widespread damage could result from a major seismic event in the New Madrid Seismic Zone (NMSZ). After selection in that study, each priority route was visually surveyed to catalog natural and man-made features that could potentially hamper rescue and relief efforts in the event of a major earthquake. The features cataloged included bridges, dams, pipelines (natural gas and petroleum), power lines, high fills of more than 15 to 20 feet in height, cut slopes, signs, buildings, faults, storage tanks, trees, and active or abandoned mines.

The report from that study recommended more detailed assessment of the fills catalogued. As a consequence, the Kentucky Transportation Cabinet requested a study of high fills and approach embankments along most of the priority routes in western Kentucky. Specifically, embankments along all or portions of the following highways in western Kentucky were evaluated: US 45, KY 58, US 60, US 62, US 68, KY 80, KY 91, KY 94, KY 121, KY 166, and US 641. These designated priority routes are unlimited access highways and secondary roads, as compared to limited access parkways or interstates. The counties included in the study are Ballard, Caldwell, Christian, Carlisle, Calloway, Fulton, Graves, Hickman, Livingston, Logan, Lyon, Marshall, McCracken, Todd, Triggs, and Warren.

Site-specific embankment stability studies were also requested for the approach embankments for the U.S. 51, Mississippi River Bridge in Wickliffe, Kentucky, and the U.S. 41 Ohio River Twin Spans in Henderson, Kentucky. The findings of these studies are also reported herein.

The seismic risk of the western Kentucky region is well documented. A recent summary is provided in a report to the Kentucky Transportation Cabinet by Street *et al.* (1996). They described seismic risk associated with four distinct seismic zones in the Kentucky region, along with risk associated with background seismicity not connected to a specific seismic zone. The four seismic zones are depicted in Figure 1.1. The body wave magnitude, $m_{b,Lg}$, for 50 and 500 year events for each zone and the local event due to background seismicity were recommended by Street *et al.* (1996) as indicated in Table 1.1. Street *et al.* (1996) then used an earthquake model to prepare synthetic bedrock earthquake records for each county in Kentucky. The peak ground acceleration varies from county to county due to attenuation of the motion as it travels from the source to the county seat of each county. The peak ground acceleration for each of the sixteen counties referenced above for both the 50 and 500 year events, along with the number of embankments assessed in each county, is shown in Table 1.2. The ground motions were prepared in sets of three for each county, representing vertical motion, horizontal motion parallel to the direction of wave travel, and horizontal motion perpendicular to the direction of wave travel. Details of the model, the assumptions made in the preparation of time histories, and the actual time histories may be found in the report by Street *et al.* (1996).

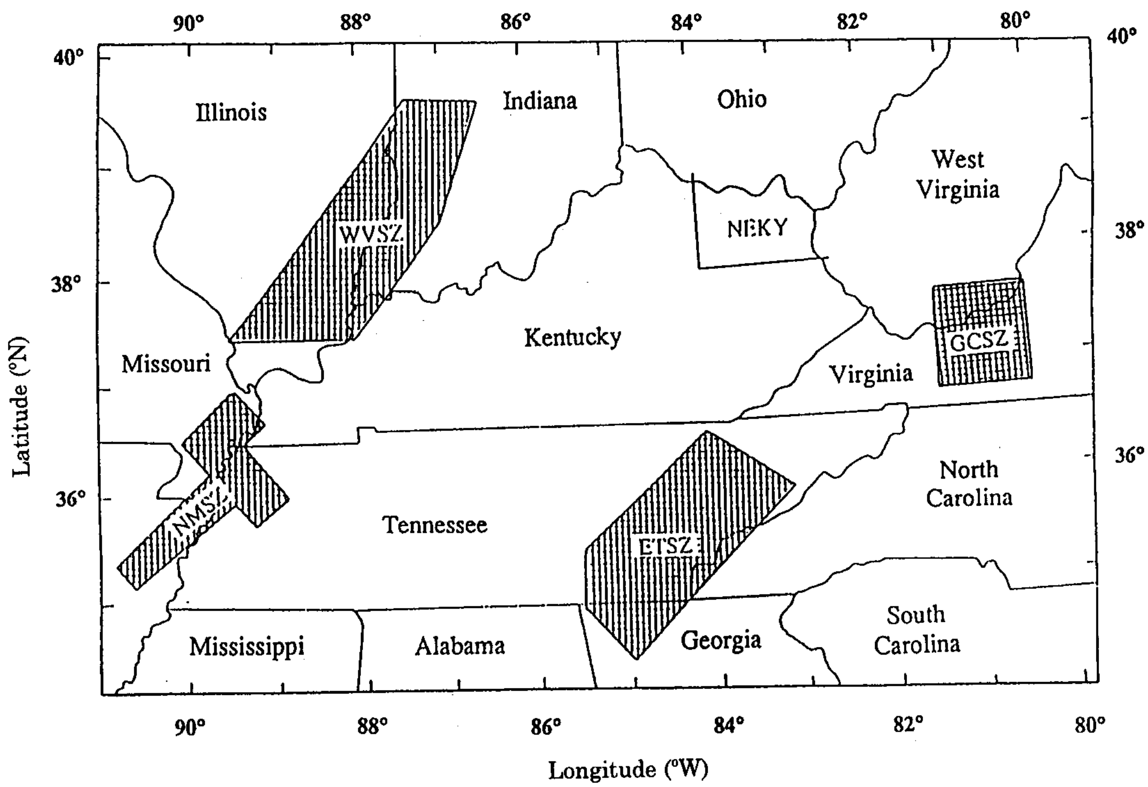


Figure 1.1 Regions of New Madrid Seismic Zone (NMSZ), Wabash Valley Seismic Zone (WVSZ), Gile County Seismic Zone (GCSZ), and Eastern Tennessee Seismic Zone (ETSZ) from Street *et al.* (1996).

Table 1.1. Seismic Zones and Body Wave Magnitude, $m_{b,Lg}$, for 50 and 500-Year Events

Seismic Zone	*50-year event*	*500-year event*
New Madrid	6.3	7.0
Wabash Valley	5.5	6.3
Eastern Tennessee	4.7	6.2
Giles County, Virginia	4.3	6.2
Local event, depending on county	4.5 - 5.3	4.5 – 5.5

Table 1.2. Peak Ground Acceleration (PGA) recommended by Street *et al.* (1996) and Embankment Count for each County.

County Name	*County Seat*	*50 year PGA (percent of gravity)*	*500 year PGA (percent of gravity)*	*No. of embankments*
Ballard	Wickliffe	26.6	63.2	43
Caldwell	Princeton	8.8	17.8	4
Christian	Hopkinsville	9.4	14.4	25
Carlisle	Bardwell	26.2	60.8	19
Calloway	Murray	8.3	27.1	38
Fulton	Hickman	26.8	58.7	12
Graves	Mayfield	14.5	41.3	74
Hickman	Clinton	30.8	60.5	7
Livingston	Smithland	12.5	25.4	7
Logan	Russellville	9.1	9.7	22
Lyon	Eddyville	8.6	20.1	22
Marshall	Benton	14.1	27.2	73
McCracken	Paducah	13.4	30.9	31
Todd	Elkton	9.1	11.1	14
Trigg	Cadiz	8.9	17.1	18
Warren	Bowling Green	9.0	9.0	2

2. RANKING OF WESTERN KENTUCKY EMBANKMENTS

2.1 Introduction

The method used for ranking the embankments and slopes in the sixteen counties is herein titled Kentucky Embankment and Slope Stability Ranking (KESR). This study surveyed all embankments of more than five feet in height along the priority routes referenced in Chapter 1, and ranked the embankments with respect to their predicted relative seismic stability. KESR bases the priority ranking of the embankment on estimated mechanical stability only. Factors to account for relative importance, uncertainty in loading and stability, and other issues were excluded, as requested by Kentucky Transportation Cabinet. As described below, the ranking required on-site inspections, approximate measurement of slope geometry, site geology as reported in the literature, correlations between drilling data and similar geology in western Kentucky, and a model that provided a simplification of detailed, state of the art seismic stability analyses typically followed for specific sites.

KESR assumes one of three types of embankment behavior during a major seismic event: (A) loss of the embankment, (B) significant movement without loss of the embankment, and (C) no significant movement. Ranking of embankments exhibiting no significant movement was based on the estimated factor of safety. Embankments predicted to exhibit significant movement were ranked using estimated embankment deformation. Loss of the embankment was only assumed in the event of high liquefaction potential for the specified seismic loading, or in the event that the predicted displacement exceeds 10 centimeters. All embankments under category (A) were considered high priority embankments and were ranked equally critical.

KESR requires estimation of representative ground motion, soil mechanics properties, geometry of the embankment and foundation, and a tool for estimating mechanical performance of the embankments. Following is a summary of the methods used to do this. The results are intended to provide relative ranking of the embankments, not to predict actual expected performance. Since the model used to predict mechanical behavior defines the soil properties, site geometry, and ground motion parameters that are required, the model of mechanical behavior will be addressed first.

2.2 Limit Equilibrium Slope Stability

The potential for slope movement to occur during an earthquake is assessed using a two dimensional limit equilibrium stability analysis developed specifically for KESR. The stability analysis is summarized by Sutterer *et al.* (1998). It considers critical circular and wedge shaped failures for each of the slopes using a numerical formulation of the static equilibrium for the conditions shown in Figure 2.1. As described in Sutterer *et al.* (1998), 98 pseudo static analyses of hypothetical homogeneous slopes showed that seismically loaded embankments with a uniform foundation soil, and slope inclinations flatter than 1 horizontal to 1 vertical and steeper than about 4 horizontal to 1 vertical, generally failed as a base failure. Steeper slopes are generally subject to a toe circle failure in the embankment alone, a condition modeled by Figure 2.1a. Most highway embankments fall within the range dominated by base failures, although occasional steep embankments may be more likely to fail as a sliding wedge, so these two modes of failure were simulated in this analysis.

Following the work of Janbu (1954), but incorporating the possibility of different undrained strengths of the embankment and foundation and a horizontal K_h acceleration coefficient as shown in Figure 2.1, Sutterer *et al.* (1998) found the factor of safety, FS, can be computed as:

$$FS = \left[\frac{R_1 - R_2}{D_1 + K_h \cdot D_2} \right] \cdot \frac{S_1}{g_1 \cdot H} \tag{2-1a}$$

$$R_1 = 40 \cdot \sqrt{\frac{d}{r}} \cdot r \cdot (d + 12 \cdot r) \cdot (2 - \boldsymbol{l}) \tag{2-1b}$$

$$R_2 = \sqrt{\frac{1+d}{r}} \cdot (9 \cdot (1+d)^2 + 40 \cdot (1+d) \cdot r + 480 \cdot r^2) \cdot \boldsymbol{l} \tag{2-1c}$$

$$D_1 = 40 \cdot \sqrt{2} \cdot \left(1 + b^2 + 3d + 3d^2 - 3r - 6dr - 3bx + 3x^2\right) \tag{2-1d}$$

$$\begin{aligned} D_2 = {} & 40 \cdot \sqrt{2} \cdot (b + 3bd - 2 \cdot (-d \cdot (d - 2r))^{3/2} - 2 \cdot ((-1-d) \cdot (1 + d - 2r))^{3/2} \\ & - 3br - 3x - 6dx + 6rx \end{aligned} \tag{2-1e}$$

Most of the parameters are defined in Figure 2.1. S_1 is the undrained shear strength of the foundation soil beneath the embankment, $\boldsymbol{l}$ is the ratio S_2/S_1, where S_2 is the undrained shear strength in the embankment. The parameter γ is the density of the soil in both layers. For the values of x and r that result in the lowest factor of safety, designated x_c and r_c, the term in brackets in equation (2-1a) is the stability number for the designated slope. Equation (2-1b) can be rearranged for FS =1, giving the critical K_{hf} causing failure:

$$K_{hf} = \frac{(R_1 - R_2) \cdot \dfrac{S_1}{g_1 \cdot H} - D_1}{D_2} \tag{2-1f}$$

Although a base failure predominates for the slope geometry typically encountered in highway embankments, a wedge failure extending upward from the toe of the embankment may be most critical for steeper slopes. This failure geometry is depicted in Figure 2.1b. For this condition, the factor of safety is indicated by:

$$FS = \frac{2 \cdot (1 + a^2)}{(a - b) \cdot (1 + a \cdot K_h)} \cdot \frac{S}{g \cdot H} \tag{2-2a}$$

So that for a FS = 1, the critical K_{hf} causing failure is found to be

$$K_{hf} = \frac{1}{a} \cdot \left[\frac{2 \cdot (1 + a^2)}{(a - b)} \cdot \frac{S}{g \cdot H} - 1 \right] \tag{2-2b}$$

With a being the variable that is optimized for the minimum K_{hf} and S selected as the estimated shear strength along the base of the failure wedge. Given equations (2-1) and (2-2) for estimating the yield acceleration, the minimum yield acceleration of the two is selected as being most likely for each specific embankment.

Selection of K_h for equations (2-1a) and (2-2a) is subject to judgement. The horizontal earthquake acceleration in slope stability analyses often ranges from 1/2 to 1 times the peak ground acceleration (PGA) predicted for the site. Although any acceleration exceeding a slope's yield acceleration should cause some plastic deformation, the peak acceleration is often a single "spike" of motion of very brief duration and thus often causes little if any significant movement. A reasonable value of K_h would be more on the order of 2/3 of the predicted PGA, which was the value selected for the limit equilibrium component of KESR.

Use of equation (2-1) in a spreadsheet with an optimization function provided reliable estimates of the above parameters over the designated slope inclinations. Specifically, the Solver® function in Microsoft Excel 97® was used to find r_c and x_c to minimize the factor of safety. Microsoft Excel Solver® uses the Generalized Reduced Gradient (GRG2) nonlinear optimization code developed by Leon Lasdon, University of Texas at Austin, and Allan Waren, Cleveland State University (Lasdon *et al.*, 1978, Lasdon and Waren, 1989, Waren *et al.*, 1987). Application of the optimization function to the approximately 400 slopes in the database described later required about one full working day of computations using a Pentium class desktop computer. To validate the analysis after the optimization, ten specific slopes were randomly selected from the data base and subjected to modified Bishop stability analyses using a popular slope stability program for comparing the computed stability with the optimized stability from the ranking model. For all ten sites, the factor of safety using the optimization model matched the modified Bishop factor of safety. Given this validation of the model for limit equilibrium behavior, the optimization model factor of safety was used to rank all slopes with a computed factor of safety greater than one.

2.3 Slope Displacement Estimate

Using K_h equal to 2/3 of the peak ground acceleration in the above limit equilibrium analysis accounts for embankments in which the seismic acceleration never exceeds the yield acceleration. That K_h value also accounts for those embankments where the seismic acceleration very briefly exceeds the yield acceleration and thus results in little to no movement. Since the selected K_h represents one or several brief loads during the seismic event, rather than a constant load, the question remains for those slopes with a factor of safety less than one as to how far the mass actually moved while the ground motion was taking place. The estimate of slope displacement for those embankments where FS < 1.0 was based on the results of an in-depth study of synthetic central U.S. time histories using a sliding block analysis (Newmark, 1965).

The sliding block analysis has been in use for over 30 years for assessing potential deformation of a slope or embankment due to a specific ground motion. The method assumes a slope can be simulated as a wedge resting on an inclined plane. The acceleration causing the slope to yield, A_y, is determined using a pseudo static analysis like that developed above with the FS set equal to 1.0. For a specific location, seismologists can provide an estimate of the peak ground acceleration expected, A_{max}, for an event of a specific return period. For slope movement to occur, A_{max} must exceed A_y. As the ratio of A_y/A_{max} decreases, deformation increases. The sliding block analysis is simply double integration of all portions of an earthquake time history that exceed A_y.

For his work, Dodds (1997) modified the desktop PC based computer program DISP (Chugh, 1980) to examine the relation between A_y/A_{max} and sliding block deformations using 128 different synthetic time histories specifically developed to model central U.S. bedrock motions (Street *et al.,* 1996 and Wang and Street, 1997). Using the form of equation developed by Ambraseys and Menu (1988), it was assumed

$$\log_{10}(u) = \alpha + \beta_1 \log_{10}\left(1 - \frac{A_y}{A_{\max}}\right) + \beta_2 \log_{10}\left(\frac{A_y}{A_{\max}}\right) \qquad (2\text{-}3)$$

where u is the predicted displacement, in centimeters, of the embankment. For the 128 synthetic time histories considered, Dodds found the "bedrock" coefficients from equation (2-3) provided in Table 2.1 gave the best fit to the predicted displacements.

Dodds' results are based on bedrock ground motion, but the soil overburden in portions of western Kentucky is often more than 30 feet thick, and is over 100's of feet thick near the Mississippi River. Local overburden should change the ground motion and thus the displacement behavior. To investigate this, boring data and shear wave velocity profiles for nine sites throughout western Kentucky were acquired from University of Kentucky records (Street *et al.,* 1997). The sites were selected to represent the most common combinations of alluvium, continental deposits, and bedrock depth in the region. This subsurface data was compiled for use in the computer program for one dimensional wave propagation, SHAKE91 (Idriss and Sun, 1992), to model the propagation of shear waves fom the bedrock to the ground surface at each of those sites. Using several of the referenced synthetic bedrock time histories (Street *et al.,* 1996 and Wang and Street, 1997) for each of the nine sites as the bedrock motions, 38 additional ground surface motions were produced. The resulting ground surface ("soil") time histories were then used in the modified DISP program prepared by Dodds (1997), with the resulting displacements being fit to equation (2-3) to predict embankment behavior for sites in western Kentucky with deeper overburden. The resulting coefficients from equation (2-3) for "soil" sites in western Kentucky are also shown in Table 2.1.

Since the parameters α, β_1, and β_2 vary with magnitude, it is possible to use an equation to predict the variation of these parameters for magnitude 5 to 7 events on both bedrock and soil sites. The following linear relations were recommended to predict the parameters, α, β_1, and β_2 for bedrock and soil sites in western Kentucky:

$$(\alpha)_{bedrock} = 0.735 \cdot M_{b,Lg} - 4.41 \qquad (2\text{-}4a)$$

$$(\alpha)_{soil} = 1.025 \cdot M_{b,Lg} - 6.292 \qquad (2\text{-}4b)$$

$$(b_1)_{bedrock} = 0.35 \cdot M_{b,Lg} + 1.94 \quad (2\text{-}5a)$$

$$(b_1)_{soil} = 0.35 \cdot M_{b,Lg} + 1.94 \quad (2\text{-}5b)$$

$$(b_2)_{bedrock} = 0.21 - 0.15 \cdot M_{b,Lg} \quad (2\text{-}6a)$$

$$(b_2)_{soil} = -0.794 - 0.056 \cdot M_{b,Lg} \quad (2\text{-}6b)$$

Equation (2-3) is plotted in Figure 2.2 using equations (2-4) through (2-6) to predict parameters for a magnitude 7 event for both bedrock and soil sites. Also shown in Figure 2.2 is the relation developed by Ambraseys and Menu (1988). It is noteworthy that the results for the "soil" sites for central U.S. events closely approximates the behavior predicted by Ambraseys and Menu for predominantly western U.S. sites. Further examination of the response spectra for the bedrock time histories used by Dodds indicated a significant high frequency (10-30 hz) component in which the PGA of the time history occurred. This is not typical for most measured bedrock motion, in which the PGA typically occurs at substantially lower frequencies. It was concluded the high frequency component was an outcome of the synthetic time history model used to generate the motions, and might not be representative of true peak acceleration behavior on the sites. While this may not be significant to some dynamic structural analyses, the Newmark method normalizes findings to the PGA, so the use of these synthetic time histories in which PGA occurs at unusually high frequencies is not likely appropriate. For this reason, this study used the relations shown in equations (2-4b), (2-5b), and (2-6b) based on Newmark response to ground surface time histories in which the high frequency component had been filtered out by the SHAKE91 analyses.

To complete the estimation of potential displacement, it is necessary to determine the acceleration causing yield for each embankment. This is achieved by setting the factor of safety equal to 1.0 in equations (2-1a) or (2-2a), whichever is critical, and optimizing for r_c and x_c as demonstrated in equations (2-1f) and (2-2b). It is then assumed that $A_y = K_{hf}$ from equations (2-1f) and (2-2b). For values of A_y/A_{max} less than one, some displacement would be expected to occur during the intervals when the ground acceleration exceeds the yield acceleration. The magnitude of that deformation may be quite small, but it will increase as A_y/A_{max} decreases.

By combining equations (2-3), (2-4b), (2-5b), (2-6b) and (2-1f) or (2-2b) in a ranking analysis, it is possible to quickly estimate potential displacements for multiple embankments. The ratio A_y/A_{max} is designated the yield factor, Y, for this analysis. The displacement, u, retains the dimension of centimeters because the critical displacement is generally the same regardless of embankment height, bridge length, or other characteristic dimensions of the embankment and structure.

2.4 Mechanical behavior immediately following the event

Cohesionless soils in the region may be susceptible to liquefaction. The cohesionless soils present are either alluvium or sandy/gravelly continental deposits. Of these, the alluvium will be the most likely to experience liquefaction. An assessment of the likelihood of liquefaction in cohesionless alluvium in the region was crucial to the study. Kentucky Transportation Cabinet (KTC) is compiling a database of soil boring information throughout the state of Kentucky (Pfalzer, 1995). While still in development, the database already includes thousands of samples, particularly in the western Kentucky region of concern for this study. This database includes sample-specific standard penetration blow counts, grain size data, ground water depth, geologic origin (alluvium) and soil classification. It was thus easy to load the database into a commercial spreadsheet for selection of those samples classified as coarse grained for further analysis of liquefaction susceptibility. A total of 489 samples were selected from the database for evaluation. Of these, 27 were classified as loess, 294 as continental clayey sands and gravels, and 168 as alluvium. The alluvium data consisted of 27 samples classified as fine grained, 77 as coarse grained, and the remainder could not be identified as either due to lack of data. Based on this information, approximately 3/4 of the alluvial soils in the region and within reach of typical drilling programs are coarse grained.

Determination of liquefaction potential in the referenced soils was possible using the Seed *et al.* Method (1983) based on standard penetration test N values. Numerical correction of the N values to the equivalent 1 kg/cm^2 N value was achieved by using an approximation of the C_N correction summarized by Seed *et al.* (1981) as follows:

$$C_N = \left(\frac{98.5}{\sigma'_v}\right)^{0.57} \qquad (2\text{-}7)$$

where σ'_v is expressed in kPa. Note that for σ'_v expressed in tons per square foot, or kg/cm^2, the constant 98.5 in equation (2-7) is replaced with 1.0. It was not possible to correct the N values in the database for SPT efficiency since details of the performance of those tests were not available.

The Seed *et al* Method (1983) includes recommendations for estimating the cyclic shear stress ratio, $(\tau_h)_{ave}/\sigma'_o$, induced in the soil for a specific earthquake:

$$\frac{(\tau_h)_{ave}}{\sigma'_o} \cong 0.65\frac{a_{\max}}{g}\cdot\frac{\sigma_o}{\sigma'_o}\cdot r_d \qquad (2\text{-}8)$$

Where a_{max} is the maximum earthquake ground surface acceleration and r_d is a correction factor for stress reduction. For this study, the mean effective and total stresses, σ'_o and σ_o were replaced with the effective and total vertical stresses computed using an assumption of a soil mass density of 1.92 g/cm^3 and an assumed ground water level 3 meters below the surface. These are conditions that are not unreasonable for alluvial bridge abutment sites. The stress reduction factor, r_d, was computed using the depth, z, in meters and the following equation to estimate the correction as:

$$r_d = (1 - \frac{z}{91}) \qquad (2\text{-}9)$$

This is a reasonable correction for depths up to 15 meters.

Determination of a_{max} in Equation (2-8) required interpretation of earthquake accelerations and response spectra provided by Street *et al.* (1996) for western Kentucky, and site periods also provided by Street *et al.* (1997). The latter report provides lower bound, mean, and upper bound dynamic site periods for 84 different sites throughout western Kentucky. This report also provides interpreted depths to bedrock at the various locations studied. Comparison of the site period and depths to bedrock at each site provided a good correlation, as shown in Figure 2.3. The depth to bedrock data from the report by Street *et al.* (1997) was used to infer bedrock depths for the samples in the KTC drilling samples database, and the resulting site period was subsequently determined using a linear fit to the trend indicated in Figure 2.3.

Once the site period was obtained, the 5% damped design response spectra recommended for each Kentucky county by Street *et al.* (1996) was used to estimate the amplification of the design bedrock acceleration for each county, also provided in the same report, to obtain the value of a_{max}. This was done by assuming a linear variation between the log of acceleration response and the log of period at 0.1 and 1 second on the response spectra for each event as follows:

$$\log\left(\frac{a_{\max}}{a_{peak}}\right) = m \cdot \log(T_s) + d \qquad (2\text{-}10)$$

The peak acceleration, a_{peak}, was for each specific time history, while the value of a_{max} is that used in Equation (2-8). The values for the parameters used for the different time histories are provided in Table 2.2.

None of the sample data from the KTC database included depth to bedrock since that normally exceeds the necessary depth of drilling for bridge foundations in the western Kentucky region. The depths provided by Street *et al.* (1997) were based primarily on seismic refraction testing along with some data from deep placement of seismic monitoring stations. Further, only some of the KTC database data included sufficient soil test results to complete the above computations. A total of 35 sites were assessed in this way, using both the design 50 year and 500 year event. The results are shown on Figure 2.4 along with the liquefaction limit line for soils with less than 5% fines subjected to a magnitude 7.5 event.

Inspection of the figure quickly shows that most of the cohesionless sample data analyzed indicated a “no liquefaction” condition. Of the 35 samples assessed, 26 contained between 5% and 15% fines, so the liquefaction susceptibility of those samples was even lower than that indicated in the figure. This provided evidence that liquefaction of cohesionless alluvial soils in western Kentucky is possible, though most samples would not be considered susceptible to liquefaction.

On the basis of the above study, it was concluded that the alluvium throughout the region could be characterized as having low to moderate liquefaction potential. The undrained shear strength of the alluvium at those locations where liquefaction was believed to be a significant risk was conservatively assumed to be on the order of 20 kPa, as indicated for a clean sand blow count on the order of 12-16 suggested by Seed and Harder (1990). Liquefaction was assumed to not be a problem in the cohesionless continental deposits based on the very high SPT N values observed therein.

2.5 Representative Ground Motion

As described previously, the displacement model developed by Dodds (1997) and summarized previously in equations 4 and 5 were used to estimate the displacement from the input Yield Factor, $Y = A_y/A_{max}$. These equations also require earthquake magnitude, however. For illustration purposes, this portion of the study examined only the 500 year event. The 500 year event was determined to be of magnitude 7 to 7.5 located in the New Madrid Seismic Zone (NMSZ) in southeast Missouri and northeast Arkansas (Street *et al.*, 1996 and 1997). Estimated peak bedrock accelerations, A_{max}, for this event ranged from 17 to 63% of gravity, depending on the location of the embankment being considered.

2.6 Embankment and Foundation Geometry

Embankment geometry includes embankment slope inclination, embankment height, and width and length of the crest of the embankment. Geometry also includes depth and distribution of multiple layers in the embankment. The above analyses assume a simple two-dimensional geometry like that shown in Figure 2.1. It is further assumed that the embankment is constructed of a single material, both the embankment top and base are level, and that the base of the embankment corresponds to the elevation of the toe of the embankment slope. Embankment geometry was thus defined using two parameters: height, H, and slope inclination, b, which is the ratio of horizontal to vertical slope inclination, as shown in Figure 2.1.

Embankment geometry was defined by field surveys conducted by undergraduate engineering students from the University of Kentucky. Field measurements included slope distance from toe to crest, and slope inclination angle along the same interval. The measurements were made using a hand held Brunton compass to obtain slope inclination and a surveying tape for slope length. These procedures were initially checked with careful slope measurements using a level and surveying tape. Upon verifying the methods were sufficiently accurate for the typically irregular slopes, the method was adopted as standard practice for this study. Each slope was also carefully inspected for evidence of impending failure, swampy conditions, or other terrain conditions that might be relevant to embankment stability and later assignment of stability parameters. The methods applied for this study permitted field assessment of between 8 and 12 locations per day.

Foundation stratigraphy and geometry are likely more variable than for the embankment, since the foundation is usually natural soil rather than controlled fill. The contact between softer foundation soils and a harder "bedrock" surface or stiff soils will also typically be irregular. However, definition of these conditions requires a detailed subsurface exploration, which is not possible for KESR. The slope stability component in KESR thus assumes the foundation soils have a uniform undrained shear strength, S_1, usually different from the embankment soils, and that this soil is continuous to contact with a level layer of very high strength at some depth below the embankment. The depth was selected based on the geology for that site and its proximity in the western Kentucky region.

2.7 Assignment of Soil Mechanics Properties of the Embankment and Foundation

Rapid seismic loading will cause undrained failure in cohesive soils and saturated cohesionless soils. Dry and partially saturated cohesionless soils will be subject to behavior intermediate between drained and undrained under seismic loading. Shear strengths assigned to each embankment were adjusted to reflect these cyclic loading effects. Assignment of these parameters obviously involved a level of judgement that introduces significant uncertainty. However, the relative uncertainty is likely comparable to the ground motion prediction, lower than the deformation prediction, and greater than the limit equilibrium analysis. For the purpose of ranking relative stability of the embankments, this was considered acceptable.

Soil Conservation Service (SCS) soil unit and United States Geological Survey (USGS) geologic formation was compiled for each site. The soils predominant in the western Kentucky region are of four types: (1) alluvium, (2) weathered loess, (3) sandy and gravelly old alluvium that is quite dense and geologically referred to as continental deposits, and (4) residuum. Assignment of bedrock depth and shear strength for each embankment and underlying natural soils was based on comparison between the site-specific SCS and USGS information, the KTC drilling sample database, and the information indicating bedrock depth and elevation in Street *et al.* (1997). Ground water was assumed to be below the base of the embankment in the foundation. The local site geometry and terrain was also given careful consideration in judging the likely parameters.

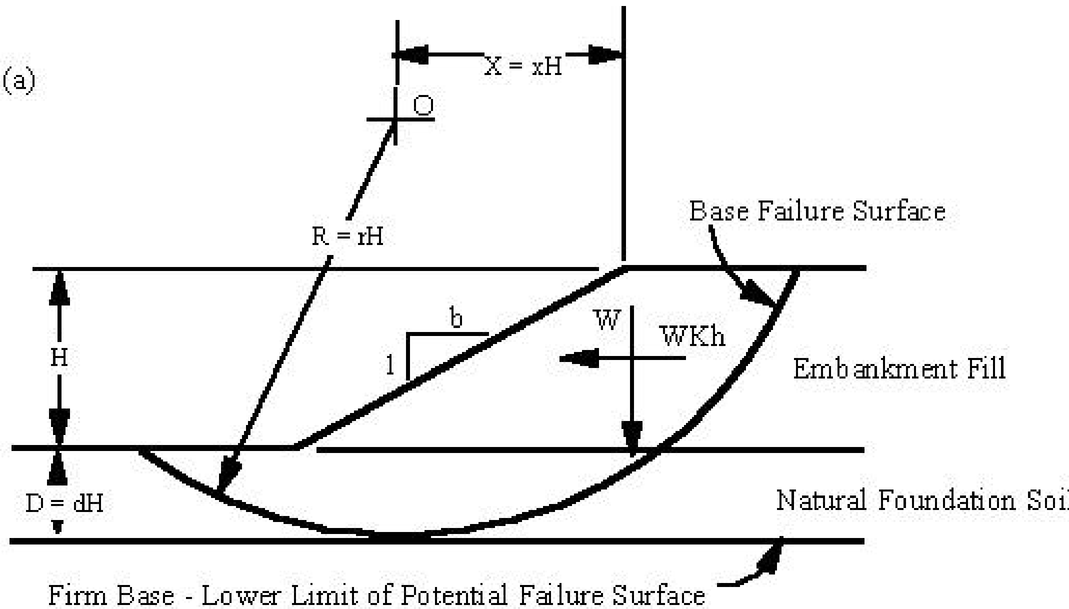
Assignment of shear strength for cohesionless materials was based on standard penetration resistance. As noted previously, those soils judged to have significant liquefaction potential were assigned low shear strength based on work by by Seed and Harder (1990). The shear strength of the cohesive soils was selected through examination of unconfined compression data and rough correlations with standard penetration resistance obtained in the relevant deposits and reported in the referenced KTC database. These data are shown in Table 2.3. The density and shear strength of the embankment soils was conservatively estimated assuming marginal compactive effort may have been applied during construction of older embankments.

2.8 Findings and Conclusions for Ranking Analysis

Of the 408 embankments evaluated in this study, 68 could not be ranked due to unusual site conditions or inadequate data. The remaining 340 sites were ranked relative to either their factor of safety or their estimated displacement using the model described herein. Figure 2.5 depicts estimated displacement versus factor of safety (shear capacity divided by shear demand, C/D) for the sites where displacement could be estimated. This was limited to those sites with C/D values less than or just slightly above 1.0. As can be seen in the figure, as slope stability decreased, larger displacements were observed, providing a stronger indication of risky embankments than that obtained from a factor of safety analysis. The analysis eliminated the misleading condition of how to assess a factor of safety less than one, and instead forced consideration of the possible displacements that may be observed, a better indication of the consequences of a failure.

The findings for all of the embankments are provided in the Appendix. The data is sorted by county, with the most critical embankments listed first and the least critical listed last in each county group. Each site was classified as either class A, B, C, or Z. Class Z embankments were not ranked, as noted above. Class A embankments were those sites where the estimated displacement exceeded 10 cm. Class B sites featured C/D ratios less than one, but displacements less than 10 cm. Class C embankments had a C/D ratio greater than or equal to 1.0. For the 500 year event, 35% of the embankments were rated class A while another 20% were class B. Only 1% of the embankments were class A for the 50 year event, with an additional 22% being class B. A more detailed assessment is recommended for the class A embankments. It is recommended that class B sites also be subjected to at least a preliminary assessment at critical locations along priority routes. The scope of evaluation appropriate for specific embankments should be determined by a qualified geotechnical engineer. Table 2.4 provides a summary of the embankment ranking for all of the counties containing embankments.

(a)



	Density	Shear Strength
Embankment Fill	$\gamma 2 = \gamma 1$	$S2 = \lambda S1$
Natural Foundation Soil	$\gamma 1$	$S1$

(b)

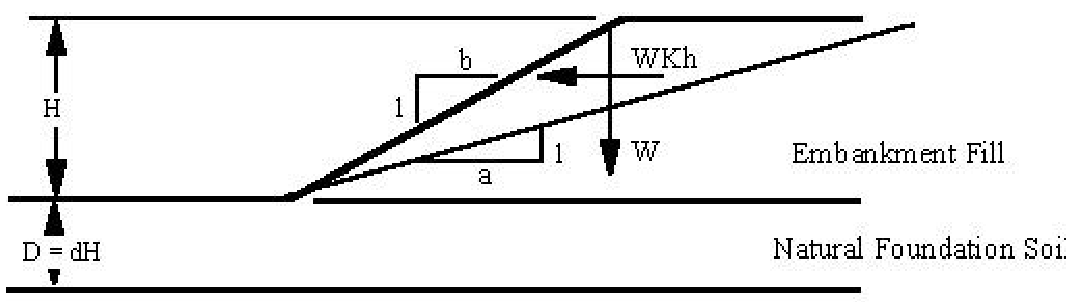


Figure 2.1. Parameters defining pseudo-static (a) circular base failure and (b) single wedge failure.

Table 2.1. Coefficients for Central U.S. Displacement Model

Magnitude Range	α	β_1	β_2
Bedrock Sites			
4.5<Mb,Lg<5.5	-0.69	3.67	-0.56
5.5<Mb,Lg<6.5	-0.1	4.09	-0.65
6.5<Mb,Lg<7.5	0.78	4.37	-0.86
Soil Sites			
4.5<Mb,Lg<5.5	-1.044	2.726	-1.011
5.5<Mb,Lg<6.5	-0.388	2.503	-1.248
6.5<Mb,Lg<7.5	1.006	2.378	-1.122

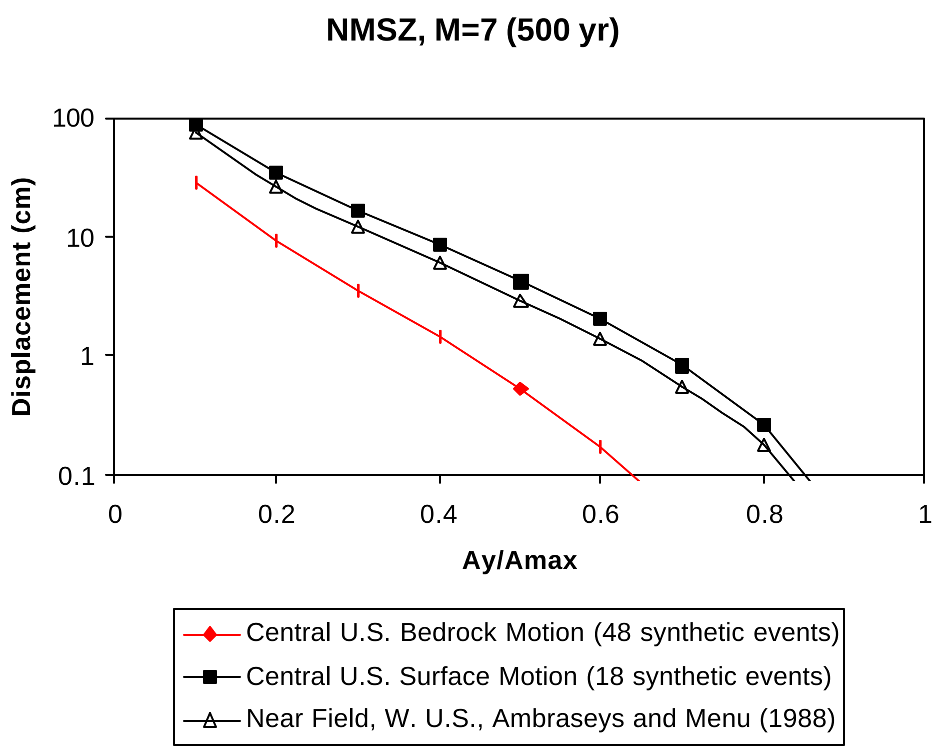


Figure 2.2. Deformation model based on acceleration ratio.

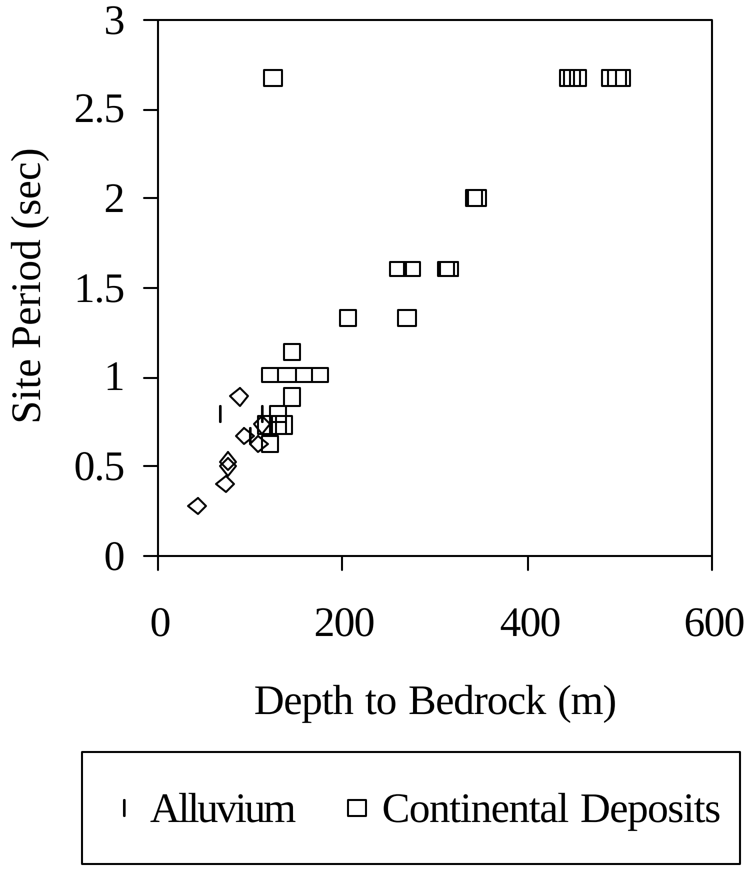


Figure 2.3. Correlation between depth to bedrock and site period for data acquired in western Kentucky.

Table 2.2 Parameters used for Estimating Local Amplification at Specific Sites

	County						
	Ballard Carlisle Fulton	McCracken Graves Marshall	Calloway Trigg Lyon	Ballard Carlisle Fulton	McCracken Graves	Marshall Calloway	Trigg Lyon
Return Period (years)	50			500			
Approx. Amplitude (g)	0.3	0.15	0.09	0.6	0.4	0.3	0.19
d	-0.106	0.063	0.093	0.045	-0.111	-0.139	-0.244
m	-0.555	-0.361	-0.376	-0.317	-0.512	-0.553	-0.647

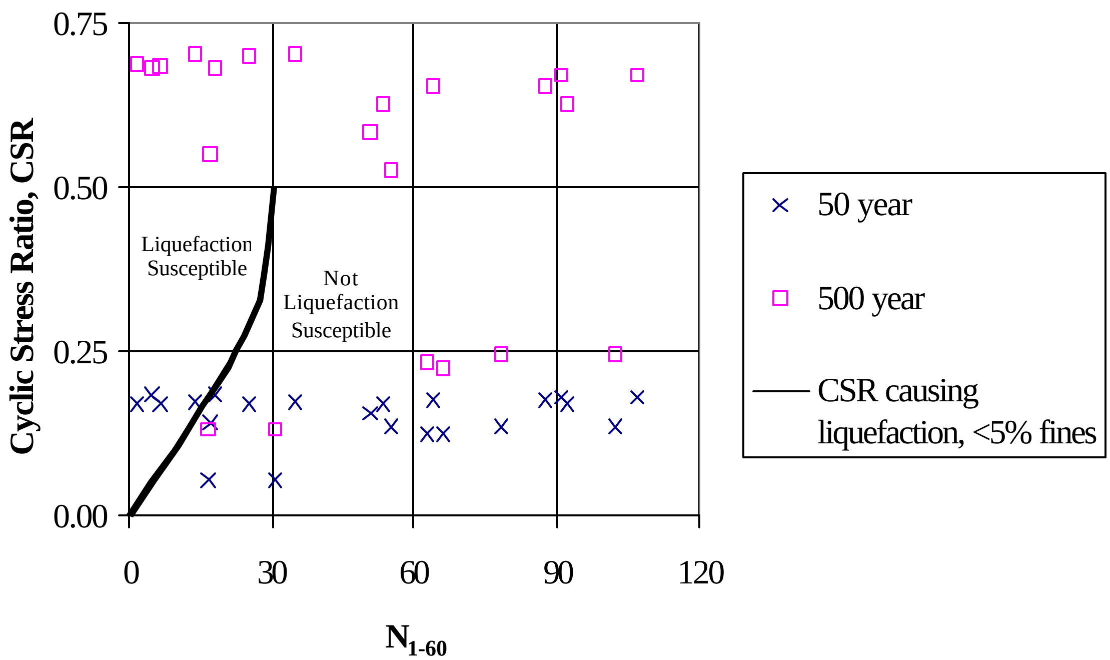


Figure 2.4. Liquefaction susceptibility of alluvial soils and continental deposits in the western Kentucky region.

Table 2.3. Selected density and strength parameters for ranking model

Geologic Formation	Mass density (g/cc)	Shear Strength (kg/cm^2)
Alluvium	1.92	0.20
Weathered Loess	1.84	0.35
Continental Deposits	2.00	0.75
Residuum	2.08	1.00
Embankment	2.00	0.50

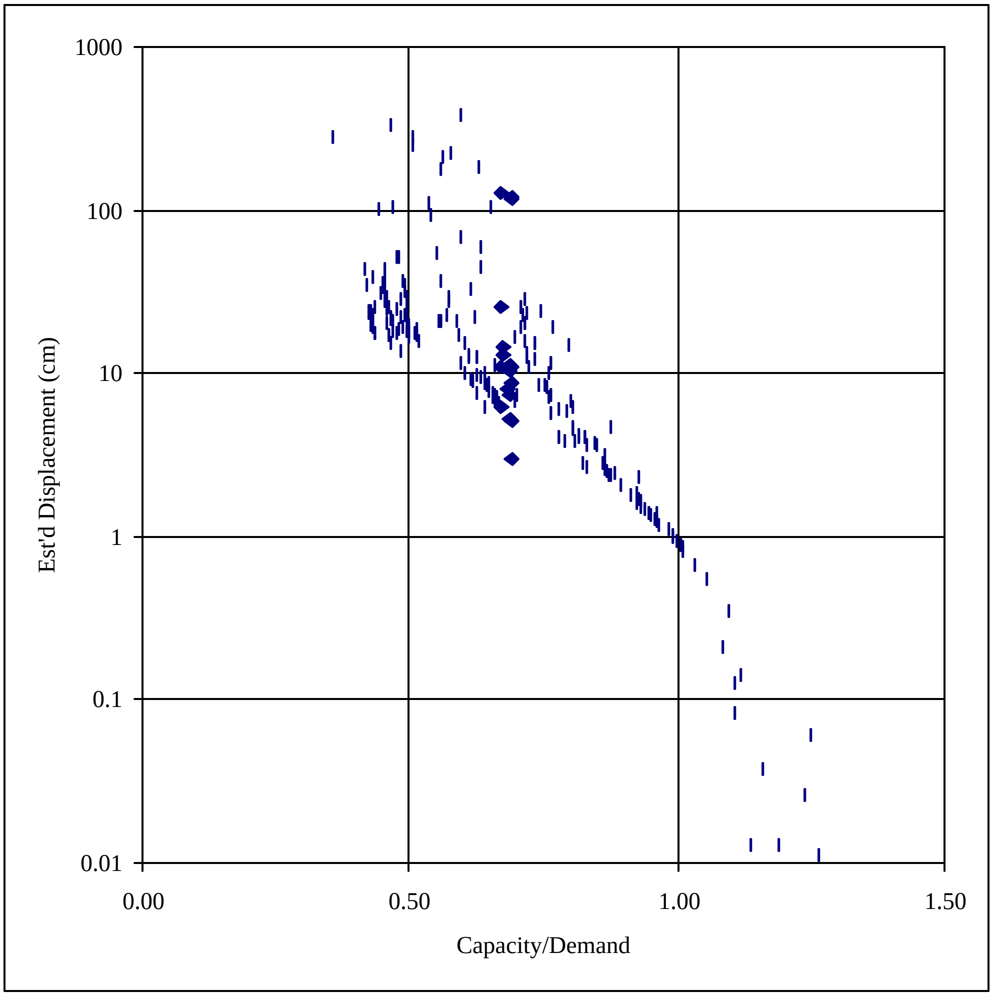


Figure 2.5. Comparison between factor of safety (capacity/demand) and estimated displacement using the KESR model.

Table 2.4 Summary of Embankment Ranking – Number of Embankments per County in each Class

County	County Symbol	PGA (%g)	Class A	Class B	Class C	Not Ranked
50 year event						
Ballard	BA	26.6	1	26	11	5
Caldwell	CD	8.8	0	0	4	0
Christian	CH	9.4	0	0	15	10
Carlisle	CL	26.2	0	15	4	0
Calloway	CW	8.3	0	3	30	5
Fulton	FU	26.8	1	8	0	3
Graves	GR	14.5	2	9	57	6
Hickman	HI	30.8	0	4	0	3
Livingston	LI	12.5	1	0	5	1
Logan	LO	9.1	0	0	11	11
Lyon	LY	8.6	0	2	17	3
Marshall	MA	14.1	1	14	54	5
McCracken	MC	13.4	0	6	16	9
Todd	TO	9.1	0	0	7	5
Trigg	TR	8.9	0	2	14	2
TOTAL			6	89	245	68
% OF TOTAL			1%	22%	60%	17%
500 year event						
Ballard	BA	63.2	35	1	2	5
Caldwell	CD	17.8	0	0	4	0
Christian	CH	14.4	0	0	15	10
Carlisle	CL	60.8	19	0	0	0
Calloway	CW	27.1	2	23	7	6
Fulton	FU	58.7	8	0	0	4
Graves	GR	41.3	35	27	6	6
Hickman	HI	60.5	4	0	0	3
Livingston	LI	25.4	2	0	4	1
Logan	LO	9.7	0	0	11	11
Lyon	LY	20.1	4	1	14	3
Marshall	MA	27.2	20	23	26	5
McCracken	MC	30.9	14	4	4	9
Todd	TO	11.1	0	0	6	8
Trigg	TR	17.1	2	1	13	2
TOTAL			145	80	112	73
% OF TOTAL			35%	20%	27%	18%

3. APPROACH EMBANKMENTS FOR U.S. 51 OHIO RIVER BRIDGE, WICKLIFFE, KENTUCKY

3.1 Project Description

The location of the Ohio River bridge for U.S. 51 near Wickliffe, Kentucky is shown on Figure 3.1, which is a composite of the USGS topographic survey sheets for Wickliffe, Kentucky (1983), Barlow, Kentucky (1977), Cairo, Illinois, (1978), and Wyatt, Missouri (1978). As shown in Figure 3.1, the bridge is in the Ohio River flood plain, approximately 1 mile north of the current confluence of the Ohio and Mississippi Rivers, at approximately Ohio River mile 980.4. The bridge was constructed in the 1930's, with the structure recently studied for seismic stability by Harik *et al.* (1998). The span crosses the Ohio River to southern Illinois at the northern edge of the New Madrid Seismic Zone (NMSZ). The study summarized herein was requested as a preliminary assessment of the seismic stability of the approach embankments for the main span of the two lane bridge for US 51 across the Ohio River. A preliminary assessment was requested because of the proximity of the site to the New Madrid seismic zone.

Most of the roadway from Wickliffe to the Ohio River crossing is supported on raised embankment along the flood plain in an area known as Willow Slough. Assessment of this approximately 3 miles of embankment approaching the Ohio River bridge was not a part of the study. However, since this study was not for a specific exploration, but rather for an assessment of the merit of carrying out such an exploration, the general recommendations provided herein may be relevant to the embankment extending from Wickliffe to the Ohio River bridge.

3.2 Seismic and Geologic Setting

The ongoing seismic activity and the occurrence of three or possibly four major earthquakes along the NMSZ in 1811-1812 is well documented and the subject of considerable research. As described in Chapter 2, Street *et al.* (1996) developed time histories, peak ground accelerations (for bedrock sites), and general recommendations for characterizing the potential seismic load on Kentucky bridges and highways in this region. Street *et al.* (1996) used a deterministic analysis to develop their recommendations for the design peak ground acceleration. The United States Geological Survey has an ongoing program to develop seismic guidelines for the U.S., including NMSZ. USGS uses a probabilistic analysis method for its recommendations for peak ground acceleration. Figure 3.2 depicts the isoseismals for USGS recommended peak acceleration (%g) with 10% probability of exceedance in 50 years, as developed by the USGS National Seismic Mapping Project (1999). The isoseismals shown on Figure 3.2 compare favorably to the peak ground acceleration with a 50 year return period recommended by Street *et al.* (1996) shown in Table 1.2. Isoseismals for other events are also available from USGS. For example, the USGS recommended peak acceleration (%g) with 2% probability of exceedance in 50 years at the bridge site is about 60% of gravity, which agrees well with the 500 year event recommended by Street *et al*. Clearly, the bridge site is a location subject to infrequent but potentially very large ground accelerations.

The upper soil profile in this area is alluvial, comprised of interlayered gravel, sand, silt, and clay, along with varying percentages of organic materials. The alluvium in this area is generally underlain by silty gravels and silty sands of Tertiary or early Quarternary age. These terrace deposits are also often referred to as continental deposits, which were placed by moving water, but are older than recent alluvium and generally denser. Beneath the continental deposits, exploration would be expected to find deposits of the Claiborne formation, comprised of even older terrace deposits and Eocene age alluvium. The original plans for the bridge include logged auger borings, but the borings shown on the plans were actually acquired along the alignment of the Illinois Central Railroad bridge at approximately Ohio River mile 977.7, about 2.7 miles upstream from the U.S. 51 bridge. These borings indicate the Claiborne formation upper boundary was quite variable, with an approximate average el. 220 feet above msl. The normal Ohio River level at the bridge location is approximately el. 290 feet above msl. Some geologic maps define the top of the Claiborne formation in this area as the top of bedrock. However, a better definition of true bedrock for seismic stability analyses is material possessing compression wave velocities in excess of 3,500 feet per second.

Two seismic refraction surveys performed in the area of the bridge were available for this study. One was in Fort Defiance State Park, immediately south of the bridge. This survey was reported to have been quickly performed for no particular project as a matter curiosity one afternoon by Ron Street of University of Kentucky and several of his graduate students (Street, 1997). Dr. Street emphasized the survey was not subjected to his normal degree of care in performance and interpretation, and thus is less accurate than a normal survey. This survey indicated compression wave velocities exceeding 3,500 feet per second at about el. –95 feet. The second seismic refraction survey was performed with a higher degree of care about 1 mile east-southeast of the bridge on the Kentucky side of the river. The data from that survey was collected for the study reported by Street *et al.* (1997). True bedrock, as indicated by compression wave velocities exceeding 3,500 feet per second, was indicated in this survey to be at el. –295, 600 feet below the ground surface.

3.3 Method of Study

In light of the proximity of the bridge to the NMSZ and preliminary indication from Harik (1997) that the superstructure was not likely to be mitigated to withstand a 500 year event, a preliminary study of embankment seismic stability was recommended without pursuing costly geotechnical testing. It was suspected that liquefaction, a major contributing factor to poor embankment stability at the bridge location, would be found to be highly likely for a 500 year event, and not likely for a 50 year event. Further, the embankment at the immediate approach to the main Ohio River crossing would be suspected to be similarly stable to the nearly 3 miles of embankment along Willow Slough from Wickliffe to the Ohio River bridge. Mitigation and investigation of embankment stability would thus be necessary along the entire length of that embankment, a work effort well beyond the scope of this study. In light of this, the method followed in the preliminary analysis of the US 51 crossing of the Ohio River was to use existing data to estimate the likely range of behavior during the design 50 and 500 year events.

A seismic stability analysis involves modeling: (1) the earthquake motion at the source, (2) the attenuation of the source motion as it travels to the bedrock at the site, (3) the amplification of the motion as it travels from the bedrock to the ground surface, (4) the mechanical response of the soils to the amplified motion, and (5) the response of structures supported by the soils at the site. There is substantial uncertainty involved in each of these steps. The time histories provided by Street *et al*. (1996) allow this study to bypass steps (1) and (2) in the above sequence.

Based on the above referenced borings acquired about 2.7 miles upstream of the bridge, and on the results of the two seismic refraction surveys referenced above (Street, 1997; Street *et al*., 1997), a model of assumed stratigraphy at the site was developed. This estimated statigraphy was then used with the time histories provided by Street *et al*. (1996) in the one-dimensional, total stress analysis program SHAKE91 (Idriss *et al*., 1992) to estimate amplification of ground motion from the bedrock to the ground surface at the bridge location. In order to account for uncertainty in the assumed stratigraphy, ranges of relevant soil properties were assigned to each soil layer in the assumed subsurface model, and multiple runs of SHAKE91 were performed for the ranges of soil properties in an attempt to bound the potential behavior at the site. The assumed stratigraphy is shown in Table 3.1. The shear wave velocities in Table 3.1 were estimated from the ranges of compression wave velocities from the referenced refraction surveys using elastic theory.

The SHAKE91 analysis also requires the user to provide the variation of modulus and damping with shear strain for each layer. The design equations developed by Hardin and Drnevich (1972) were used for this purpose. The maximum shear modulus for each layer, G_{max}, is that indicated in Table 3.1. The variation of shear modulus with shear strain can be defined using the concept of reference strain, g_r.

$$g_r = \frac{t_{\max}}{G_{\max}} \qquad (3\text{-}1)$$

t_{max} is the shear stress at failure for each layer. The value of shear modulus, G, for a given strain, γ, is then defined by Hardin and Drnevich (1972) as

$$\frac{G}{G_{\max}} = \frac{1}{1+\dfrac{g}{g_r}} \qquad (3\text{-}2)$$

A maximum damping, D_{max}, of 28% was assumed for each of the soils. The variation of damping, D, with shear strain was defined following Hardin and Drnevich (1972) as

$$\frac{D}{D_{\max}} = 1 - \frac{G}{G_{\max}} \qquad (3\text{-}3)$$

Hardin and Drnevich (1972) and others suggest more specific models of shear modulus and damping variation with shear strain, taking into account the plasticity and grain size of the soil, but for a preliminary study of this type, such refinements are not necessary.

Given the variation of shear modulus and damping from one strata to the next and with shear strain, SHAKE91 assumes total stress conditions to create a one-dimensional model of the soil column from the bedrock to the surface. This one-dimensional model is subjected to the bedrock earthquake motion at the base, and is left free to move at the top (ground surface). The program outputs include acceleration, shear stress, and shear strain versus time in each of the strata and at the ground surface. Figure 3.3 depicts the maximum shear strain, shear stress, and acceleration versus depth for average (norm) soil properties in Table 3.1, along with the same data for the best case (low) and worst case (high) soil properties varying about the average. These results are for the 50 year event. Similar results were generated for the 500 year event.

There are two noteworthy aspects of the data shown in Figure 3.3. First, review of Figure 3.3(a) shows higher strains were encountered near the ground surface and at a depth of about 70 feet below the ground surface. Due to the higher potentially induced strain in these zones, higher shear stresses would be expected at these depths, presenting the opportunity for liquefaction or slope failure in these regions. Of these two zones, the shallower is more critical for slope stability. Second, the maximum acceleration at the ground surface for the worst case scenario in the model is about 28% of gravity for the 50 year event. The maximum bedrock acceleration for that event is 26.6% of gravity, as shown in Table 1.2. Thus, the model predicted only minor amplification of the bedrock acceleration in the soil column for the stratigraphy assumed. Obviously, larger amplification is possible, but the modeling herein indicated this is not highly likely at the site. Evaluation of the 500 year event gave similar results, but the accelerations, strains and shear stresses were much larger.

The shear strength of any soil layer is a function of the normal effective stress, which depends on the depth of the layer, and the water pressure within the layer. Shear strength increases with effective stress. Liquefaction is the failure of loose sandy soils caused by an increase in the pore pressure, which leads to a low effective stress and thus loss of strength. Liquefaction of sandy alluvial soils is a common occurrence near the epicenter of a strong earthquake. Whenever the shear stress induced in the soil by the earthquake exceeds the shear strength, failure occurs. Thus, the shear strength must be compared to the shear stress. To do this, the earthquake-induced shear stress in each layer is first divided by the effective stress in the same layer. This normalized shear stress, since it is due to a cyclic load, is often called cyclic stress ratio. Seed *et al.* (1983) showed good correlation between the standard penetration test (SPT) resistance measured using a conventional drill rig and the limiting cyclic stress ratio causing liquefaction of a deposit. Seed *et al.* (1983) showed that, for a specific magnitude event, there is a curved boundary between liquefaction-susceptible soils and soils not susceptible to liquefaction. That boundary for a magnitude 7 event is shown in Figure 3.4 for both sandy soils with an average grain size (D50) less than 0.15 mm, and sandy soils with an average grain size greater than 0.25 mm. Figure 3.4 shows that for soils with a standard penetration resistance of 20 drops per foot, a higher cyclic stress ratio is required to cause liquefaction of a soil with D50 < 0.15 mm. Thus, soils containing higher percentages of clay and silt are less susceptible to liquefaction because they require higher cyclic shear stresses to fail.

The modified penetration resistance shown on Figure 3.4 is the number of hammer drops per foot of sampler travel using a conventional standard penetration hammer. Details of the methods used to modify the results for non-standard SPT methods are provided in Seed *et al.* (1983). For the U.S. 51 Ohio River crossing, the range of potential SPT modified penetration resistance values was selected based on typical values at various depths in the sandy and alluvial deposits of the Ohio River flood plain. Two soil profiles in the upper 40 feet were considered since there was evidence of silty, sandy, and gravelly deposits, all potentially liquefiable, in the subsurface conditions considered for this analysis. The selected potential range of resistance counts was from a drop count as low as 6 near the surface to as high as 22 at depths of 30-40 feet. It is generally accepted that liquefaction is not highly likely at depths greater than 30-40 feet, so only the potential values in the upper 40 feet were considered in this study. For the range of N values relevant to the two assumed soil profiles, and taking into account the range of potential cyclic stress ratios at the corresponding depths for the 50 and 500 year events, zones of likely conditions at the site were defined. These zones are indicated by the cross hatched boxes in Figure 3.4 for the 500 year event, and Figure 3.5 for the 50 year event.

3.4 Findings and Recommendations for U.S. 51 Ohio River Bridge, Wickliffe

Figures 3.4 and 3.5 depict the potential liquefaction behavior of the alluvial soils in the area of the U.S. 51 Ohio River Bridge near Wickliffe. The potential behavior for a design 500 year earthquake is shown in Figure 3.4. Most of the cross-hatched region is in the area of "zone of liquefaction" for relatively coarse sands (D50 > 0.25 mm) for both profiles considered. For finer grained sands, including those sands containing clays or fines, over one-half of the potential liquefaction behavior is in the zone of liquefaction. This suggests that liquefaction is highly likely in the alluvial soils near the bridge if the design 500 year earthquake occurs. The occurrence of such liquefaction would result in loss of embankment stability and potentially loss of the bridge abutment near the liquefied zones. It is recommended a plan be in place for emergency repair of the embankment, abutment, and adjoining bridge span in the event of a 500 year event. It should be noted this study did not evaluate the stability of the foundations for the bridge itself, and liquefaction of the soils around these foundations is also a likelihood, possibly causing collapse of the structure. Extensive remediation of liquefiable deposits adjoining foundations was required for the new bridge crossing the Mississippi River near Cape Girardeau, Missouri, approximately 28 miles northwest of the US51 bridge.

The potential liquefaction behavior for the design 50 year earthquake is shown in Figure 3.5. Note the range of likely cyclic stress ratios is considerably lower than that shown in Figure 3.4. Less than 50% of the cross hatched area falls in the "zone of liquefaction" for coarse sands, and very little of the cross hatched area falls in the zone of liquefaction for finer sands that likely include more silt and clay fines. These findings indicate there is a low risk of liquefaction of pockets or zones of clean sands for a 50 year event. This risk is further enhanced in low relative density sands near embankments by the additional in situ shear stresses caused by the embankments.

Conditions that may be marginally favorable for liquefaction during a 50 year event likely exist at several zones along the approximate 3 mile embankment that extends from Wickliffe to the Ohio River Bridge. It would be difficult to delineate all such zones, even if a detailed subsurface investigation were performed. Thus, costly remediation of marginally liquefiable deposits would likely be necessary along the full length of the embankment to assure no failures or deformation due to a 50 year event. It is possible that a 50 year event could cause numerous smaller slump failures along the embankment face and some embankment settlement, but it is unlikely that the magnitude of the failures would be sufficient to interrupt the flow of traffic in both lanes. Temporary repairs to provide two lanes of traffic, if a lane were lost, could be completed expeditiously by emergency equipment. It is recommended a plan be in place for the completion of such repairs in the event of the occurrence of a 50 year earthquake.

3.5 Summary

A 500 year earthquake may result in loss of at least a portion of the approach embankment and possibly the bridge abutment for this structure. Deformations of the abutment due to a 500 year event could exceed 3.28 feet. If remediation to withstand a 500 year event is to be considered for this embankment, a more detailed geotechnical study will be necessary.

A 50 year event presents little to no risk of area-wide liquefaction, although isolated pockets of clean loose sands may cause localized slumps or failures along portions of the approach embankment. There is a remote chance of a flow failure if a highly liquefiable layer of clean sand exists over a significant zone adjacent to the embankment. Remediation of all such deposits would be difficult and costly, as compared to the cost associated with repair if such failures occurred. Deformations at this abutment due to liquefaction of the soils in this area would likely be minor, although this cannot be ascertained without a detailed subsurface investigation. It is unlikely that a 50 year event would cause sufficient deformation of the abutment or soils in the approach embankment to result in conditions that cannot be quickly repaired for temporary access, or that would cause extended closure of the bridge while repairs are made.

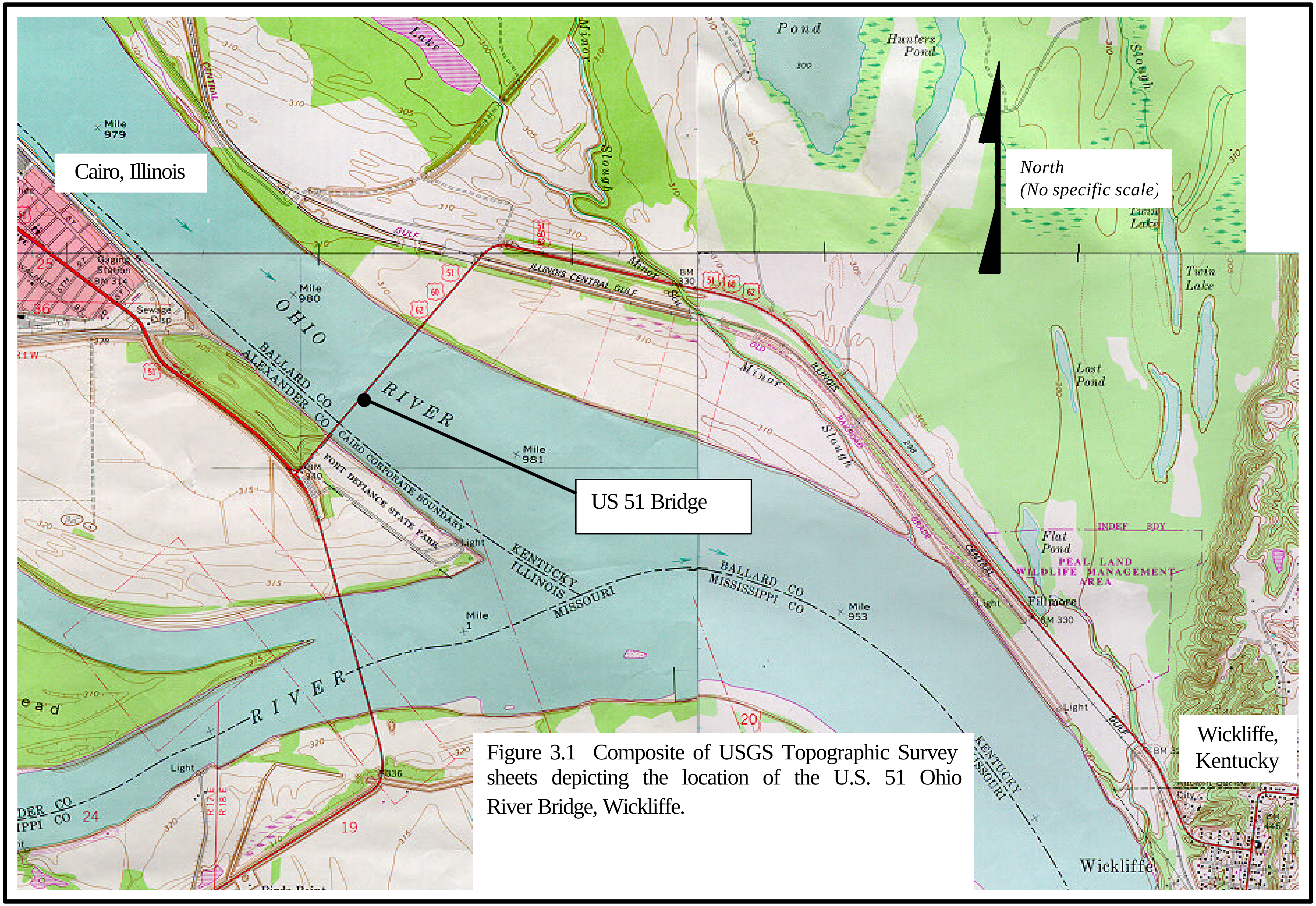


Figure 3.1 Composite of USGS Topographic Survey sheets depicting the location of the U.S. 51 Ohio River Bridge, Wickliffe.

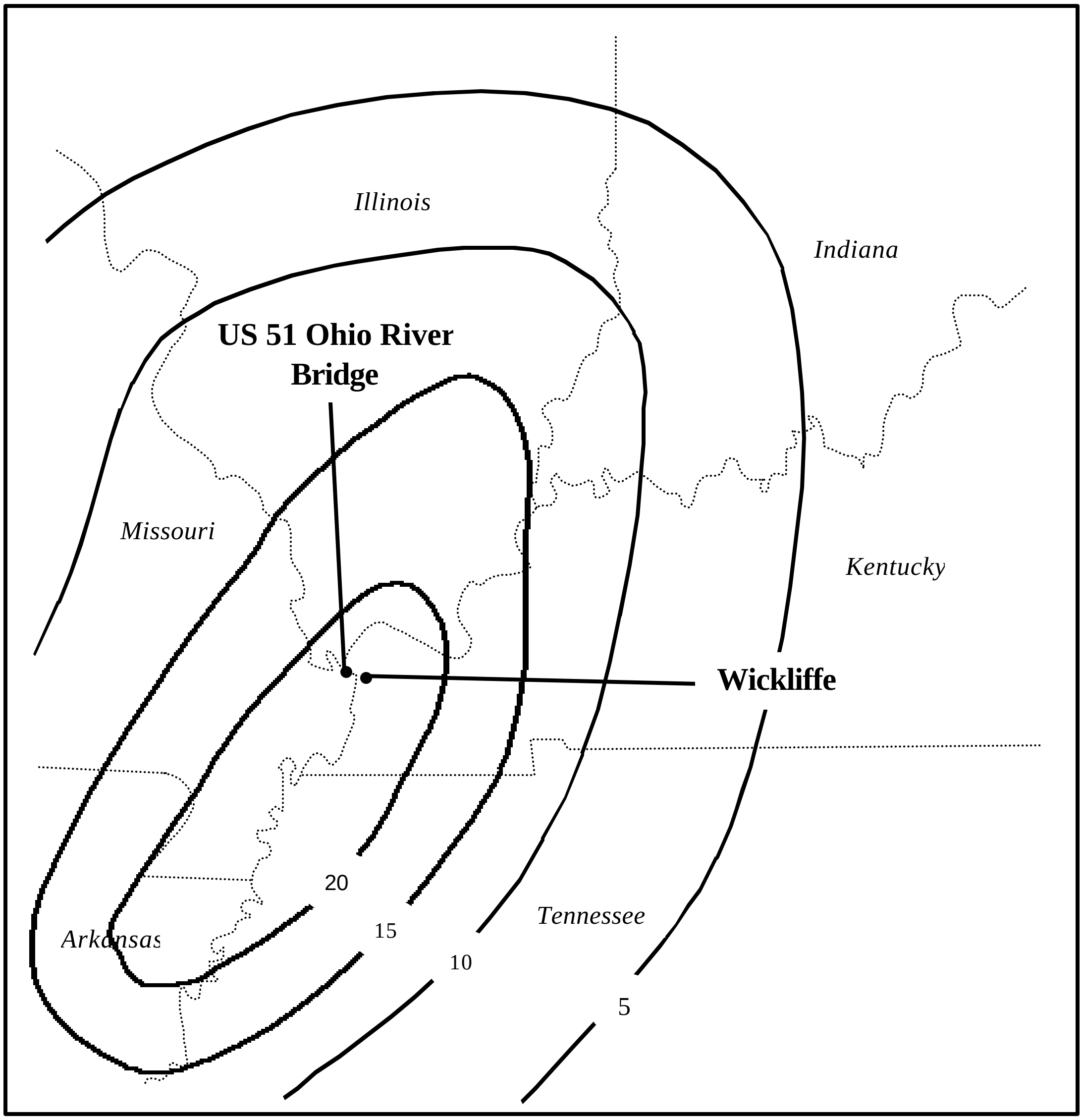
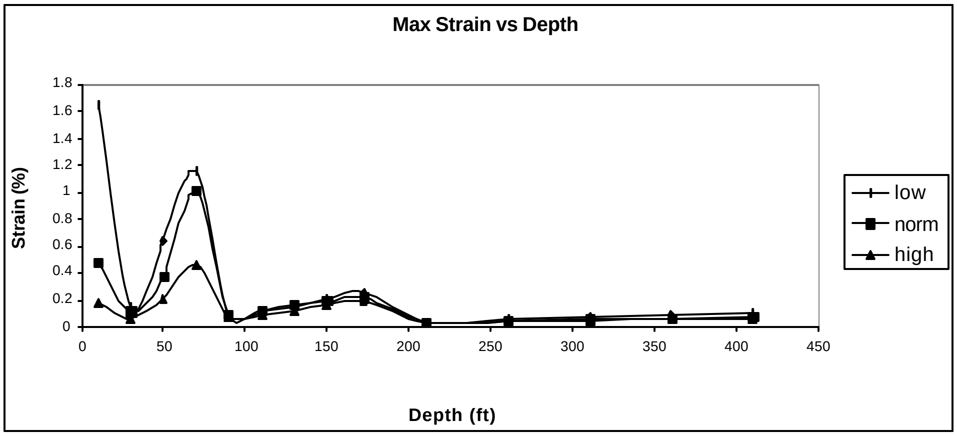


Figure 3.2. Location of US 51 Ohio River bridge along with isoseismals for peak acceleration (%g) with 10% probability of exceedance in 50 years (USGS National Seismic Mapping Project, 1999).

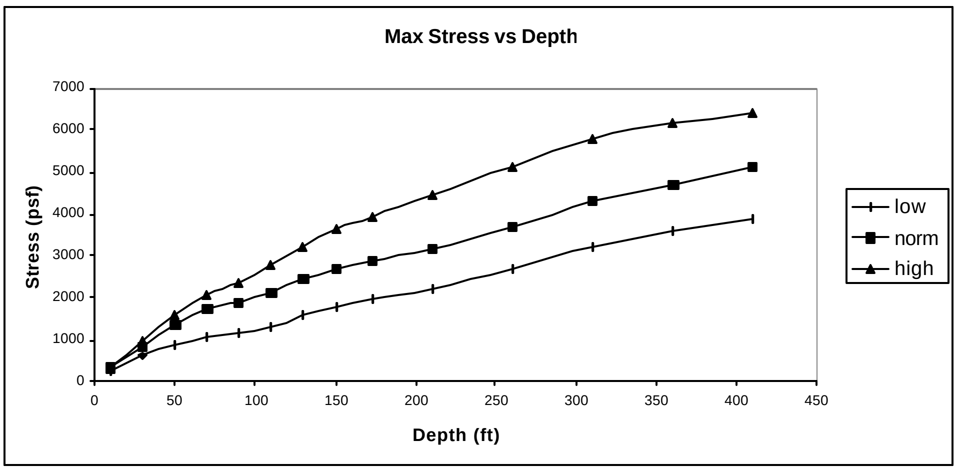
Table 3.1 Assumed Stratigraphy for U.S. 51 Ohio River Bridge, Wickliffe.

Layer No.	Thickness (ft)	Shear Wave Velocity (ft/sec)	Shear Modulus (psf)
1	10	380	538
2	20	650	1640
3	20		
4	20		
5	20	935	3529
6	20		
7	20		
8	20		
9	23		
10	37	1710	12258
11	50		
12	50		
13	50		
14	50		
Below Layer 14		4625	93003

(a)



(b)



(c)

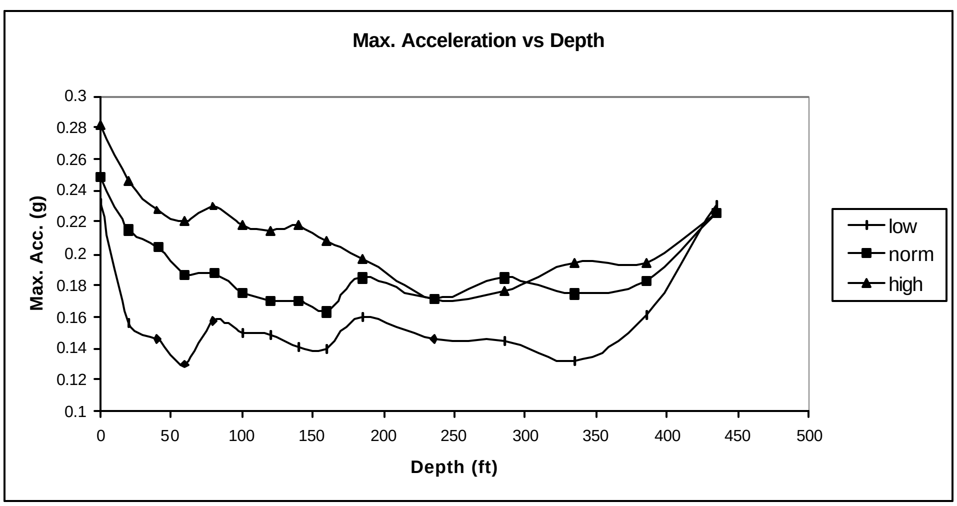


Figure 3.3. Estimated maximum stress, strain and acceleration versus depth for 50 year event at U.S. 51 Ohio River Bridge, Wickliffe.

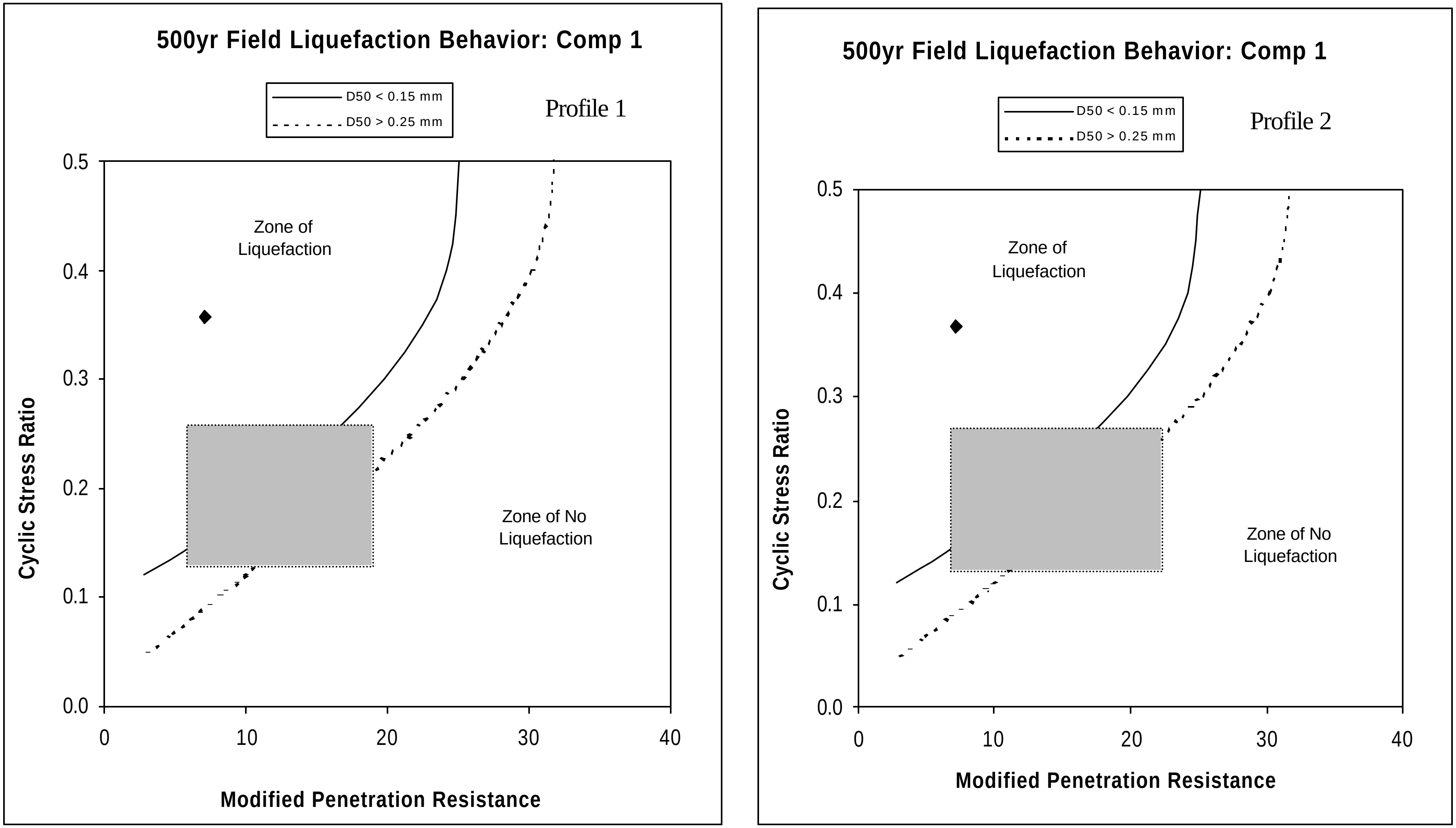


Figure 3.4. Region of likely alluvial sand penetration resistance versus estimated 500 yr event cyclic stress ratio for U.S. 51 Ohio River Bridge, Wickliffe. Liquefaction boundaries shown are for M = 7 earthquake.

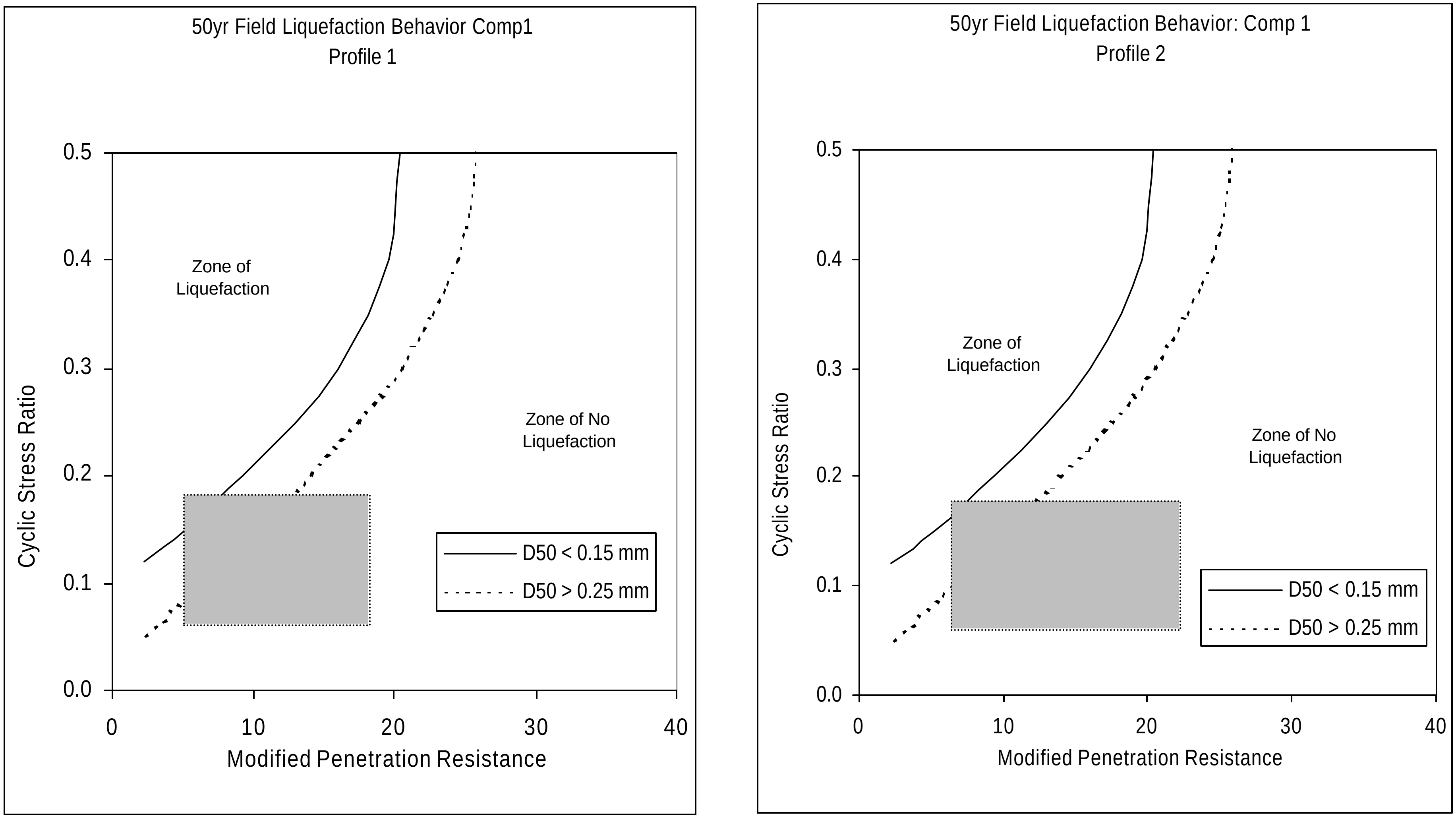


Figure 3.5. Region of likely alluvial sand penetration resistance versus estimated 50 yr event cyclic stress ratio for U.S. 51 Ohio River Bridge, Wickliffe. Liquefaction boundaries shown are for M = 6.3 earthquake.

4. APPROACH EMBANKMENTS FOR U.S. 41 OHIO RIVER BRIDGE, HENDERSON, KENTUCKY

4.1 Project Description

The twin Ohio River bridges for U.S. 41 are located just north of Henderson, Kentucky just upstream of Ohio River milepoint 787. The location of the twin spans is shown on Figure 4.1, as depicted on the USGS topographic survey sheet for the Evansville South quadrangle. The structure was recently studied for seismic stability by Harik et al. (1999a and b). The multi-span bridge extends approximately 5400 feet from abutment to abutment across the Ohio River to southern Indiana and the city of Evansville, Indiana. The two bridges were not constructed at the same time, with the northbound span being designed in 1929 and the southbound span designed in 1963.

The study summarized herein was requested as an assessment of the seismic stability of the approach embankments for the two bridges. As shown on Figure 4.1, both ends of the bridges are in Kentucky, so analyses were performed for the abutments at both ends.

4.2 Seismic Setting

The bridges are located approximately 140 miles (225 km) northeast of the New Madrid Seismic Zone (NMSZ) and on the eastern edge of the region designated by Street *et al.* (1997) as the Wabash Valley Seismic Zone (WVSZ). As described in Chapter 3, guidelines for peak ground acceleration and recommended synthetic earthquake time histories have been developed for each county in Kentucky (Street *et al.* 1996, 1997). Street *et al.* (1996) recommended both the 50 and 500 year peak ground acceleration for Henderson, Kentucky be 13.9 percent of gravity. The 500 year peak ground acceleration is associated with the 500 year event in the WVSZ, while the 50 year peak acceleration is associated with the 50 year event in the NMSZ, which was more critical than the 50 year event in the WVSZ. Also as noted in Chapter 3, the United States Geological Survey has an ongoing program to develop seismic guidelines for the U.S., including the area of Henderson, Kentucky. The USGS uses a probabilistic analysis to develop its guidelines while Street *et al.* used a deterministic analysis. Figure 4.2 depicts the isoseismals for USGS recommended peak acceleration (%g) with 10% probability of exceedance in 50 years, as developed by the USGS National Seismic Mapping Project (1999). The 50 year peak acceleration recommended by Street *et al.* (1996) compares favorably to the isoseismals shown on Figure 4.2.

A geotechnical earthquake analysis of the Evansville, Indiana area by Rockaway and Frost (1997) examined liquefaction potential, ground motion amplification, and landslide susceptibility. That study considered extreme events occurring in the WVSZ with body wave magnitudes (Mb) of 6.5, 7.3 and 8.0, and an event in the NMSZ with a body wave magnitude of 7.3. Note the smallest WVSZ earthquake considered in that study coincides approximately in magnitude to the 500 year event shown in Table 1.1 recommended by Street *et al.* (1996). However, attenuation of Rockaway and Frost's Mb = 6.5 event in the WVSZ over a distance of 50 km to Evansville resulted in a estimated peak ground acceleration of 26% of gravity, twice that postulated for the 500 year event by Street *et al.*(1996) for Henderson, approximately 14 km south of Evansville. This discrepancy can be partly attributed to assumption of different attenuation functions and the greater distance to Henderson as opposed to Evansville. The discrepancy also emphasizes the significant disparity that may be encountered in similar, equally valid studies completed by different investigators using different assumptions. Rockaway and Frost (1997) found the Mb = 6.5 WVSZ event could result in light to moderate liquefaction of soils in many portions of the Evansville area with several zones indicating the potential for severe liquefaction.

4.3 Geologic Setting and Previous Subsurface Data

The 1973 USGS geologic map for the Evansville South quadrangle indicates alluvial soils in the area of the abutments, which are in the flood plain of the Ohio River. Alluvial soils in the Ohio River flood plain generally include interlayered gravels, sands, silts and clays wih some orgainic zones. The granular deposits are usually relatively loose, and the clayey deposits are generally compressible and of low to moderate strength. Test borings were available for portions of the newer bridge. These boring verified layers of loose to medium dense sands and silts, soft to medium consistency clays, some gravel seams, and mixtures of all of these. The borings also indicated sandstone and shale bedrock was encountered at elevations ranging from El 240 to 280 feet msl. Normal pool elevation for the Ohio River at this location is about el. 342 feet msl.

4.4 Method of Study

Determination of the seismic stability of the approach embankments for the two spans requires evaluation of the shear strength of the soils in and below the embankment, resolution of the seismic loading conditions, and development of a model for comparison of the shear strength to the loading conditions. Unlike the U.S. 51 study summarized in Chapter 3, the approach embankments for the U.S. 41 bridges are less likely to fail during both the 50 and 500 year events due to less severe loading conditions, so a more detailed analysis is appropriate to verify performance. For this study, crosshole geophysical testing was used at both the north and south abutments to prepare an accurate shear stiffness profile for modeling local amplification of ground motion from the bedrock to the ground surface. A single test boring was obtained at both abutments to profile samples for classification, undisturbed samples of clayey layers for laboratory testing, and a standard penetration test profile for estimating friction angle and liquefaction susceptibility of the sandy layers in the profile. It was not possible within the scope of the study to prepare a detailed survey of the site geometry or profile of the embankments, nor was it possible to obtain a sequence of borings along the abutments to develop the subsurface stratigraphy along the slopes in the abutment areas. The study conducted was adequate for a reliable assessment of liquefaction susceptibility and an approximation of behavior of embankment slopes during seismic loading, thus permitting a valid estimate of seismic stability of the approach embankments.

4.5 Drilling Program

Drilling was initiated at the north abutment in May of 1997 along the north side of an access road beneath the bridges, near the centerline of the median between the bridges. The drilling at this location was unsuccessful due to poor surface and subsurface conditions, including potential rubble or rip rap fill in the region between the two spans. In addition, rainfall and subsequent flooding of the Ohio River submerged this test location before drilling could be completed. The location of this unsuccessful exploration is shown on Figure 4.3. As river levels subsided, a slightly elevated location at the toe of the west edge of the embankment for the southbound lanes was selected for another drilling attempt. This test drilling was successful at the location indicated on Figure 4.3. The test boring, designated N-1, was completed on June 2, 1997 to a depth of 102 feet without encountering bedrock. Further advancement of the drilling augers was not possible due to the high groundwater levels, sand intrusion on the augers, and a layer of cobbles at that depth. The surface elevation at the drilling location was approximately el. 370 feet msl. Drilling conducted in the 1960's for design of the southbound bridge indicated bedrock at approximately el. 260 feet msl at this location, and this was assumed for this study.

In situ testing and sampling during the drilling included standard penetration testing (ASTM D 1586) and undisturbed sampling using the conventional Shelby tube method (ASTM D 1587). Disturbed samples collected during the standard penetration testing were retained in sealed containers for further examination and testing in the laboratory. The results of the drilling are summarized on the boring log for test hole N-1 provided in Figure 4.4.

A second objective for the drilling was to set up the site for crosshole geophysical testing. This geophysical testing required three cased holes on an approximately 10 foot spacing over as much of the full depth of the soil profile as possible. The drilling for N-1 provided one of these holes, so two more were advanced adjacent to N-1. The three cased borings were installed in general accordance with ASTM D 4428. The borings were advanced using 8 inch O.D. hollow augers while maintaining a hydrostatic head of water inside the augers higher than the surrounding groundwater. The borings were cased with 3 inch diameter casing placed inside the hollow augers, and grout was tremied around the casing, while extracting the augers from around the casing. While grouting the casing for geophone 2, approximately 20 feet from N-1, excessive grout take indicated some loss of integrity of the hole. As noted below, this was believed to have caused the casing for geophone 2 to be ineffective for the crosshole testing. Each of the borings was completed at the surface with a surface-flush mount monitoring well cover to provide a secure permanent installation with clearance for mowing. The boring layout, along with the spacing between the borings, is shown in Figure 4.5.

Interpretation of the crosshole test data requires a precise determination of the spacing between borings throughout the test depth. Typical drilling does not assure perfectly vertical borings, so a borehole deviation survey is necessary to determine deviation of each of the casings from vertical. This was performed for all three cased borings using a SINCO slope inclinometer. The results of this survey are also shown on Figure 4.5.

The test drilling for the south abutment was also completed on June 2, 1997. The test boring, designated S-1, was advanced to auger refusal at a depth of 80 feet in shale bedrock at the location depicted in Figure 4.6. The ground surface at S-1 was approximately el. 365 feet msl. For reference and comparison, Figures 4.3 and 4.6 were prepared to the same scale. The ground surface at S-1 was approximately el. 365 feet msl. The results of the drilling are summarized on the boring log for test hole S-1 provided in Figure 4.7. As described above for the north abutment, two additional holes were advanced, and all three were cased, to permit crosshole geophysical testing. The three borings were cased in the same manner as the north abutment. The casing locations at the ground surface, and alignment with depth, are shown on Figure 4.8.

4.6 Geophysical Testing

Ground motions in the cased boreholes at the north and south abutments were recorded using Mark Products Model L-10 Borehole Geophones. These geophones employ three velocity transducers to record small ground motions in the three axial directions (vertical and horizontal at 90 degrees). Only the vertical motion transducer (Z) is used in the crosshole testing. The approximately 34" long geophone assembly is equipped with spring-loaded clamps that are triggered when the geophone contacts a hard casing bottom. Upon triggering, the clamp extends against the casing, locking the geophone at that depth. Measurements for crosshole testing were thus initiated at the bottom of the holes, and testing on successive intervals was completed by pulling the geophones out of the hole to successively smaller depths with the clamp extended.

The crosshole shear wave hammer is manufactured by Bison Instruments (Model 1465-1). The hammer is designed to generate vertically polarized shear waves at specific depths through up and down blows of the hammer assembly on an anvil clamped at the desired depth in one of the end boreholes. Down blows were achieved by allowing the hammer to fall on the anvil, while up blows were achieved by jerking upward on the hammer cable. Typically, it was possible to obtain stronger up blows than down blows during testing.

All signals from the geophones were collected, evaluated, and displayed using a Hewlett Packard 3562A Dynamic Signal Analyzer and hard drive module. The analyzer permitted immediate viewing of wave trains, along with comparison of two independent wave trains, while the data was being collected. Upon evaluation of the results of a test, the data was stored on a 3-1/2" floppy disk and later backed up to the hard drive for more careful laboratory assessment of the data.

Crosshole testing was performed in accordance with ASTM D 4428 over the full depth of cased borings at both sites. A measurement at the very bottom of each cased hole was not possible due to clearance required at the bottom of the cased boring for shear wave hammer movement. Data was recorded in the time domain upon being triggered by an accelerometer mounted on the shear wave hammer. The vertical (Z-axis) motion from the geophones in the near and far cased holes were recorded simultaneously to observe the time delay between arrival at the two holes. The arrival time and known distance between the borings was used to directly compute the shear wave velocity between the casings at each depth tested. Crosshole tests were performed on 5 feet depth intervals in the casings. In order to assure the arrival time of a shear wave and not a P (compression) wave, which could be produced by he hammer under some conditions, both up and down blow wave arrivals are evaluated at each depth, producing wave arrivals that occur in opposite directions.

The interpreted shear wave velocity measurements from the crosshole testing are shown on Figures 4.9 and 4.10. The wave velocities between each of the three cased holes is provided.

During crosshole testing, early triggering of the recording system is possible due to high hammer acceleration at the start of hammer drop or hammer pull. This delay, which has been observed to be as much as 1 to 2 msec, can cause a slight error in the measured shear wave velocity, and is most frequently observed on hammer pulls, which are typically more forceful than a hammer drop, but can also occur due to hammer interaction with the multiple cables inside the cased hole. The shear wave velocity from GP1 to GP2 is not influenced by this trigger error, and is thus considered the best estimate of Vs at each depth. Unfortunately, very weak wave arrivals at GP2 sometimes prevent designation of a wave arrival at that geophone. In those cases, only the calculated wave velocity from the hammer to Geophone 1 is shown on the figures, and the influence of early triggering, if it occurred, could not be removed. This occurred throughout the crosshole testing at the north abutment, probably due to difficulties with grouting the casing for geophone 2 during drilling.

4.7 Laboratory Testing

Water content (ASTM D 2216), grain size (ASTM D 422), and Atterberg limits (ASTM D 4318) analyses were completed for specimens collected during the drilling. The specimens were classified in accordance with the unified soil classification system (ASTM D 2487). The density and integrity of undisturbed specimens was determined in the laboratory. The results of these tests are shown on the boring logs for N-1 and S-1, Figures 4.4, 4.7, and 4.11. The undisturbed specimens were assessed for advanced strength testing such as consolidated isotropic undrained triaxial shear testing. Several consolidated undrained triaxial tests were attempted, but judged to be only marginally representative due to the disturbed condition of the samples. The undrained shear strength determined by these tests is also provided on the boring logs for N-1 and S-1.

4.8 Liquefaction Susceptibility Analyses

Liquefaction susceptibility was assessed at the abutments using the method recommended by Seed *et al*. (1983). As described in Chapter 3, the method requires the cyclic stress ratio and the standard penetration resistance of each sand layer in the profile. The cyclic stress ratio is the ratio of the shear stresses induced by cyclic loading to the in situ effective stress without cyclic loading. The standard penetration test (ASTM D 1586) provides a rough measure of the dynamic shear strength of the soil in each layer. Thus the method is a comparison of capacity (strength) to demand (imposed shear stress). For each sand layer in the ground being characterized, the cyclic stress ratio (shear stress divided by in situ mean normal stress) is plotted versus the standard penetration test resistance (an indicator of strength). Seed *et al*. (1983) recommended boundaries between potentially liquefiable and non-liquefiable sands for different magnitude events and different fines contents in the sand.

Details of the method for evaluating liquefaction susceptibility are described in Chapter 3. To review, the shear modulus of the soils in different layers is used in a one dimensional total stress analysis to model the amplification of the bedrock motion as it travels to the ground surface at the site. The software SHAKE91 (Idriss *et al.*, 1992) was used to perform this total stress analysis. Program inputs, including the variation of modulus and damping with strain, were developed in accord with the methods described in chapter 3. The program outputs include acceleration, shear stress, and shear strain versus time in each of the strata and at the ground surface. An example of this data for the 500 year event at the south abutment is shown in Figure 4.12. The significant amplification of the peak acceleration at the bedrock (13.9% of gravity) to the surface (23% of gravity) is notable. Significant amplification was apparent for both the 50 and 500 year events at the north and south abutments. Despite the amplification of maximum acceleration, induced shear strains and shear stresses in the profile were not high. After identifying the induced shear stresses using SHAKE91, these were converted to a cyclic stress ratio by dividing by the in situ effective overburden stress. The cyclic stress ratio in the sandy layers at the north and south abutments were compared to the standard penetration resistance to determine if there was evidence of liquefiable deposits.

Typical plots of standard penetration resistance versus cyclic stress ratio for the 50 and 500 year events at both the north and south abutments are shown in Figures 4.13 and 4.14. With the exception of one or two "outlier" data points, nearly all of the points are in the zone of no liquefaction on the plots. The standard penetration resistance for those points above the liquefaction line on the plots is excessively low and of suspect accuracy. It is likely that particular test was performed on highly disturbed specimens, though there remains a possibility that the data point represents a truly liquefiable zone in the subsurface.

The liquefaction study described above and in Chapter 3 was based on the assumption of level ground conditions with no induced shear stresses due to a slope or embankment. Of course, in both cases, there is an embankment in the area of concern. A modification factor for the cyclic stress causing liquefaction, K_α, is recommended by Seed and Harder (1990) as being appropriate. However, the value of K_α for sands with relative density greater than about 45% is 1.0 or greater, implying that induced shear stress may actually contribute to increased shear strength. It was not possible to accurately estimate the relative density of the sands in the area of the north and south abutments, but consideration of the observed standard penetration resistances suggests relative densities greater than about 40 to 45%. Thus, as a slightly conservative measure, no correction for induced shear stress was applied to the evaluation of liquefaction susceptibility.

In summary, the liquefaction susceptibility analysis for the north and south abutments indicated a low likelihood of liquefaction for the 500 year earthquake and no significant likelihood of liquefaction for the 50 year event. Thus the following seismic stability assessment for the two abutments did not include soil shear strengths assuming the occurrence of liquefaction in the abutment area.

4.9 Seismic Abutment Stability

Additional drilling to delineate the variation of stratigraphy along the centerline of the approach embankments for the northbound and southbound lanes was not within the scope of this study. Thus it was not possible to prepare a reliable stratigraphic section for each of the four approach embankments for the two bridges. The backwater slough shown just south of the north abutments, as shown on Figure 4.1, was present at the time of this exploration, and appeared potentially larger than that depicted on the figure. It is believed the presence of this slough will contribute to reduced stability of the north abutments. The water body shown on Figure 4.1 just north of the south abutments was not present at the time of this study, and the topography in that area was relatively flat. Based on the existing geometry at the time of drilling, the assumption of somewhat consistent layering of strata beneath the embankments, and some conservative assumptions for subsurface conditions in the region of the embankments, simple pseudo-static slope stability analyses were performed for the four approach embankments. This type of analysis is essentially a conventional slope stability analysis with the addition of a horizontal earthquake acceleration of about 2/3 the peak acceleration, applied in a direction that favors slope movement. The accelerations used for the 50 and 500 year events were between 13 and 16% of gravity, which was 2/3 of the peak surface acceleration indicated by the SHAKE91 analysis for the 50 and 500 year earthquakes.

These preliminary slope stability analyses indicated a factor safety between 0.9 and 1.1 for the stability of the north abutments. . The rough approximation of expected behavior for the south abutments indicated a factor of safety between 1.0 to 1.2 for both the 50 and 500 year events. A factor of safety of 1.0 is often considered acceptable for seismic stability, as this marginal level of stability is often based on conservative assumptions, and even if some slippage does occur, the movement, barring liquefaction of the foundation soils, would typically be relatively minor and repairable at a cost comparable to or less than the cost of remediation.

The details of these analyses are not provided herein since they were not based on an extensive test boring program, which is necessary for development of a reliable stratigraphic section throughout the area of the four embankments. The presence of a weak zone not detected by this exploration could reduce the actual factor of safety.

4.10 Summary

Liquefaction susceptibility analyses for the 50 and 500 year events indicated little to no significant risk of liquefaction at the abutments for these design earthquakes. Preparation of a detailed stratigraphy for both north and south abutment embankments was not possible due to the limited budget provided for exploration, so only preliminary slope stability assessments were possible for the embankments. Those preliminary slope stability analyses indicated pseudo-static factors of safety for the 50 and 500 year events on the order of or slightly greater than 1.0. These factors of safety are acceptable for earthquake loading, as they indicate the possibility of some slope deformation during the seismic loading, but not outright failure since there is little evidence liquefaction will occur. These analyses were preliminary, however, and the actual factor of safety could be lower if an undiscovered loose sand or similar weak layer exists in the stratigraphy at the toe of any of the embankments.

Remediation of the embankments to improve stability or reduce liquefaction potential is not recommended. It is recommended there be an emergency program in place that will provide for rapid deployment of earthmoving equipment so that repairs can be made in the event of an earthquake which causes some deformation of the embankments sufficient to prevent normal flow of traffic to the bridges. It should also be noted that this study did not include investigation of the foundations beneath the bridges. It is possible that foundation failure could occur due to seismic loading, either including or not including the occurrence of liquefaction in looser deposits in the main river channel.

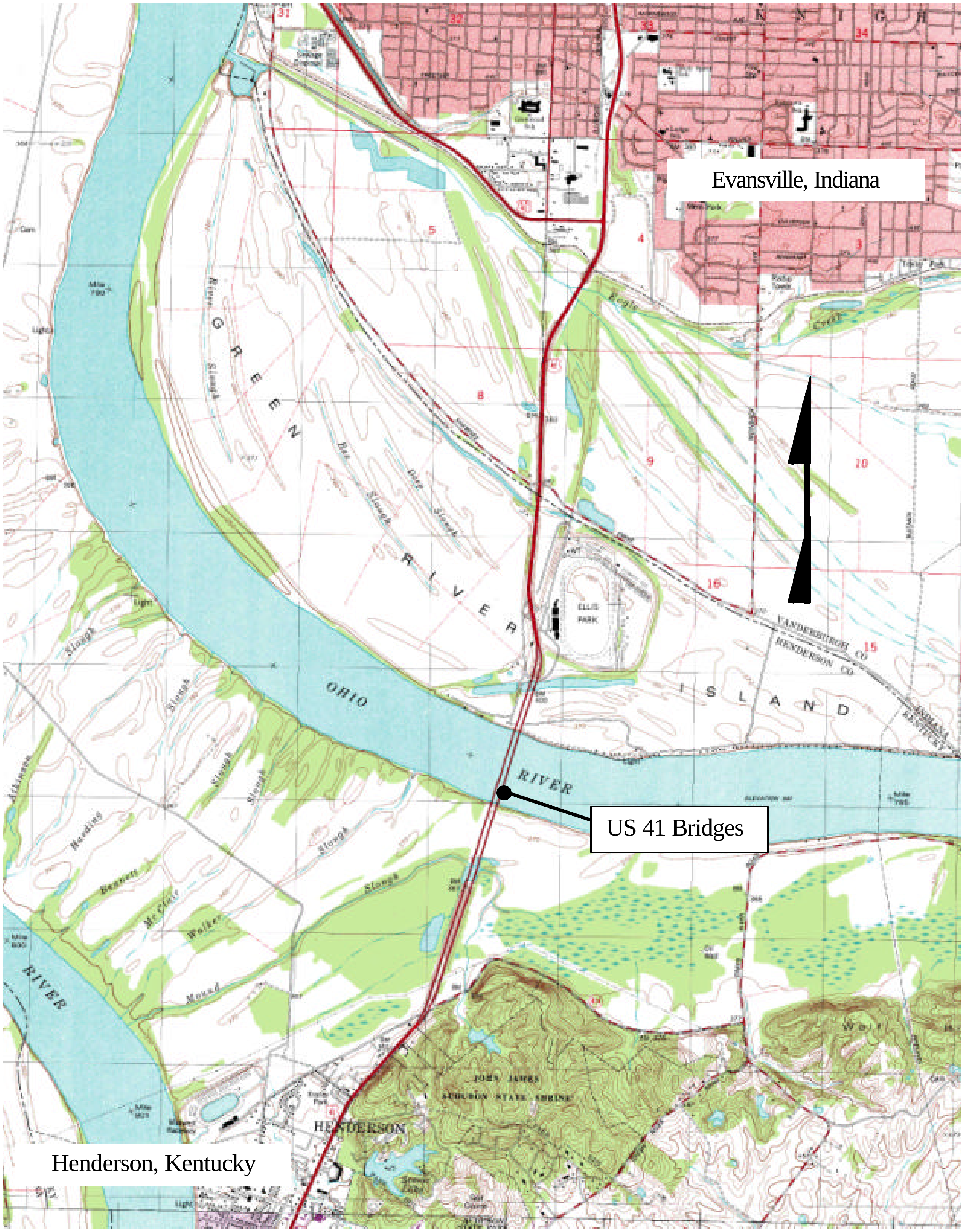


Figure 4.1 Location of U.S. 41 Ohio River Bridges as depicted on USGS Topographic Survey sheet for Evansville, Indiana.

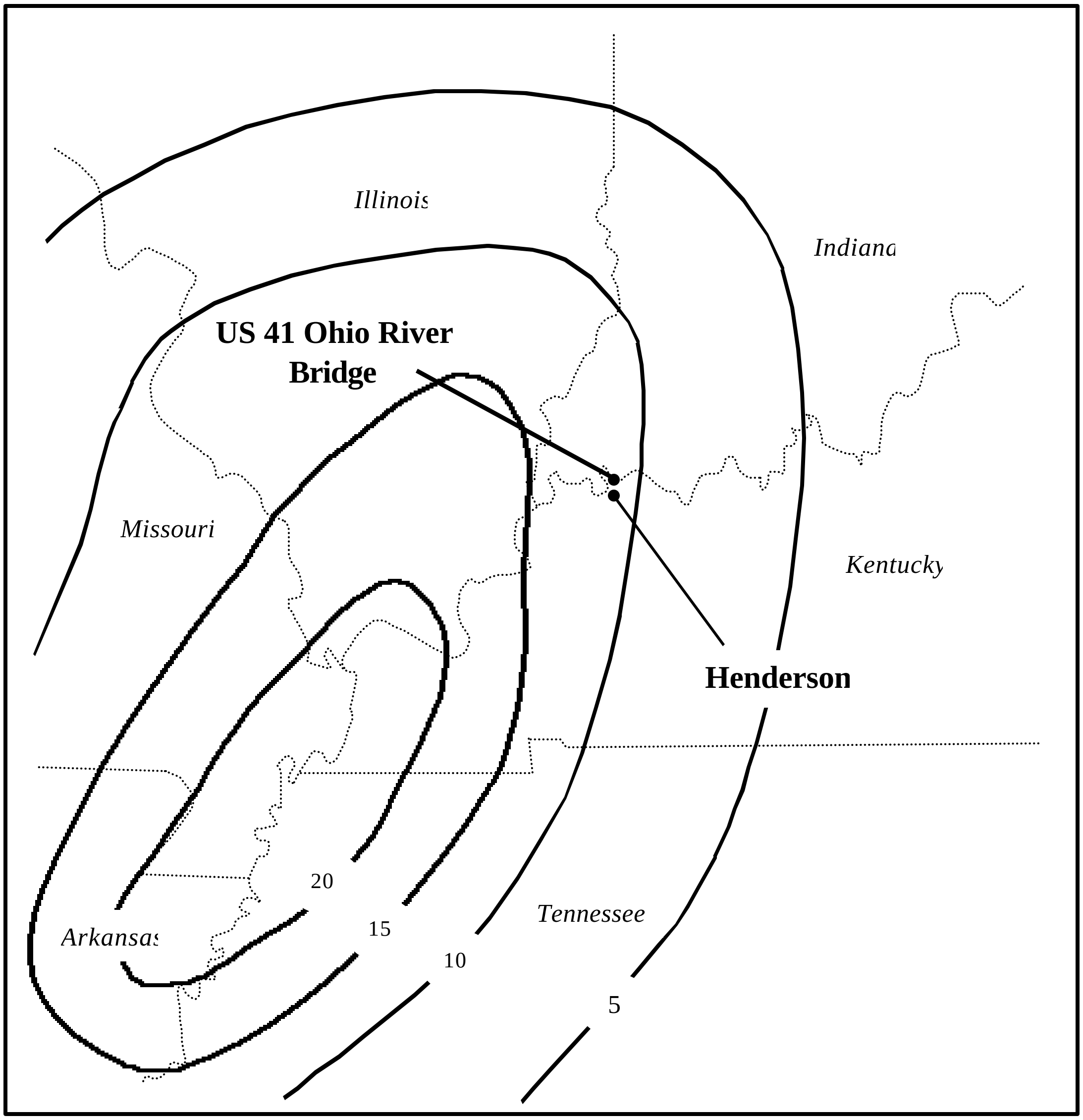


Figure 4.2. Location of US 41 Ohio River bridge along with isoseismals for peak acceleration (%g) with 10% probability of exceedance in 50 years (USGS National Seismic Mapping Project, 1999).

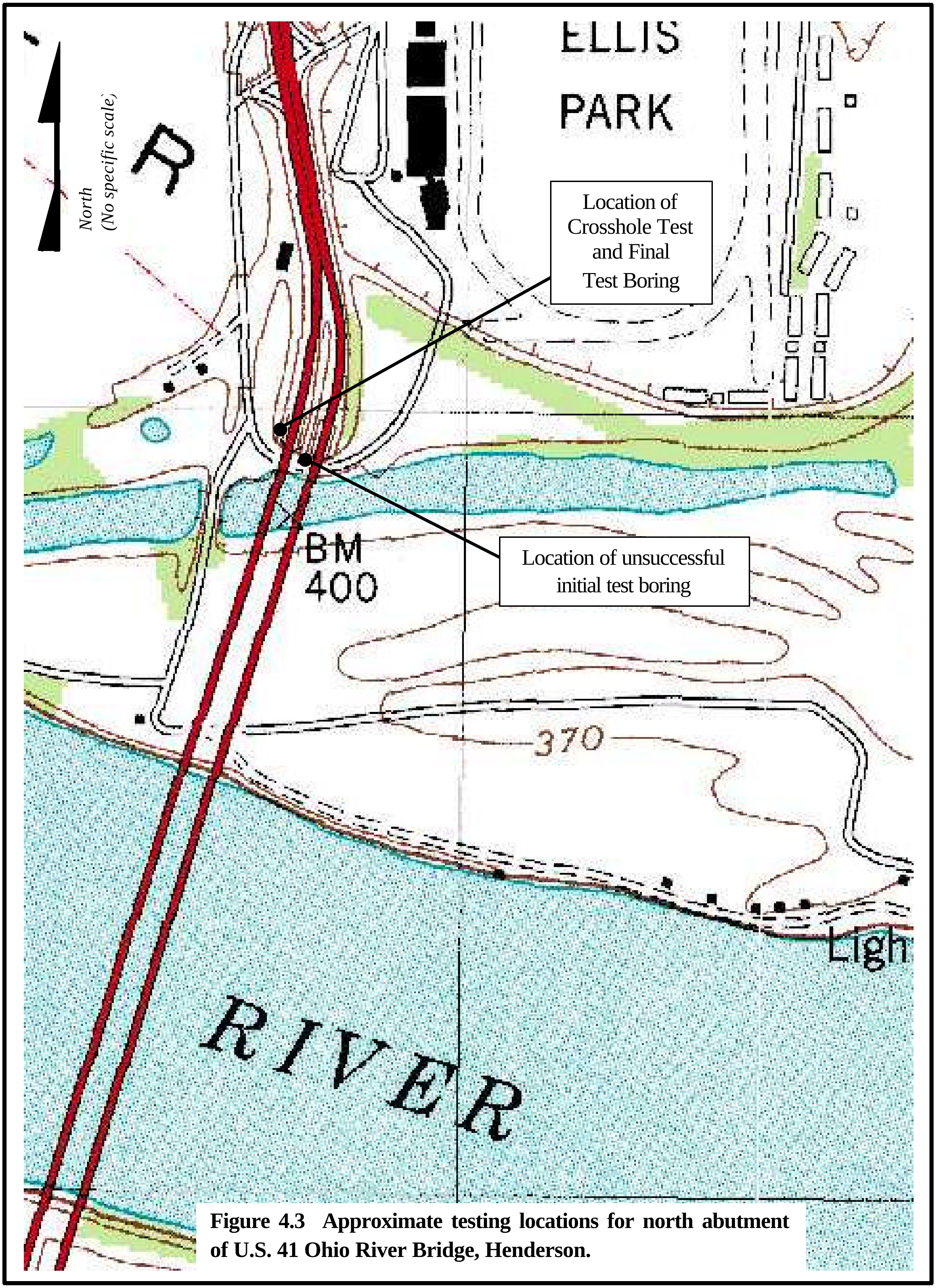


Figure 4.3 Approximate testing locations for north abutment of U.S. 41 Ohio River Bridge, Henderson.

BORING LOG

PROJECT U.S.41 BRIDGE OVER OHIO RIVER
LOCATION HENDERSON CO., KY
DRILLER GEO-DRILL INC.
SURFACE ELEVATION ______ DRILLING METHOD H.S.A W/HEAD OF WATER

BORING NO. N1
SHEET 1 OF 4
PROJECT NO. ______
DATE DRILLED 6/2/97

DEPTH (FT.)	NUMBER	TYPE	RECOVERY (IN/IN)	BLOWS (PER 6 IN.)	DESCRIPTION	UNIFIED SOIL CLASSIFICATION	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFIED COMPRESSIVE STRENGTH (KSF)	LIQUID LIMIT	PLASTIC LIMIT	DEPTH (FT.)
5	1	SS	15/18	1-1-2	Brown LEAN CLAY	CL		40.6					5
10	2	SS	19/18*	1-3-4		CL		36.8			45	25	10
15	3	ST											15
20	4	SS	12/18	2-2-2		CL		32.6			35	21	20
25	5	SS	13/18	2-1-1	Grey SILT With Sand	ML		32.8					25
30	6	SS	18/18	7-9-11	Grey LEAN CLAY With Sand	CL		28.3					30

WATER LEVEL:	REMARKS:
______ NO GROUND WATER NOTED AT TIME OF DRILLING 7 FT. WHILE DRILLING ______ FT. ______ HRS. AFTER DRILLING ______ FT. ______ HRS. AFTER DRILLING	

Figure 4.4a

BORING LOG

PROJECT	U.S.41 BRIDGE OVER OHIO RIVER	BORING NO.	N1
LOCATION	HENDERSON CO., KY	SHEET	2 OF 4
DRILLER	GEO-DRILL INC.	PROJECT NO.	
SURFACE ELEVATION		DATE DRILLED	6/2/97
DRILLING METHOD	H.S.A W/HEAD OF WATER		

DEPTH (FT.)	NUMBER	TYPE	RECOVERY (IN/IN)	BLOWS (PER 6 IN.)	DESCRIPTION	UNIFIED SOIL CLASSIFICATION	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFIED COMPRESSIVE STRENGTH (KSF)	LIQUID LIMIT	PLASTIC LIMIT	DEPTH (FT.)
					Brown, Fine, Poorly Graded SAND With Silt	SP-SM		35.3					
35	7	SS	24/18*	4-7-10		SP-SM							35
40	8	SS	12/18	14-18-19		SP-SM							40
45	9	SS	12/18	10-18-19		SP-SM							45
50	10	SS	12/18	10-18-19	Brown, Medium, Well Graded SAND With Gravel	SW							50
55	11	SS	13/18	9-7-13		SW							55
					Brown, Fine, Well Graded GRAVEL With Sand	GW							
60	12	SS	18/18	18-17-20	Brown, Medium, Well Graded SAND	SW							60

WATER LEVEL:	REMARKS:
____ NO GROUND WATER NOTED AT TIME OF DRILLING	
7 FT. WHILE DRILLING	
____ FT. ____ HRS. AFTER DRILLING	
____ FT. ____ HRS. AFTER DRILLING	

Figure 4.4b

BORING LOG

PROJECT U.S.41 BRIDGE OVER OHIO RIVER | BORING NO. N1
LOCATION HENDERSON CO., KY | SHEET 3 OF 4
DRILLER GEO-DRILL INC. | PROJECT NO.
SURFACE ELEVATION ________ DRILLING METHOD H.S.A W/HEAD OF WATER | DATE DRILLED 6/2/97

DEPTH (FT.)	NUMBER	TYPE	RECOVERY (IN/IN)	BLOWS (PER 6 IN.)	DESCRIPTION	UNIFIED SOIL CLASSIFICATION	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFIED COMPRESSIVE STRENGTH (KSF)	LIQUID LIMIT	PLASTIC LIMIT	DEPTH (FT.)
					Brown, Medium, Well Graded SAND								
65	13	SS	18/18	10-15-15		SW							65
70	14	SS	17/18	10-12-19		SW							70
75	15	SS	23/18*	6-9-13		SW							75
					Brown, Fine, Poorly Graded SAND								
80	16	SS	22/18*	8-13-18		SP							80
					Brown, Coarse, Well Graded SAND With Gravel	SW							
85	17	SS	14/18	3-6-9	Olive, Fine, Poorly Graded SAND With Silt	SP-SM							85
90	18	SS	23/18*	21-23-25	Olive, Medium, Well Graded SAND With Silt	SW							90

WATER LEVEL:	REMARKS:
____ NO GROUND WATER NOTED AT TIME OF DRILLING 7 FT. WHILE DRILLING ____ FT. ______ HRS. AFTER DRILLING ____ FT. ______ HRS. AFTER DRILLING	

Figure 4.4c

BORING LOG

PROJECT U.S.41 BRIDGE OVER OHIO RIVER
LOCATION HENDERSON CO., KY
DRILLER GEO-DRILL INC.
SURFACE ELEVATION
DRILLING METHOD H.S.A W/HEAD OF WATER

BORING NO. N1
SHEET 4 OF 4
PROJECT NO.
DATE DRILLED 6/2/97

DEPTH (FT.)	NUMBER	TYPE	RECOVERY (IN/IN)	BLOWS (PER 6 IN.)	DESCRIPTION	UNIFIED SOIL CLASSIFICATION	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFIED COMPRESSIVE STRENGTH (KSF)	LIQUID LIMIT	PLASTIC LIMIT	DEPTH (FT.)
					Olive, Medium, Well Graded SAND With Silt								
95	19	SS	22/18*	10-15-18		SW-SM							95
100	20	SS	0/18	18-24-35	No Sample Recovery								100
					Drilling Terminated at 101 FT. Without Auger Refusal								
105													105
110													110
115													115
120													120

WATER LEVEL:

____ NO GROUND WATER NOTED AT TIME OF DRILLING
7 FT. WHILE DRILLING
____ FT. ____ HRS. AFTER DRILLING
____ FT. ____ HRS. AFTER DRILLING

REMARKS:

Figure 4.4d

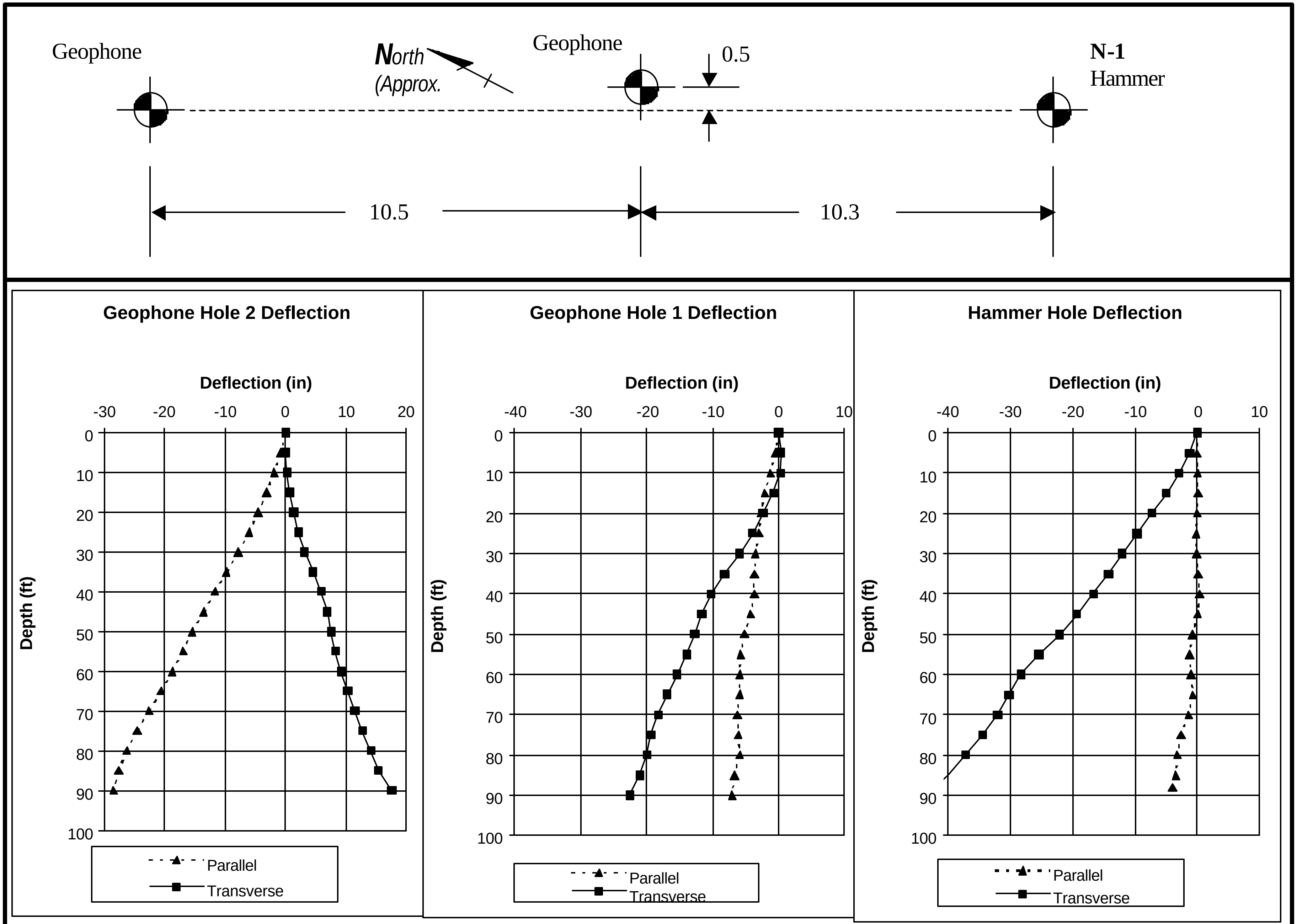


Figure 4.5. Boring layout and results of deviation survey (+Parallel=Northwest, +Transverse=Northeast) **HENDERSON N-1**

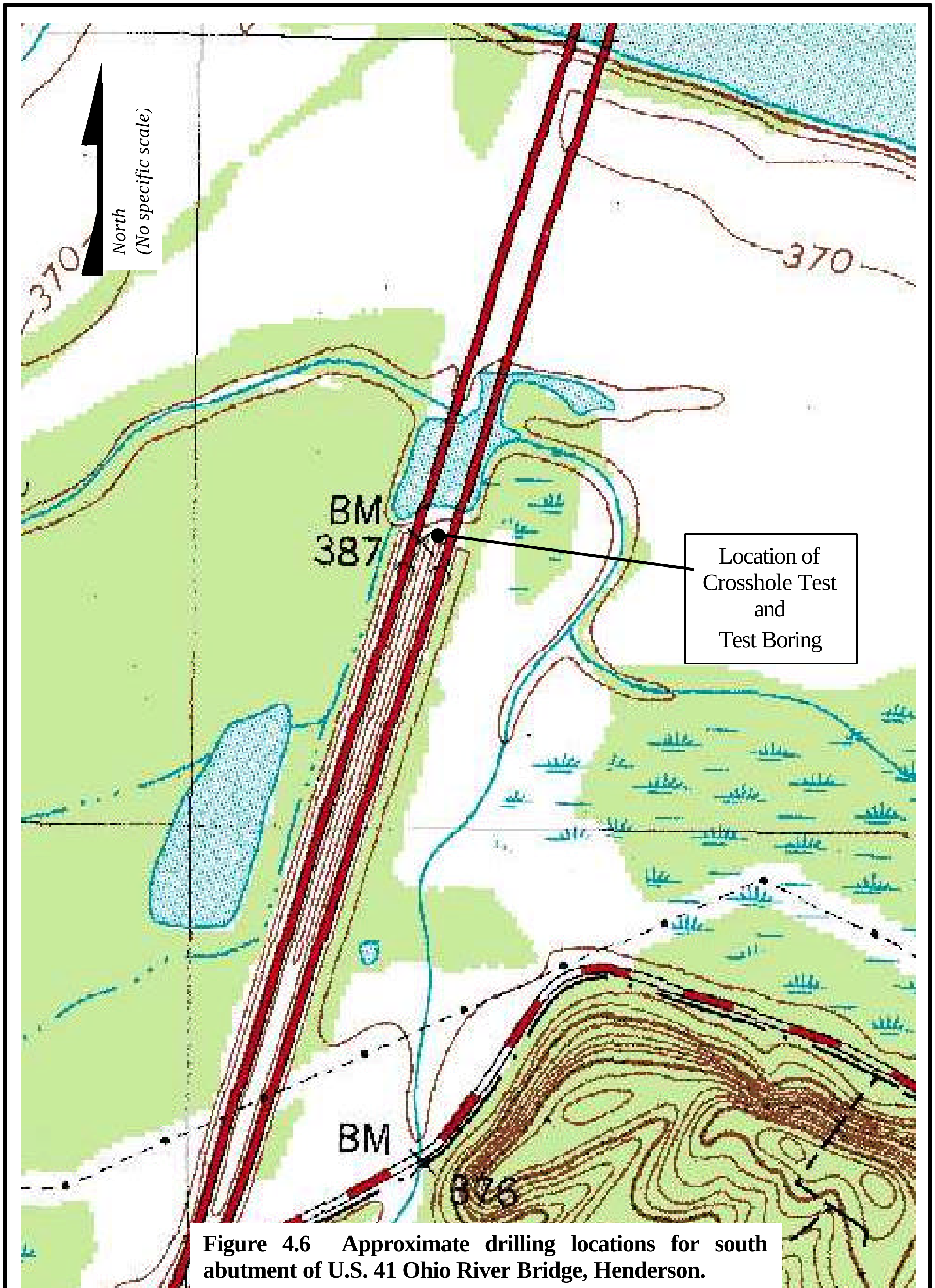


Figure 4.6 Approximate drilling locations for south abutment of U.S. 41 Ohio River Bridge, Henderson.

BORING LOG

PROJECT U.S.41 BRIDGE OVER OHIO RIVER
BORING NO. S1
LOCATION HENDERSON CO., KY
SHEET 1 OF 3
DRILLER GEO-DRILL INC.
PROJECT NO.
SURFACE ELEVATION
DRILLING METHOD H.S.A W/HEAD OF WATER
DATE DRILLED 6/2/97

DEPTH (FT.)	NUMBER	TYPE	RECOVERY (IN/IN)	BLOWS (PER 6 IN.)	DESCRIPTION	UNIFIED SOIL CLASSIFICATION	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFIED COMPRESSIVE STRENGTH (KSF)	LIQUID LIMIT	PLASTIC LIMIT	DEPTH (FT.)
5	1	SS	9/18	4-5-5	Brown Lean CLAY With Sand	CL		15.2					5
10	2	SS	8/18	4-5-5	Brown Lean CLAY	CL		22.3					10
15	3	SS	11/18	4-6-8		CL		25.3			29	19	15
20	4	SS	14/18	5-2-2	Brown, Fine, Poorly Graded SAND, Sometimes With Silt	SP-SM							20
25	5	SS	6/18	3-3-4		SP-SM							25
30	6	SS	6/18	2-6-6		SP-SM							30

WATER LEVEL:	REMARKS:
____ NO GROUND WATER NOTED AT TIME OF DRILLING 7 FT. WHILE DRILLING 7 FT. 24 HRS. AFTER DRILLING ____ FT. ____ HRS. AFTER DRILLING	

Figure 4.7a

BORING LOG

PROJECT U.S.41 BRIDGE OVER OHIO RIVER
LOCATION HENDERSON CO., KY
DRILLER GEO-DRILL INC.
SURFACE ELEVATION ______ DRILLING METHOD H.S.A W/HEAD OF WATER

BORING NO. S1
SHEET 2 OF 3
PROJECT NO. ______
DATE DRILLED 6/2/97

DEPTH (FT.)	NUMBER	TYPE	RECOVERY (IN/IN)	BLOWS (PER 6 IN.)	DESCRIPTION	UNIFIED SOIL CLASSIFICATION	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFIED COMPRESSIVE STRENGTH (KSF)	LIQUID LIMIT	PLASTIC LIMIT	DEPTH (FT.)
35	7	SS	18/18	6-18-8	Brown Medium Well Graded SAND, Sometimes With Silt	SW							35
40	8	SS	18/18	7-14-13		SW							40
45	9	SS	13/18	6-11-10		SW							45
50	10	SS	10/18	6-7-10	Brown Medium Well Graded SAND With Gravel & Silt	SW							50
55	11	SS	18/18	5-10-10	Brown Medium Well Graded SAND With Silt	SW							55
60	12	SS	11/18	13-17-21		SW							60

WATER LEVEL:

____ NO GROUND WATER NOTED AT TIME OF DRILLING
7 FT. WHILE DRILLING
7 FT. 24 HRS. AFTER DRILLING
____ FT. ____ HRS. AFTER DRILLING

REMARKS:

Figure 4.7b

BORING LOG

PROJECT U.S.41 BRIDGE OVER OHIO RIVER
LOCATION HENDERSON CO., KY
DRILLER GEO-DRILL INC.
SURFACE ELEVATION ______ DRILLING METHOD H.S.A W/HEAD OF WATER

BORING NO. S1
SHEET 3 OF 3
PROJECT NO. ______
DATE DRILLED 6/2/97

DEPTH (FT.)	NUMBER	TYPE	RECOVERY (IN/IN)	BLOWS (PER 6 IN.)	DESCRIPTION	UNIFIED SOIL CLASSIFICATION	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFIED COMPRESSIVE STRENGTH (KSF)	LIQUID LIMIT	PLASTIC LIMIT	DEPTH (FT.)
65	13	SS	14/18	7-9-9	Brown Medium Poorly Graded SAND With Silt	SP							65
70	14	SS	12/18	16-15-28	Grey Medium Well Graded SAND With Gravel	SW							70
75	15	SS	8/18	8-15-15	Grey Medium Poorly Graded SAND	SP							75
80	16	SS	3/18	50/3	Grey, FINE, Poorly Graded GRAVEL With Sand	GP							80
					Auger Refusal in Shale at 80 FT.								
85													85
90													90

WATER LEVEL:

____ NO GROUND WATER NOTED AT TIME OF DRILLING
7 FT. WHILE DRILLING
7 FT. 24 HRS. AFTER DRILLING
____ FT. ____ HRS. AFTER DRILLING

REMARKS:

Figure 4.7c

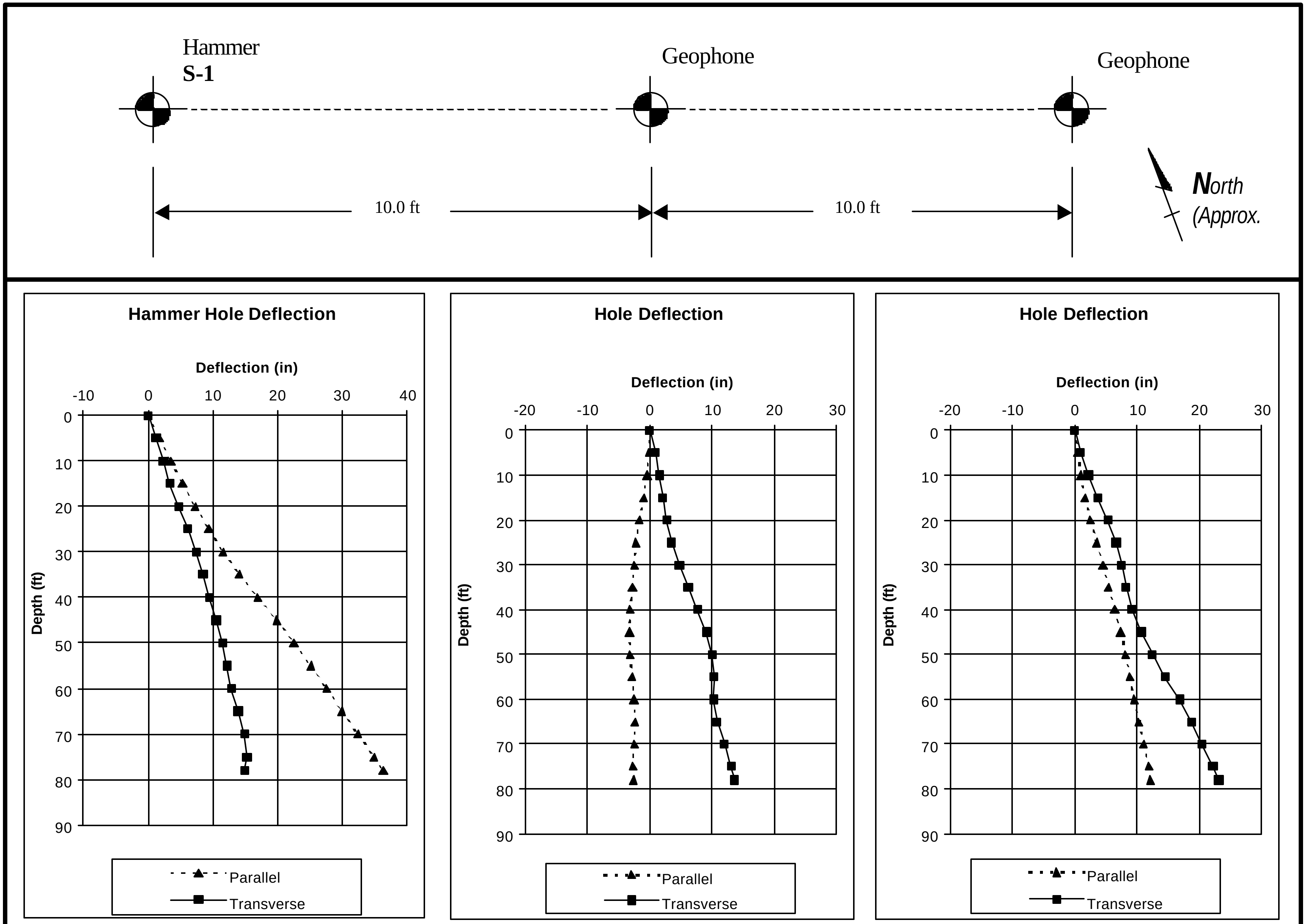


Figure 4.8. Boring layout and results of deviation survey (+Parallel=East, +Transverse=South) **HENDERSON S-1**

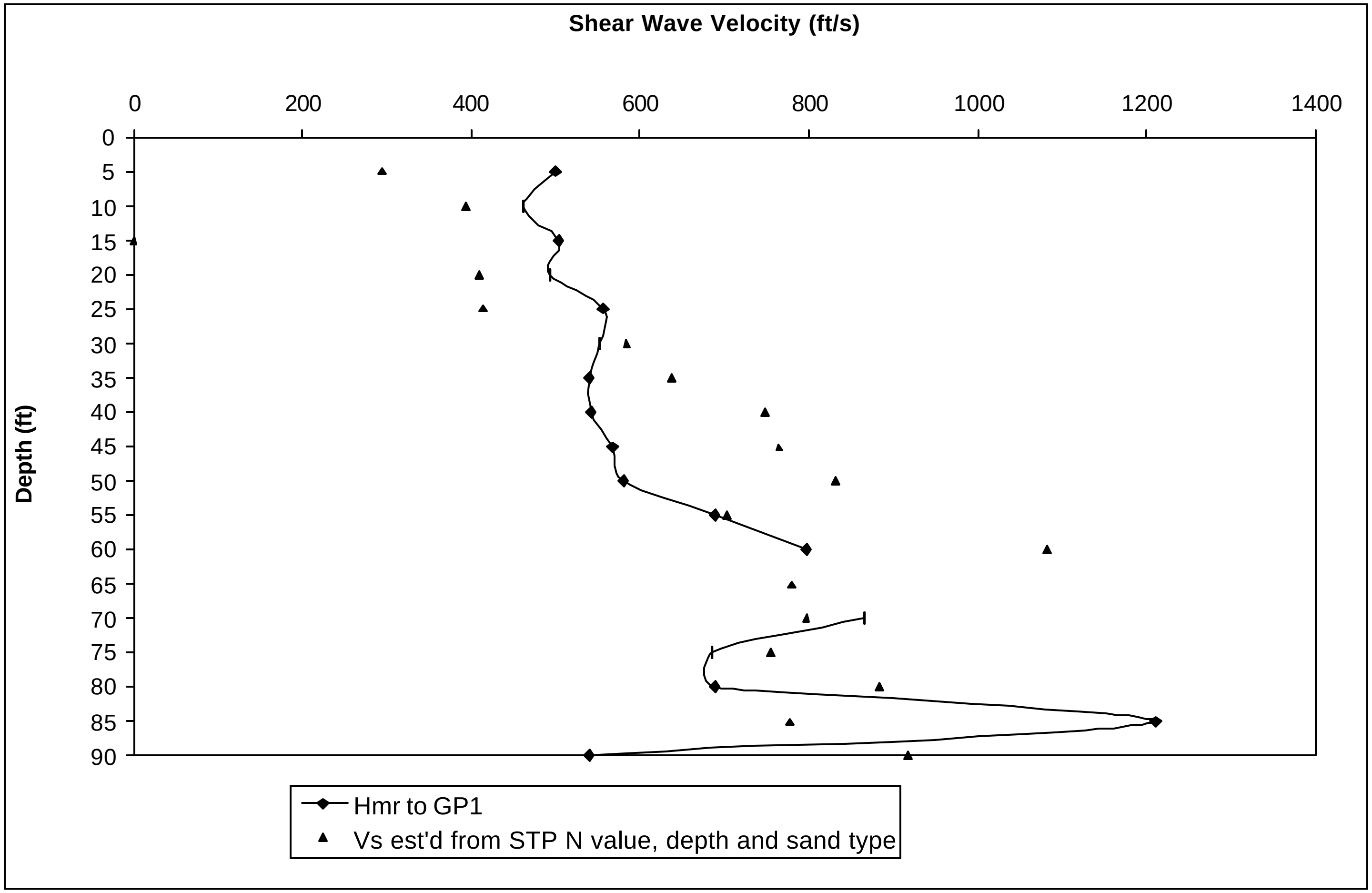


Figure 4.9. Crosshole shear wave velocities for the north abutment, along with estimated shear wave velocities based on SPT N values, and with interpreted upper and lower bounds for wave velocities, accounting for trigger delay during

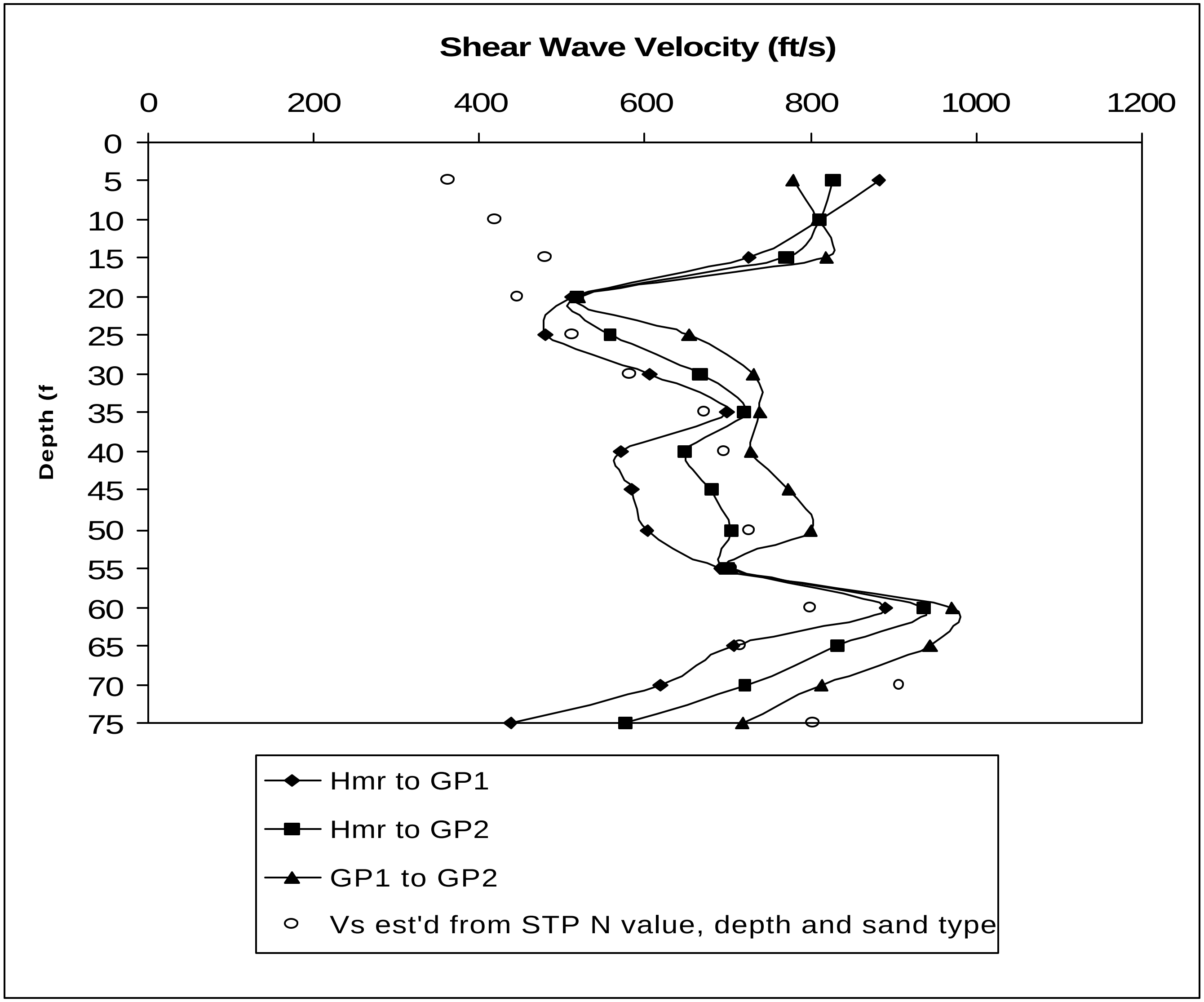


Figure 4.10. Crosshole shear wave velocities for the south abutment, along with estimated shear wave velocities based on SPT N values, and with interpreted upper and lower bounds for wave velocities, accounting for trigger delay during

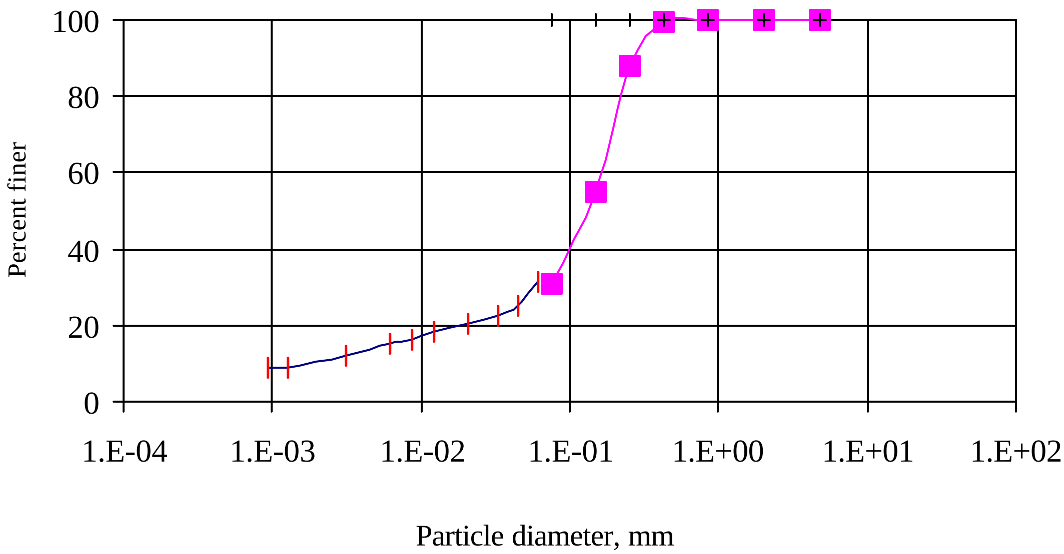


Figure 4.11a. Grain Size Distribution - N1, 24-25.5 ft.

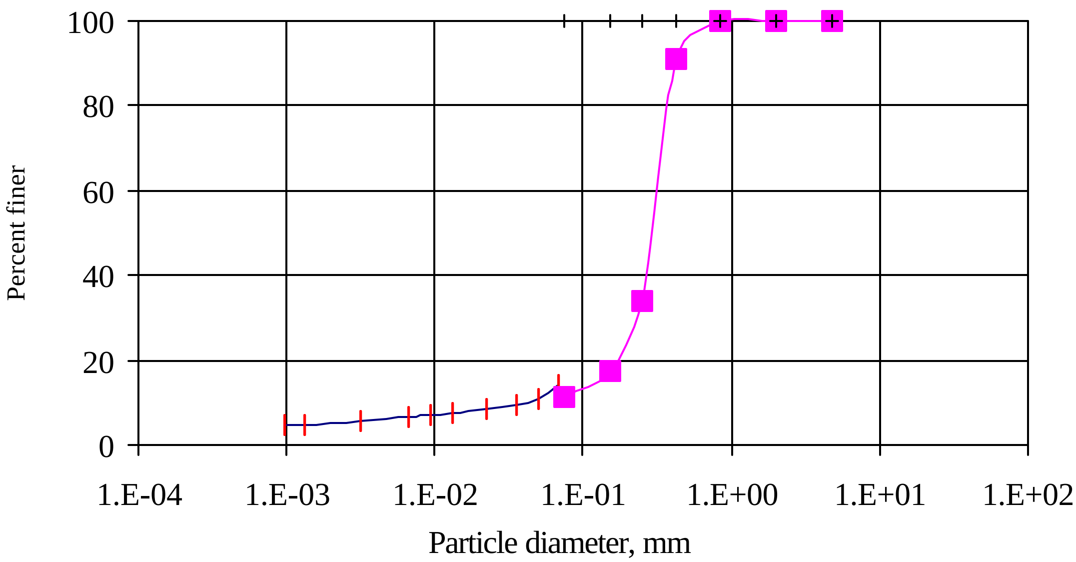


Figure 4.11b. Grain Size Distribution, Boring N1, 34-35.5 feet

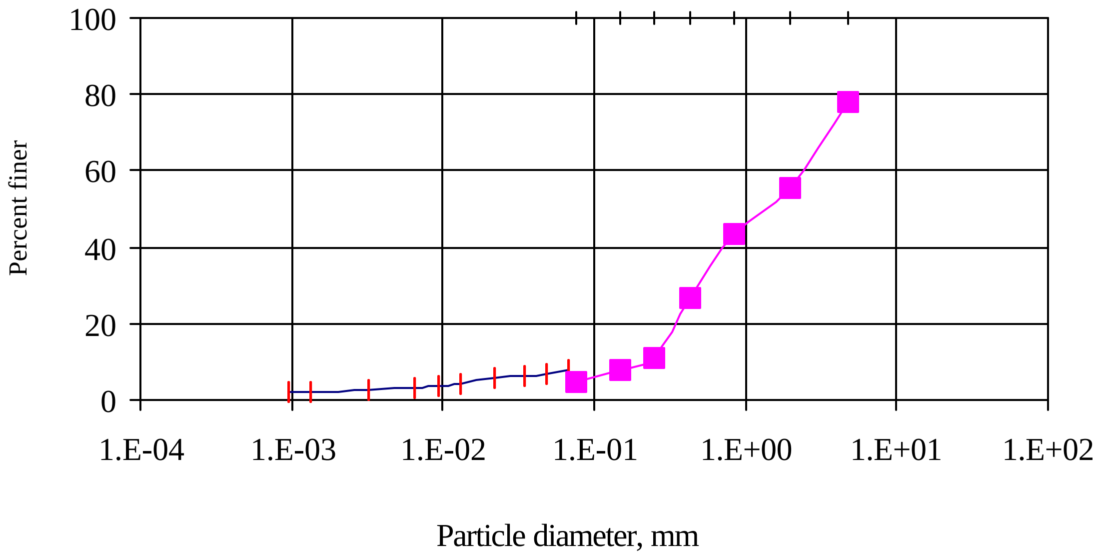


Figure 4-11c. Grain Size Distribution - N1, 54-55.5 ft

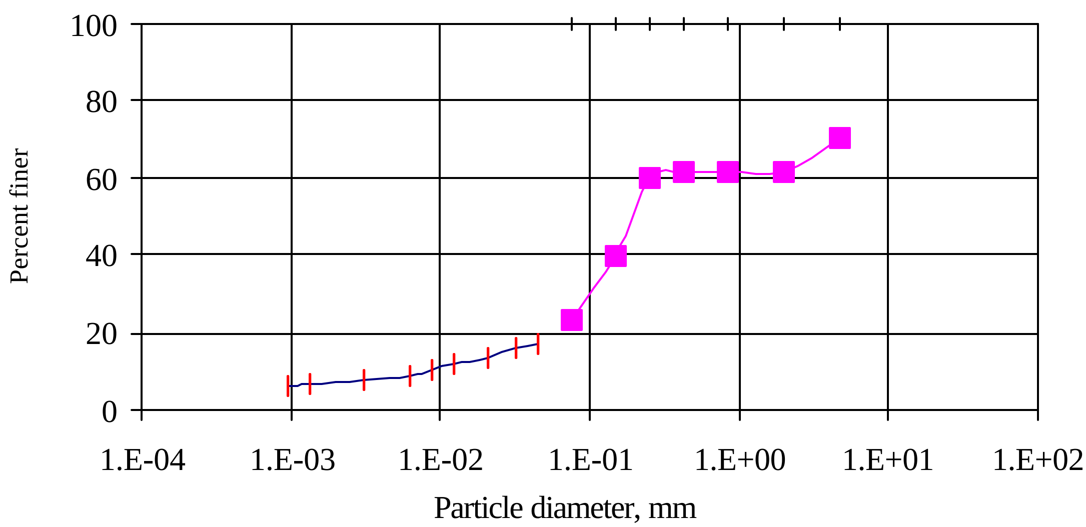


Figure 4-11d. Grain Size Distribution - S1, 18.5-20 ft.

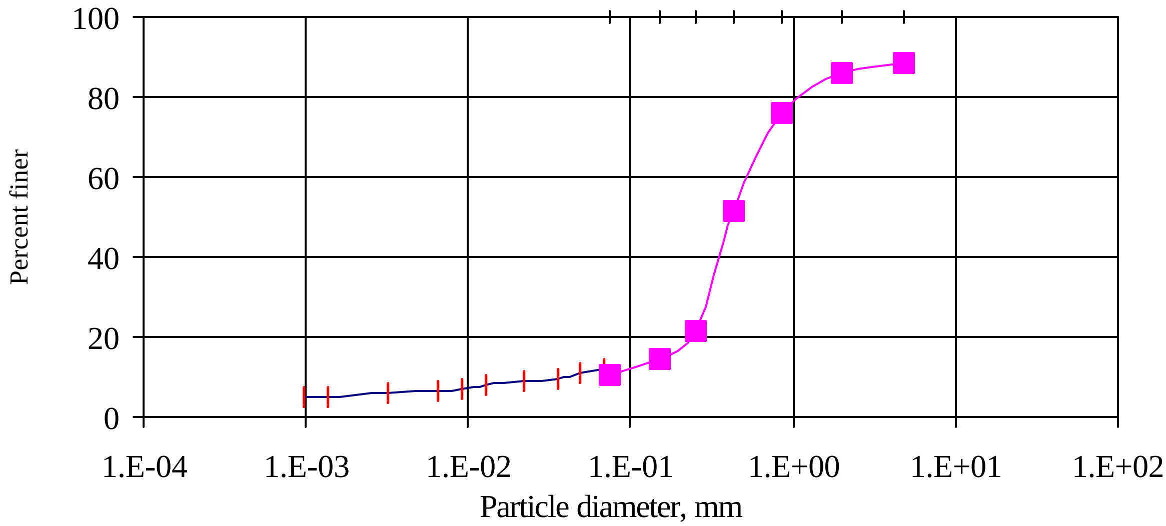


Figure 4-11e. Grain Size Distribution - S1, 38.5-40 ft.

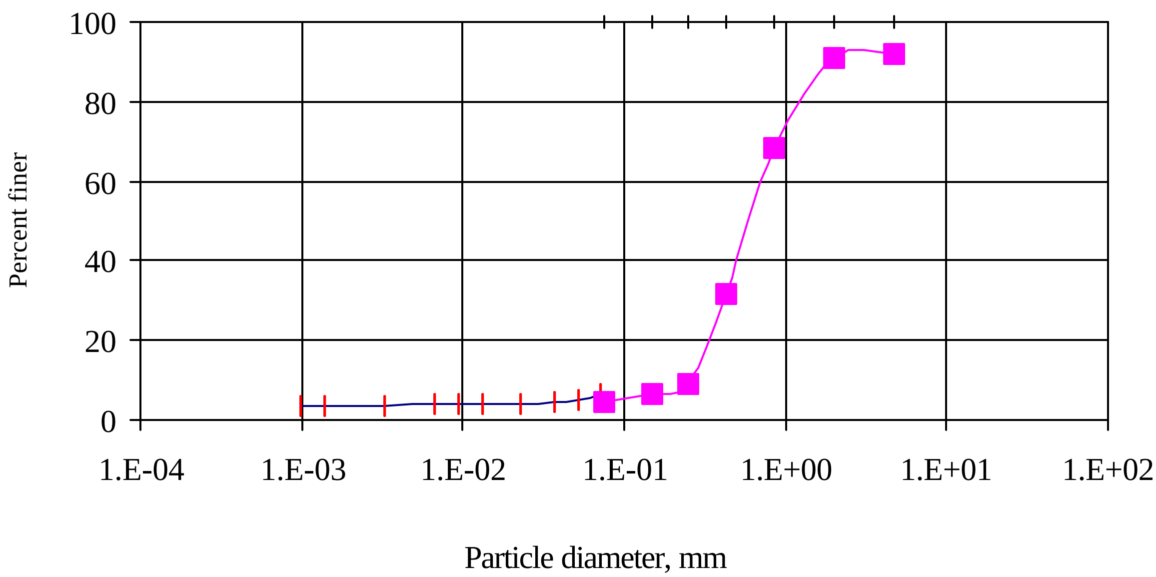


Figure 4-11f. Grain Size Distribution - S1, 48.5-50 ft.

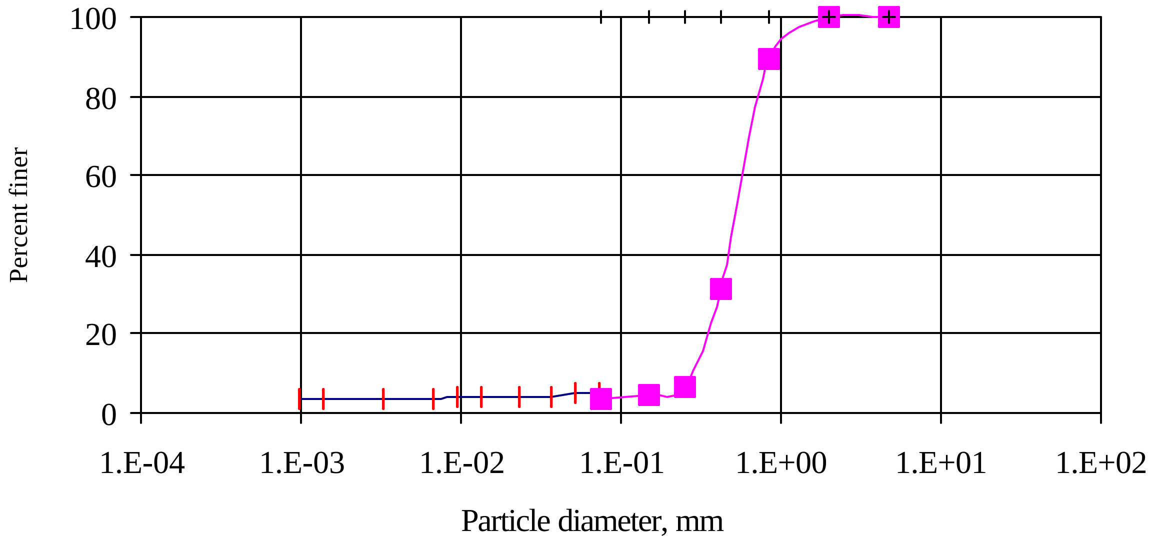


Figure 4-11g. Grain Size Distribution - S1, 53.5-55 ft.

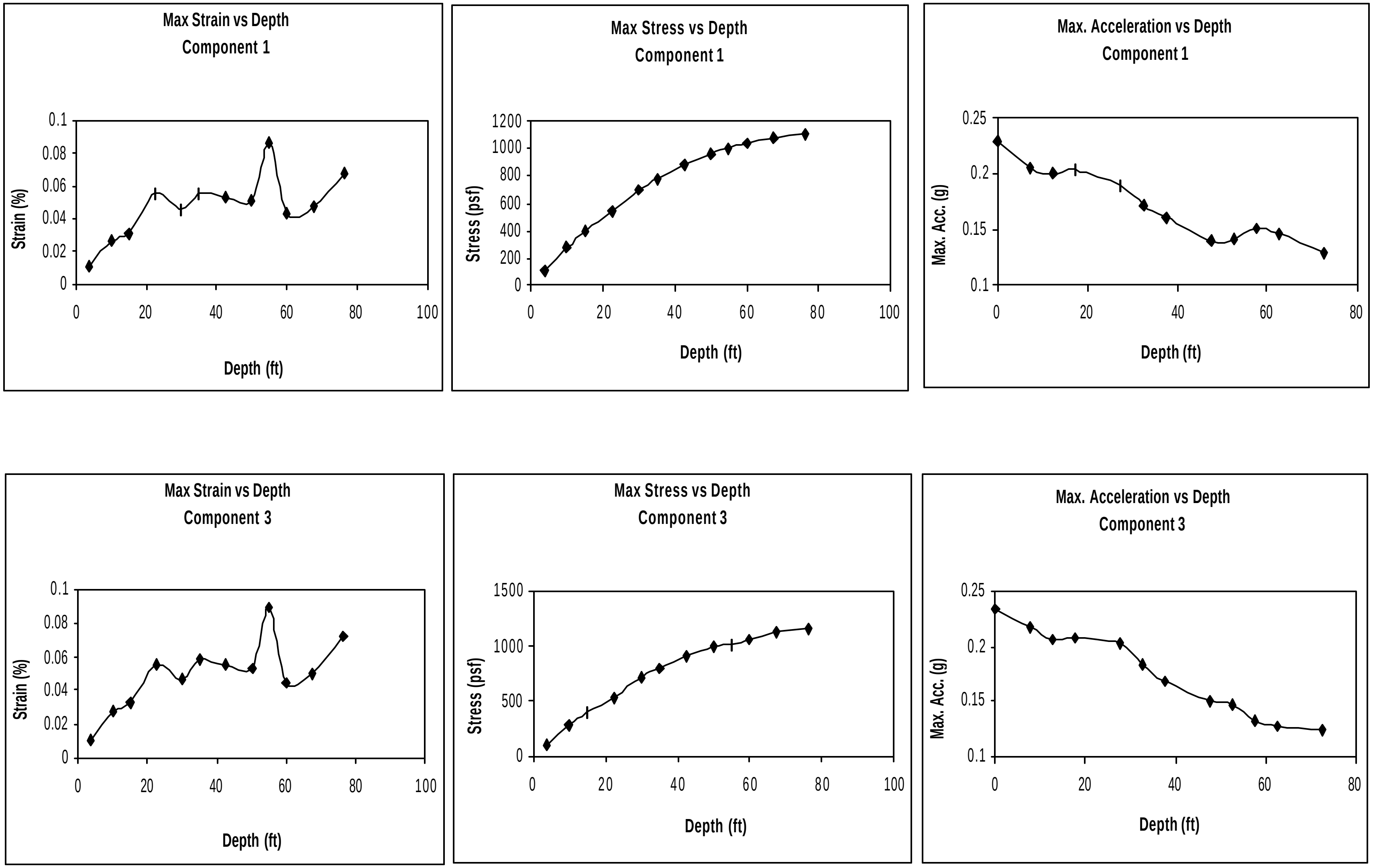


Figure 4.12. Maximum shear stress, shear strain and acceleration plots for the 500 year event.

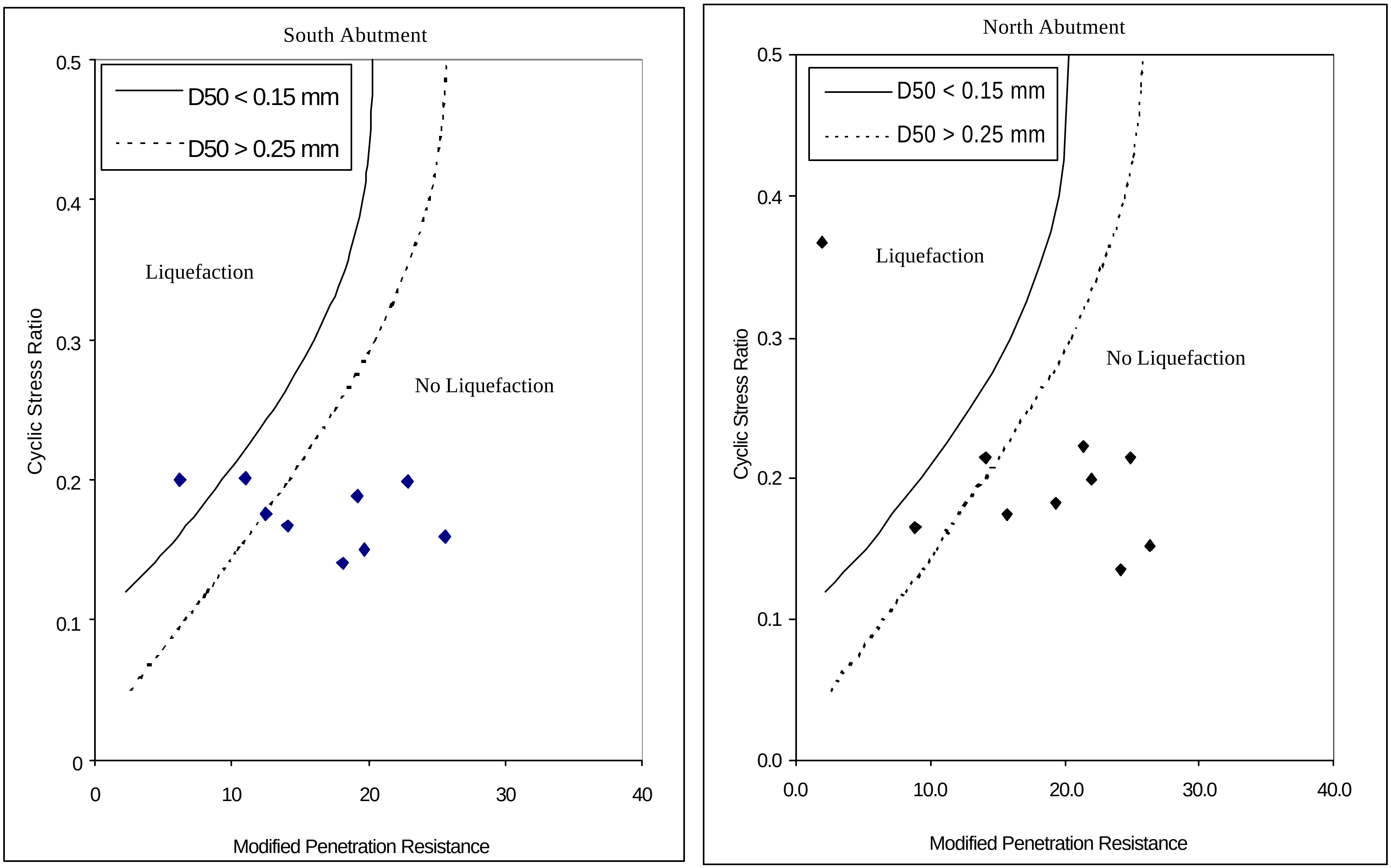


Figure 4.13. Liquefaction susceptibility for 500yr event field liquefaction behavior at U.S. 41 Ohio River

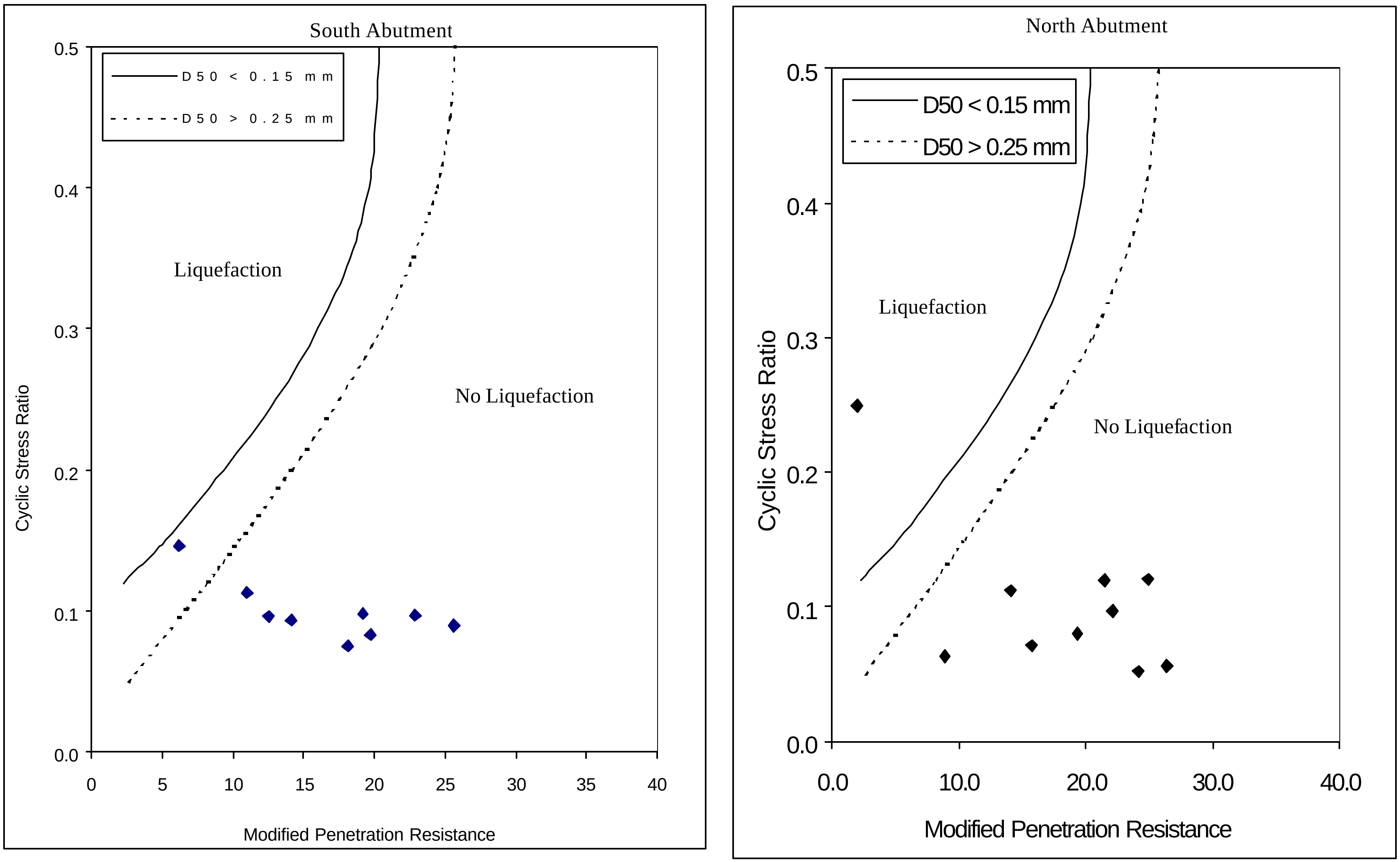


Figure 4.14. Liquefaction susceptibility for 50yr event field liquefaction behavior at U.S. 41 Ohio River Bridges, Henderson. Boundaries are for magnitude 6.3 event in NMSZ.

5. CONCLUSIONS AND RECOMMENDATIONS

This report is comprised of a summary of the efforts on three separate projects completed under the same contract. Conclusions and recommendations for each of the three projects are provided at the end of Chapters 2, 3, and 4.

The embankment rating completed using the KESR model was for ranking purposes only. The estimated factors of safety and estimated embankment deformations were obtained using a simplified model, information obtained from rapid field inspections, and subsurface surmised from USGS and SCS published data. These findings cannot substitute for a site-specific geotechnical evaluation.

The stability assessment for the US51 Ohio River crossing and the evaluation completed for the US41 twin spans crossing the Ohio River were both limited, as described in the respective chapters, by the initial project budget, as these were not part of the original project scope proposed. However, the findings should be adequate to use for planning. If detailed recommendations for either Ohio River crossings are desired, a detailed geotechnical investigation should be completed. Neither the US41 or US51 bridge foundations were evaluated. In both cases, the stability of the foundations may be critical for the 50 year or 500 year events. A geotechnical evaluation of these is recommended if a complete seismic evaluation of the crossings is desired.

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APPENDIX

RANKINGS OF WESTERN KENTUCKY EMBANKMENTS

500 YEAR EVENT

Ranking, along with estimated displacement or factor of safety for **500 year event**.										
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	500 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated disp., cm)	C/D (Factor of Safety)
Ballard County										
A	1	BA	121	6.5	Fill	63.2	24	0.044	277.8	
A	2	BA	121	6.7	S. of 62/60/51 Jntn	63.2	24	0.091	105.2	
A	3	BA	60	3.5	Bridge E	63.2	19	0.146	51.7	
A	4	BA	60	3.5	Bridge W	63.2	19	0.146	51.3	
A	5	BA	60	3.85	Fill	63.2	20	0.162	43.5	
A	6	BA	60	11.51	Bridge E	63.2	16	0.162	43.5	
A	7	BA	60	1.95	Bridge	63.2	15	0.174	38.5	
A	8	BA	60	11.55	Fill W	63.2	14	0.184	35.1	
A	9	BA	60	1	Fill	63.2	16	0.196	31.5	
A	10	BA	60	1.95	Bridge	63.2	15	0.203	29.5	
A	11	BA	60	3.1	Fill	63.2	14	0.206	28.7	
A	12	BA	60	1.25	Fill	63.2	13	0.220	25.7	
A	13	BA	121	3.15	Stovall Crk Bridge North	63.2	11	0.221	25.3	
A	14	BA	60	3.8	Fill	63.2	13	0.223	25.0	
A	15	BA	60	3.93	Culvert Bridge	63.2	10	0.228	24.0	
A	16	BA	60	11.85	Bridge E	63.2	23	0.228	23.9	
A	17	BA	60	11.85	Bridge W	63.2	23	0.231	23.4	
A	18	BA	60	11.3	Culvert	63.2	9	0.234	22.8	
A	19	BA	60	10.23	Fill (Bridge Fill) W	63.2	15	0.235	22.7	
A	20	BA	60	1.94	Fill	63.2	9	0.235	22.6	
A	21	BA	60	1.15	Fill	63.2	12	0.235	22.6	
A	22	BA	60	0.8	Fill	63.2	12	0.237	22.2	
A	23	BA	60	3.68	Fill	63.2	12	0.237	22.2	
A	24	BA	60	10.23	Fill (Bridge Fill) E	63.2	15	0.246	20.7	
A	25	BA	60	2.9	Fill	63.2	10	0.250	20.0	
A	26	BA	60	2.3	Fill	63.2	7	0.253	19.6	
A	27	BA	60	5.74	Fill (Bridge Fill) W	63.2	15	0.255	19.3	
A	28	BA	60	2.55	Fill	63.2	10	0.262	18.3	
A	29	BA	60	0.75	Fill	63.2	10	0.263	18.2	
A	30	BA	60	5.74	Fill (Bridge Fill) E	63.2	15	0.264	17.9	
A	31	BA	60	5.32	Bridge	63.2	11	0.264	17.9	
A	32	BA	60	5.45	Fill	63.2	11	0.264	17.9	
A	33	BA	121	0	Mayfield Crk Bridge South	63.2	8	0.270	17.1	
A	34	BA	121	0	Mayfield Crk Bridge North	63.2	8	0.270	17.1	
A	35	BA	60	3.5	Fill	63.2	8	0.274	16.6	
B	1	BA	121	7.3	S. of 62/60/51 Jntn	63.2	28	0.517	3.0	
C	1	BA	121	7.7	S. of 62/60/51 Jntn	63.2	23			1.0
C	2	BA	121	7.7	S. of 62/60/51 Jntn	63.2	16			1.1
Z		BA	121	3.15	Stovall Crk Bridge South	63.2	10			
Z		BA	121	5.3	Shelton Crk Bridge South	63.2	8			
Z		BA	121	5.3	Shelton Crk Bridge North	63.2	8			
Z		BA	60	12.5	Bridges & Fills (<=5')	63.2				

Ranking, along with estimated displacement or factor of safety for **500 year event**.

Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	500 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated disp., cm)	C/D (Factor of Safety)
Z		BA	60	1.85	Fill (<=5')	63.2				
Caldwell County										
C	1	CD	62	3	Fill	17.8	26			1.9
C	2	CD	62	2	Fill	17.8	19			2.3
C	3	CD	62	1.1	Fill	17.8	14			3.2
C	4	CD	62	0.65	Fill	17.8	14			3.3
Christian County										
C	1	CH	68/80	18.18	Bridge W	14.4	21			1.5
C	2	CH	91	4.43	Bridge N	14.4	20			1.5
C	3	CH	68/80	18.18	Bridge E	14.4	21			1.7
C	4	CH	91	4.43	Bridge S	14.4	20			1.7
C	5	CH	68/80	3.5	Fill	14.4	25			2.2
C	6	CH	68/80	3.6	Fill	14.4	25			2.2
C	7	CH	68/80	11.2	Pennyrile Pkwy Bridge	14.4	21			2.5
C	8	CH	68/80	4.65	Fill	14.4	16			3.2
C	9	CH	68/80	4.68	Muddy Branch Bridge	14.4	16			3.2
C	10	CH	91	11.26	Bridge N	14.4	18			3.6
C	11	CH	68/80	10.76	Bridge W	14.4	28			3.7
C	12	CH	91	13.7	Bridge S	14.4	12			5.3
C	13	CH	91	11.26	Bridge S	14.4	18			5.9
C	14	CH	91	13.7	Bridge N	14.4	12			6.3
C	15	CH	91	2.16	Bridge N	14.4	5			6.8
Z		CH	68/80	10.76	Bridge E	14.4	28			
Z		CH	68/80	3.5	App Fill Ltl Sinking Fk Crk	14.4				
Z		CH	68/80	4.65	Muddy Fk Crk	14.4				
Z		CH	68/80	10.76	N Fk Little Rvr Drdg	14.4				
Z		CH	68/80	11.4	Pennyrile Brdg	14.4				
Z		CH	68/80	18.18	Brdg S Fk Little Rvr	14.4				
Z		CH	68/80	3.56	Bridge	14.4				
Z		CH	68/80	4.75	Fill	14.4				
Z		CH	68/80	11.4	Fill	14.4				
Z		CH	91	2.16	Bridge S	14.4				
Carlisle County										
A	1	CL	121	1.2	Fill	60.8	18	0.178	37.3	
A	2	CL	121	4.8	Fill	60.8	17	0.178	37.2	
A	3	CL	121	0.05	Fill	60.8	18	0.181	36.1	
A	4	CL	121	0.4	Fill	60.8	17	0.189	33.7	
A	5	CL	121	4.2	Fill	60.8	16	0.195	31.9	
A	6	CL	62	3.88	Fill Bridge Fill E	60.8	16	0.198	30.9	
A	7	CL	121	2.85	Fill	60.8	16	0.203	29.7	
A	8	CL	121	4.9	Fill	60.8	15	0.204	29.5	
A	9	CL	121	5.1	Fill	60.8	15	0.207	28.6	
A	10	CL	121	0.2	Fill	60.8	15	0.212	27.5	
A	11	CL	121	2.51	Fill	60.8	14	0.224	24.7	

Ranking, along with estimated displacement or factor of safety for **500 year event**.										
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	500 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated disp., cm)	C/D (Factor of Safety)
A	12	CL	62	3.88	Fill Bridge Fill W	60.8	12	0.246	20.8	
A	13	CL	121	9.38	Brdge South	60.8	11	0.256	19.2	
A	14	CL	121	9.38	Brdge North	60.8	12	0.257	18.9	
A	15	CL	121	2.5	Fill	60.8	10	0.266	17.7	
A	16	CL	121	4.5	Fill	60.8	10	0.267	17.5	
A	17	CL	121	9.1	Brdge South	60.8	10	0.269	17.2	
A	18	CL	121	9.1	Brdge North	60.8	9	0.281	15.7	
A	19	CL	62	6.04	Bridge	60.8	6	0.300	13.7	
Calloway County										
A	1	CW	641	14.45	Fill	27.1	39	0.052	222.1	
A	2	CW	94	24	Fill	27.1	43	0.280	15.9	
B	1	CW	94	17.1	Jonathan Creek W	27.1	20	0.366	8.6	
B	2	CW	641	15.7	Fill	27.1	18	0.369	8.4	
B	3	CW	94	11.07	Fill	27.1	21	0.373	8.2	
B	4	CW	94	11.44	Clarks River Bridge	27.1	27	0.373	8.2	
B	5	CW	641	5.5	Fill	27.1	15	0.415	6.1	
B	6	CW	94	17.1	Jonathan Creek E	27.1	17	0.420	5.9	
B	7	CW	94	11.3	Clarks River Bridge	27.1	23	0.454	4.7	
B	8	CW	641	15.85	Fill	27.1	16	0.456	4.6	
B	9	CW	641	8.92	Fill	27.1	15	0.471	4.1	
B	10	CW	641	5.65	Fill	27.1	15	0.474	4.1	
B	11	CW	641	1.1	Fill	27.1	15	0.474	4.1	
B	12	CW	94	16.5	Elm Grove Bridge(Jonathan Crk) E	27.1	15	0.474	4.1	
B	13	CW	121	23.9	0.91 mi S. of CW-GR county line	27.1	13	0.509	3.2	
B	14	CW	94	23.03	Fill E	27.1	13	0.526	2.8	
B	15	CW	641	15.6	Fill	27.1	12	0.535	2.6	
B	16	CW	641	8.95	Fill	27.1	12	0.542	2.5	
B	17	CW	641	8.9	Fill	27.1	11	0.543	2.5	
B	18	CW	94	11.4	Fill	27.1	23	0.549	2.4	
B	19	CW	94	23.03	Fill W	27.1	13	0.569	2.0	
B	20	CW	94	1.77	Williams Creek E	27.1	11	0.581	1.9	
B	21	CW	94	11.1	Clarks River Bridge	27.1	21	0.586	1.8	
B	22	CW	121	23.5	1.31 mi S. of CW-GR county line	27.1	10	0.596	1.7	
B	23	CW	94	16.5	Elm Grove Bridge(Jonathan Crk) W	27.1	15	0.599	1.6	
C	1	CW	641	1.2	Fill	27.1	15			1.0
C	2	CW	641	5.75	Fill	27.1	17			1.0
C	3	CW	641	5.6	Fill	27.1	15			1.0
C	4	CW	94	1.77	Williams Creek W	27.1	11			1.0
C	5	CW	94	6.44	Haynes Creek E	27.1	15			2.1
C	6	CW	121	20.6	0.97 mi S. of W. Fk. of ClarksRiver	27.1	12			2.6
C	7	CW	94	6.44	Haynes Creek W	27.1	14			2.6
Z		CW	641	5.49	Bridge	27.1	15			
Z		CW	641	1.15	Bridge	27.1	15			
Z		CW	121	21.57	App. Fills for W. Fk. of Clarks	27.1	11			

Ranking, along with estimated displacement or factor of safety for **500 year event**.										
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	500 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated disp., cm)	C/D (Factor of Safety)
					River					
Z		CW	641	15.81	Bridge	27.1				
Z		CW	641	15.65	Bridge	27.1				
Z		CW	641	5.66	Bridge	27.1				
Fulton County										
A	1	FU	94	29.15	Fill	58.7	29	0.038	332.1	
A	2	FU	94	29.2	Fill	58.7	28	0.092	102.3	
A	3	FU	94	17.85	Bridge	58.7	18	0.220	25.7	
A	4	FU	94	17.22	Bridge E	58.7	17	0.238	22.0	
A	5	FU	94	17.7	Fill	58.7	18	0.240	21.7	
A	6	FU	94	17.22	Bridge W	58.7	17	0.241	21.5	
A	7	FU	166	8.9	Fill	58.7	14	0.249	20.3	
A	8	FU	166	9.03	Bridge (Jnct 1125)	58.7	14	0.2847	15.4	
Z		FU	94	24.04	Bridge @ Flood Stage	58.7	13			
Z		FU	166	2.09	Culvert	58.7	15			
Z		FU	94	25.52	Bridge	58.7	14			
Z		FU	94	24.22	Bridge	58.7	10			
Graves County										
A	1	GR	58/80	6.68	Mayfield Crk Bridge 3 E	41.3	24	0.043	284.6	
A	2	GR	58/80	6.68	Mayfield Crk Bridge 3 W	41.3	24	0.048	247.8	
A	3	GR	121	20.19	ICRR App. Fills South	41.3	29	0.055	210.3	
A	4	GR	121	20.19	ICRR App. Fills North	41.3	29	0.061	179.8	
A	5	GR	121	11.73	Mayfield Bypass App. Fill South	41.3	29	0.087	111.3	
A	6	GR	121	7.96	Mayfield Crk Bridge North	41.3	24	0.099	92.3	
A	7	GR	94	2.1	Fill	41.3	24	0.121	68.6	
A	8	GR	58	5.3	Fill	41.3	25	0.141	54.4	
A	9	GR	58	5.1	Fill	41.3	25	0.179	36.9	
A	10	GR	94	4.8	Fill	41.3	20	0.192	32.5	
A	11	GR	121	22.3	0.3 mi S. of Co. Line	41.3	21	0.204	29.4	
A	12	GR	121	21.7	0.9 mi S. of Co. Line	41.3	21	0.209	28.1	
A	13	GR	121	7.96	Mayfield Crk Bridge South	41.3	17	0.234	22.8	
A	14	GR	121	19.2	N. of Jntn of 1213/121	41.3	19	0.237	22.2	
A	15	GR	121	11.73	Mayfield Bypass App. Fill North	41.3	19	0.243	21.1	
A	16	GR	58/80	6.68	Mayfield Crk Bridge 4 W	41.3	12	0.245	20.8	
A	17	GR	58/80	6.68	Mayfield Crk Bridge 2 E	41.3	12	0.245	20.8	
A	18	GR	121	6.68	Mayfield Crk Bridge 1 W	41.3	12	0.245	20.8	
A	19	GR	58/80	6.68	Mayfield Crk Bridge 1 E	41.3	12	0.245	20.8	
A	20	GR	58/80	6.68	Mayfield Crk Bridge 2 W	41.3	12	0.245	20.8	
A	21	GR	121	22	0.6 mi S. of Co. Line	41.3	17	0.269	17.3	
A	22	GR	94	4.3	Fill	41.3	21	0.284	15.4	
A	23	GR	45	1.8	Jackson Creek	41.3	15	0.292	14.5	
A	24	GR	121	16.1	0.8 mi N. of 440/121	41.3	15	0.308	13.0	
A	25	GR	94	1.68	Fill	41.3	15	0.309	12.8	
A	26	GR	94	2.9	Bridge	41.3	15	0.311	12.6	

Ranking, along with estimated displacement or factor of safety for **500 year event**.

Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	500 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated disp., cm)	C/D (Factor of Safety)
A	27	GR	94	2.96	Fill	41.3	15	0.311	12.6	
A	28	GR	121	18.8	0.3 mi S. of Jntn of 1213/121	41.3	15	0.311	12.6	
A	29	GR	121	8.27	Mayfield Crk Branch Brdg North	41.3	11	0.323	11.6	
A	30	GR	121	8.27	Mayfield Crk Branch Brdg South	41.3	11	0.323	11.6	
A	31	GR	94	4.9	Fill	41.3	13	0.329	11.1	
A	32	GR	58/80	12.25	Panther Crk App. Fills W	41.3	13	0.330	11.1	
A	33	GR	45	6.09	Bridge W	41.3	19	0.344	10.0	
A	34	GR	58/80	12.25	Panther Crk App. Fills E	41.3	13	0.345	9.9	
A	35	GR	45	10.54	Bridge W	41.3	23	0.346	9.9	
B	1	GR	45	1.68	Bayou Chien Creek	41.3	19	0.352	9.5	
B	2	GR	121	8.75	Kess Crk Bridge South	41.3	9	0.357	9.2	
B	3	GR	58	0.51	Culvert	41.3	14	0.361	8.9	
B	4	GR	58/80	6.68	Mayfield Crk Bridge 4 E	41.3	12	0.362	8.8	
B	5	GR	94	1.85	Fill	41.3	24	0.362	8.8	
B	6	GR	45	7.86	Bridge E	41.3	16	0.365	8.6	
B	7	GR	45	13.1	Richard Creek Bridge	41.3	14	0.365	8.6	
B	8	GR	121	8.14	Mayfield Crk Overfl. Brdg North	41.3	11	0.366	8.6	
B	9	GR	121	5.2	N. of Ky 1890/121	41.3	11	0.375	8.0	
B	10	GR	45	6.09	Bridge E	41.3	19	0.380	7.8	
B	11	GR	58	0.51	Culvert	41.3	14	0.383	7.6	
B	12	GR	121	8.14	Mayfield Crk Overfl. Brdg South	41.3	10	0.384	7.5	
B	13	GR	45	7.86	Bridge EE	41.3	16	0.387	7.4	
B	14	GR	58	3.9	Fill	41.3	10	0.388	7.4	
B	15	GR	58/80	12.44	Panther Crk Fork App. Fills W	41.3	10	0.390	7.3	
B	16	GR	45	7.8	Bridge W	41.3	16	0.392	7.1	
B	17	GR	58/80	12.44	Panther Crk Fork App. Fills E	41.3	10	0.393	7.1	
B	18	GR	45	10.54	Bridge E	41.3	23	0.395	7.0	
B	19	GR	58/80	9.9	1.7 mi. E. of Juntn 131 & 58/80	41.3	9	0.398	6.9	
B	20	GR	94	2	Bridge E	41.3	15	0.401	6.7	
B	21	GR	58	2.83	Culvert	41.3	14	0.404	6.6	
B	22	GR	58	2.83	Culvert	41.3	14	0.407	6.5	
B	23	GR	121	8.75	Kess Crk Bridge North	41.3	5	0.410	6.3	
B	24	GR	94	2	Bridge W	41.3	15	0.411	6.3	
B	25	GR	45	12.2	Opossum Creek Bridge	41.3	14	0.413	6.2	
B	26	GR	45	12.2	Opossum Creek Bridge	41.3	14	0.437	5.3	
B	27	GR	45	1.8	Jackson Creek	41.3	14	0.439	5.2	
C	1	GR	58/80	13.7	W. of Juntn KY 564/KY 58/80	41.3	20			1.3
C	2	GR	121	17.7	2.4 mi N. of 440/121	41.3	18			1.6
C	3	GR	94	3.7	Fill	41.3	15			1.7
C	4	GR	94	15.4	Fill (near 83)	41.3	14			1.8
C	5	GR	58/80	13.8	0	41.3	13			2.2
C	6	GR	121	1.9	0.4 mi S. of Jntn 564/94	41.3	5			2.8
Z		GR	94	0.19	Bridges	41.3	13			
Z		GR	94	0.19	Bridges	41.3	9			

Ranking, along with estimated displacement or factor of safety for **500 year event**.										
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	500 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated disp., cm)	C/D (Factor of Safety)
Z		GR	94	0.19	Bridges	41.3	7			
Z		GR	94	2.01	Bridge	41.3				
Z		GR	94	0.24	RR Underpass Bridge W	41.3				
Z		GR	58	5.27	Bridge	41.3				
Hickman County										
A	1	HI	58	19.85	Fill	60.5	30	0.184	35.1	
A	2	HI	94	16	Fill	60.5	17	0.238	22.1	
A	3	HI	58	19.7	Fill	60.5	30	0.260	18.6	
A	4	HI	94	16	Fill	60.5	17	0.261	18.4	
Z		HI	94	15.87	Bridge	60.5	17			
Z		HI	45	1.2	Fill & Tied-In Piles	60.5				
Z		HI	58	19.82	RR Bridge	60.5				
Livingston County										
A	1	LI	62/641	0.31	KY Lake Bridge	25.4	25	0.061	183.1	
A	2	LI	62/641	2.75	Fill	25.4	21	0.344	10.0	
C	1	LI	62/641	0.97	Quarry Rd Bridge W	25.4	30			1.3
C	2	LI	62/641	0.97	Quarry Rd Bridge E	25.4	30			1.3
C	3	LI	62/641	0.64	RR Bridge W	25.4	28			3.1
C	4	LI	62/641	0.64	RR Bridge E	25.4	28			3.1
Z		LI	62/641	2.78	Bridge	25.4				
Logan County										
C	1	LO	68/80	21.7	Fill	9.7	8			2.0
C	2	LO	68/80	9.6	RR Bridge	9.7	24			2.4
C	3	LO	68/80	9.64	Fill	9.7	24			2.5
C	4	LO	68/80	12.5	Fill	9.7	18			3.4
C	5	LO	68/80	15.1	Fill	9.7	16			3.7
C	6	LO	68/80	2.8	Fill Bridge Fill W	9.7	25			4.2
C	7	LO	68/80	20.94	Bridge W	9.7	17			4.5
C	8	LO	68/80	2.8	Fill Bridge Fill E	9.7	11			5.1
C	9	LO	68/81	25.75	Bridge E	9.7	17			5.7
C	10	LO	68/80	21.94	Bridge E	9.7	17			6.0
C	11	LO	68/80	24.75	Bridge W	9.7	17			6.0
Z		LO	68/80	10.33	Bridge	9.7	8			
Z		LO	68/80	2.8	Whip-will Crk Brdg	9.7				
Z		LO	68/80	9.64	L&N RR Brdg	9.7				
Z		LO	68/80	10.33	Town Brnch E Fk	9.7				
Z		LO	68/80	12.5	E Russ Point Brnch	9.7				
Z		LO	68/80	15.1	3.2m W 722 N Jctn	9.7				
Z		LO	68/80	20.94	Blk Lick Crk Brdg	9.7				
Z		LO	68/80	21.7	W L&N RR	9.7				
Z		LO	68/80	21.95	E L&N RR	9.7				
Z		LO	68/80	21.91	Bridge	9.7				
Z		LO	68/80	21.95	Fill (<5')	9.7				
Lyon County										

Ranking, along with estimated displacement or factor of safety for **500 year event**.										
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	500 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated disp., cm)	C/D (Factor of Safety)
A	1	LY	62/641	4.8	Fill	20.1	43	0.124	66.8	
A	2	LY	62/641	4.5	Fill	20.1	49	0.217	26.3	
A	3	LY	62/641	7.3	Fill	20.1	26	0.256	19.1	
A	4	LY	62/641	8.25	Fill	20.1	35	0.289	14.9	
B	1	LY	62/641	7.5	Fill	20.1	27	0.551	2.3	
C	1	LY	62/641	10.35	Fill	20.1	15			1.0
C	2	LY	62/641	9.2	Fill	20.1	24			1.0
C	3	LY	62/641	7.8	Fill	20.1	23			1.0
C	4	LY	62/641	1.65	Fill	20.1	15			1.0
C	5	LY	62/641	7.4	Fill	20.1	23			1.0
C	6	LY	62/641	7.6	Fill	20.1	23			1.0
C	7	LY	62/641	1.1	Fill	20.1	15			1.0
C	8	LY	62	12.8	Bridge	20.1	12			1.1
C	9	LY	62/641	9.65	Fill	20.1	16			1.1
C	10	LY	62/641	3.67	Fill	20.1	11			1.1
C	11	LY	62/641	9.85	Fill	20.1	14			1.2
C	12	LY	62/641	9.95	Fill	20.1	13			1.2
C	13	LY	62	13.4	Fill	20.1	7			1.2
C	14	LY	62	11.45	Fill	20.1	15			2.8
Z		LY	62/641	7.3	Fill	20.1				
Z		LY	62	11.6	Bridge	20.1				
Z		LY	62	12.2	Fill	20.1				
Marshall County										
A	1	MA	62	10.9	Fill	27.2	40	0.035	380.5	
A	2	MA	62	2.47	Culvert	27.2	27	0.083	119.5	
A	3	MA	62	2.55	Fill	27.2	27	0.085	116.0	
A	4	MA	80	9.9	App Fill Clark Rvr Brdg E	27.2	26	0.091	105.1	
A	5	MA	68	9.43	App Fill Jack Purch Pkwy Brdge W	27.2	25	0.206	28.9	
A	6	MA	58/80	1.12	App Fill W Fk Clarks Rvr Brdg E	27.2	24	0.219	25.8	
A	7	MA	58/80	1.12	App Fill W Fk Clarks Rvr Brdg W	27.2	24	0.219	25.8	
A	8	MA	94	0.4	Fill	27.2	27	0.220	25.7	
A	9	MA	68	9.43	App Fill Jack Purch Pkwy Brdge E	27.2	23	0.230	23.6	
A	10	MA	641	9.4	Bridge	27.2	23	0.248	20.3	
A	11	MA	641	9.45	Fill	27.2	23	0.248	20.3	
A	12	MA	68/80	27.8	App Fill Tenn Rvr Brdg	27.2	24	0.272	16.9	
A	13	MA	80	9.9	App Fill Clark Rvr Brdg W	27.2	26	0.282	15.7	
A	14	MA	68	22.48	App Fill Jon Crk Brdg East	27.2	22	0.283	15.6	
A	15	MA	68	22.48	App Fill Jon Crk Brdg West	27.2	23	0.283	15.5	
A	16	MA	80	10.4	0.5 mi E Clark Rvr Brdg	27.2	19	0.312	12.6	
A	17	MA	80	12.52	App Fill Jonathan Crk Brdg W	27.2	43	0.322	11.7	
A	18	MA	62	9.45	Fill	27.2	20	0.323	11.6	
A	19	MA	62	9.55	Fill	27.2	20	0.324	11.5	
A	20	MA	641	0.2	Fill	27.2	18	0.332	10.9	
B	1	MA	80	12.52	App Fill Jonathan Crk Brdg E	27.2	46	0.362	8.8	

Ranking, along with estimated displacement or factor of safety for **500 year event**.										
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	500 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated disp., cm)	C/D (Factor of Safety)
B	2	MA	80	10.9	1 mi E Clark Rvr Brdg	27.2	16	0.390	7.3	
B	3	MA	62	10.87	Fill	27.2	17	0.398	6.9	
B	4	MA	62	10.87	Fill	27.2	17	0.412	6.2	
B	5	MA	80	9.4	0.4 mi. west of Clark R. Brdg E.	27.2	10	0.425	5.7	
B	6	MA	80	9.4	0.4 mi. west of Clark R. Brdg W	27.2	10	0.425	5.7	
B	7	MA	94	1.6	Terrapin Creek Fill	27.2	15	0.473	4.1	
B	8	MA	80	8.4	Bridge Approach Fill	27.2	7	0.480	3.9	
B	9	MA	68	24.85	0.95mi W 68/80 Jnct	27.2	12	0.480	3.9	
B	10	MA	62	3.6	Fill	27.2	14	0.486	3.7	
B	11	MA	62	0.7	Fill	27.2	14	0.488	3.7	
B	12	MA	641	5.85	Culvert @ 1518	27.2	16	0.489	3.7	
B	13	MA	62	1.2	Fill	27.2	14	0.492	3.6	
B	14	MA	80	8.8	0	27.2	8	0.525	2.8	
B	15	MA	80	8.8	0	27.2	8	0.534	2.6	
B	16	MA	80	15.06	App Fill Clear Crk Brdg E	27.2	12	0.537	2.6	
B	17	MA	80	15.06	App Fill Clear Crk Brdg W	27.2	12	0.549	2.4	
B	18	MA	62	1.7	Fill	27.2	10	0.593	1.7	
B	19	MA	641	9.94	Fill	27.2	9	0.607	1.5	
B	20	MA	641	9.8	Fill	27.2	11	0.612	1.5	
B	21	MA	80	12.1	2.2mi E Clrk Rvr Brdg W	27.2	9	0.618	1.4	
B	22	MA	80	12.1	2.2mi E Clrk Rvr Brdg E	27.2	9	0.618	1.4	
B	23	MA	80	13	0.48mi E Jon Crk Brkg	27.2	8	0.623	1.3	
C	1	MA	641	9.85	Fill	27.2	11			1.0
C	2	MA	62	6.7	Fill	27.2	27			1.3
C	3	MA	62	6	Fill	27.2	23			1.5
C	4	MA	62	6.3	Fill	27.2	23			1.5
C	5	MA	62	6.5	Fill	27.2	21			1.6
C	6	MA	62	5.8	Fill	27.2	21			1.7
C	7	MA	62	5.7	Fill	27.2	21			1.7
C	8	MA	62	4.5	Fill	27.2	19			1.8
C	9	MA	62	7.35	Fill	27.2	20			1.8
C	10	MA	62	5.9	Fill	27.2	18			1.9
C	11	MA	68/80	26.71	0.95mi E 68/80 Jnct	27.2	19			1.9
C	12	MA	62	5.4	Fill	27.2	17			2.0
C	13	MA	62	10.19	Fill	27.2	17			2.0
C	14	MA	80	15.8	0.74mi E Clr Crk Brdg	27.2	17			2.1
C	15	MA	641	0.3	Fill	27.2	18			2.3
C	16	MA	80	13.7	1.18 mi e Jon Crk Brdg	27.2	15			2.4
C	17	MA	62	0.9	Fill	27.2	14			2.5
C	18	MA	62	1.4	Fill	27.2	12			2.7
C	19	MA	641	10.4	Fill	27.2	11			3.0
C	20	MA	62	1.9	Culvert	27.2	11			3.0
C	21	MA	80	6.6	App Fill Martin Crk Brdg	27.2	8			3.4
C	22	MA	80	6.2	App Fill Martin Crk Brdg	27.2	8			3.6

Ranking, along with estimated displacement or factor of safety for **500 year event**.										
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	500 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated disp., cm)	C/D (Factor of Safety)
C	23	MA	641	11.9	Fill	27.2	6			3.8
C	24	MA	641	12.1	Culvert	27.2	6			3.9
C	25	MA	641	12.1	Culvert	27.2	6			3.9
C	26	MA	80	6.46	App Fill Martin Crk Brdg	27.2	6			4.0
Z		MA	641	9.83	Bridge	27.2	11			
Z		MA	641	9.87	Bridge	27.2	8			
Z		MA	641	0.24	Bridge	27.2				
Z		MA	62	11.94	Kentucky Dam/Tennessee River	27.2				
Z		MA	62	9.48	Cypress Crk Drain Bridge	27.2				
McCracken County										
A	1	MC	60	19.86	Clark Mem. Bridge	30.9	26	0.079	127.3	
A	2	MC	60	11.76	RR overpass	30.9	34	0.132	60.1	
A	3	MC	60	19.86	Clark Mem. Bridge	30.9	29	0.160	44.6	
A	4	MC	62	13.45	(West) Fill	30.9	42	0.166	42.0	
A	5	MC	60	11.65	Fill	30.9	32	0.233	23.0	
A	6	MC	60	6.71	Fill	30.9	27	0.256	19.2	
A	7	MC	60	19.86	Clark Mem. Bridge	30.9	35	0.270	17.2	
A	8	MC	60	19.64	60 West Overpass	30.9	20	0.283	15.6	
A	9	MC	60	19.86	Clark Mem. Bridge	30.9	20	0.305	13.2	
A	10	MC	60	6.6	Fill	30.9	27	0.315	12.3	
A	11	MC	60	19.86	Clark Mem. Bridge	30.9	31	0.325	11.5	
A	12	MC	60	8.31	Fill	30.9	17	0.326	11.3	
A	13	MC	60	8.29	Fill	30.9	16	0.333	10.8	
A	14	MC	60	11.09	Perkins Creek Bridge	30.9	16	0.340	10.3	
B	1	MC	62	14.85	Garrison Creek Culvert/Fill W	30.9	16	0.386	7.5	
B	2	MC	62	14.85	Garrison Creek Culvert/Fill E	30.9	16	0.386	7.5	
B	3	MC	60	4.1	Massac Creek Fork Bridge	30.9	11	0.473	4.1	
B	4	MC	60	4.05	Fill	30.9	11	0.473	4.1	
C	1	MC	62	14.3	Fill near Fisher Park	30.9	23			1.4
C	2	MC	62	16.25	Buzzard Creek	30.9	22			1.5
C	3	MC	62	15.9	Buzzard Creek	30.9	17			1.8
C	4	MC	62	13.85	(West) Fill	30.9	16			1.9
Z		MC	60	4.15	Fill	30.9	11			
Z		MC	60	1.3	Culvert	30.9	9			
Z		MC	68	1.01	Two I-24 Brdgs over US 68	30.9				
Z		MC	60	8.3	Bridge	30.9				
Z		MC	60	6.69	ICRR Bridge	30.9				
Z		MC	60	4.96	Fill	30.9				
Z		MC	60	4.95	Massac Creek Bridge	30.9				
Z		MC	60	4.85	Fill	30.9				
Z		MC	62	13.91	(East) Fill	30.9				
Todd County										
C	1	TO	68/80	1.55	Bridge W	11.1	14			1.7
C	2	TO	68/80	1.55	Bridge E	11.1	14			2.1

Ranking, along with estimated displacement or factor of safety for **500 year event**.										
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	500 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated disp., cm)	C/D (Factor of Safety)
C	3	TO	68/80	9.05	Fill	11.1	11			5.0
C	4	TO	68/80	9.15	Fill	11.1	10			5.5
C	5	TO	68/80	3.15	Bridge	11.1	12			5.9
C	6	TO	68/80	3.15	Bridge	11.1	12			6.2
Z		TO	68/80	9.1	Bridge	11.1	18			
Z		TO	68/80	1.55	West Fork of Red River Bridge	11.1				
Z		TO	68/80	1.7	0.15 mi E. of Red River Bridge	11.1				
Z		TO	68/80	1.8	0.25 mi E. of Red River Bridge	11.1				
Z		TO	68/80	3.15	Branch of W. Fk. of Red River	11.1				
Z		TO	68/80	9.05	App. fills for Elk Fork Bridge	11.1				
Z		TO	68/80	2.7	NO SITE	11.1				
Z		TO	68/80	2.8	NO SITE	11.1				
Trigg County										
A	1	TR	68/80	19.2	Little River Bridge	17.1	62	0.158	45.4	
A	2	TR	68/80	7	Fill at Elbow Bay	17.1	22	0.225	24.5	
B	1	TR	68/80	0.6	App. fill to Tenn. River Bridge	17.1	23	0.452	4.7	
C	1	TR	68/80	18.6	Fill on Cadiz Bypass	17.1	50			1.1
C	2	TR	68/80	8.2	W App Fill Cumb Riv Brdg	17.1	22			1.1
C	3	TR	68/80	17.9	Bridge on Cadiz Bypass W	17.1	47			1.1
C	4	TR	68/80	10.8	E App Fill Hopson Crk Brdg	17.1	18			1.2
C	5	TR	68/80	10.8	W App Fill Hopson Crk Brdg	17.1	18			1.3
C	6	TR	68/80	2.1	Fill	17.1	37			1.3
C	7	TR	68/80	2	Fill	17.1	30			1.5
C	8	TR	68/80	18.2	Fill on Cadiz Bypass(1 sided)	17.1	34			1.7
C	9	TR	68/80	10.05	W App Fill Cumb Riv Brdg	17.1	24			1.9
C	10	TR	68/80	3.11	E Approach to Trace Bridge	17.1	24			2.2
C	11	TR	68/80	7.8	W App Fill Cumb Riv Brdg	17.1	22			2.3
C	12	TR	68/80	3.11	W Approach to Trace Bridge	17.1	24			2.3
C	13	TR	68/80	9	E App Fill Cumb Riv Brdg	17.1	19			2.5
Z		TR	68/80	17.89	App Fill Little River Brdg	17.1				
Z		TR	68/80	24.5	I-24 (Proposed)	17.1				

APPENDIX

RANKINGS OF WESTERN KENTUCKY EMBANKMENTS

50 YEAR EVENT

Ranking, along with estimated displacement or factor of safety for **50 year event**.										
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	50 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated displace., cm)	C/D (Factor of Safety)
Ballard County										
A	1	BA	121	6.5	Fill	26.6	24	0.105	14.8	
B	1	BA	121	6.7	S. of 62/60/51 Jntn	26.6	24	0.215	4.7	
B	2	BA	60	3.5	Bridge E	26.6	19	0.346	1.7	
B	3	BA	60	3.5	Bridge W	26.6	19	0.348	1.7	
B	4	BA	60	3.85	Fill	26.6	20	0.385	1.3	
B	5	BA	60	11.51	Bridge E	26.6	16	0.385	1.3	
B	6	BA	60	1.95	Bridge	26.6	15	0.414	1.1	
B	7	BA	60	11.55	Fill W	26.6	14	0.438	0.9	
B	8	BA	60	1	Fill	26.6	16	0.466	0.7	
B	9	BA	60	1.95	Bridge	26.6	15	0.483	0.7	
B	10	BA	60	3.1	Fill	26.6	14	0.491	0.6	
B	11	BA	60	1.25	Fill	26.6	13	0.522	0.5	
B	12	BA	121	3.15	Stovall Crk Bridge North	26.6	11	0.526	0.5	
B	13	BA	60	3.8	Fill	26.6	13	0.529	0.5	
B	14	BA	60	3.93	Culvert Bridge	26.6	10	0.541	0.4	
B	15	BA	60	11.85	Bridge E	26.6	23	0.543	0.4	
B	16	BA	60	11.85	Bridge W	26.6	23	0.548	0.4	
B	17	BA	60	11.3	Culvert	26.6	9	0.556	0.4	
B	18	BA	60	10.23	Fill (Bridge Fill) W	26.6	15	0.557	0.4	
B	19	BA	60	1.94	Fill	26.6	9	0.558	0.4	
B	20	BA	60	1.15	Fill	26.6	12	0.559	0.4	
B	21	BA	60	0.8	Fill	26.6	12	0.564	0.4	
B	22	BA	60	3.68	Fill	26.6	12	0.564	0.4	
B	23	BA	60	10.23	Fill (Bridge Fill) E	26.6	15	0.584	0.3	
B	24	BA	60	2.9	Fill	26.6	10	0.595	0.3	
B	25	BA	60	2.3	Fill	26.6	7	0.601	0.3	
B	26	BA	60	5.74	Fill (Bridge Fill) W	26.6	15	0.606	0.3	
C	1	BA	60	5.74	Fill (Bridge Fill) E	26.6	15			1.0
C	2	BA	60	5.45	Fill	26.6	11			1.0
C	3	BA	60	5.32	Bridge	26.6	11			1.0
C	4	BA	60	0.75	Fill	26.6	10			1.0
C	5	BA	60	2.55	Fill	26.6	10			1.0
C	6	BA	121	0	Mayfield Crk Bridge North	26.6	8			1.0
C	7	BA	121	0	Mayfield Crk Bridge South	26.6	8			1.0
C	8	BA	60	3.5	Fill	26.6	8			1.0
C	9	BA	121	7.3	S. of 62/60/51 Jntn	26.6	28			1.5
C	10	BA	121	7.7	S. of 62/60/51 Jntn	26.6	23			1.7
C	11	BA	121	7.7	S. of 62/60/51 Jntn	26.6	16			2.2
Z		BA	121	3.15	Stovall Crk Bridge South	26.6	10			
Z		BA	121	5.3	Shelton Crk Bridge South	26.6	8			
Z		BA	121	5.3	Shelton Crk Bridge North	26.6	8			
Z		BA	60	12.5	Bridges & Fills (<=5')	26.6				

Ranking, along with estimated displacement or factor of safety for **50 year event**.										
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	50 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated displace., cm)	C/D (Factor of Safety)
Z		BA	60	1.85	Fill (<=5')	26.6				
Caldwell County										
C	1	CD	62	3	Fill	8.8	26			2.6
C	2	CD	62	2	Fill	8.8	19			3.3
C	3	CD	62	1.1	Fill	8.8	14			4.3
C	4	CD	62	0.65	Fill	8.8	14			4.4
Christian County										
C	1	CH	68/80	18.18	Bridge W	9.4	21			1.9
C	2	CH	91	4.43	Bridge N	9.4	20			1.9
C	3	CH	68/80	18.18	Bridge E	9.4	21			2.2
C	4	CH	91	4.43	Bridge S	9.4	20			2.3
C	5	CH	68/80	3.5	Fill	9.4	25			2.6
C	6	CH	68/80	3.6	Fill	9.4	25			2.6
C	7	CH	68/80	11.2	Pennyrile Pkwy Bridge	9.4	21			2.9
C	8	CH	68/80	4.65	Fill	9.4	16			3.7
C	9	CH	68/80	4.68	Muddy Branch Bridge	9.4	16			3.8
C	10	CH	68/80	10.76	Bridge W	9.4	28			4.4
C	11	CH	91	11.26	Bridge N	9.4	18			4.5
C	12	CH	91	13.7	Bridge S	9.4	12			7.6
C	13	CH	91	2.16	Bridge N	9.4	5			7.9
C	14	CH	91	11.26	Bridge S	9.4	18			8.0
C	15	CH	91	13.7	Bridge N	9.4	12			8.0
Z		CH	91	2.16	Bridge S	9.4				
Z		CH	68/80	10.76	Bridge E	9.4	28			
Z		CH	68/80	3.5	App Fill Ltl Sinking Fk Crk	9.4				
Z		CH	68/80	4.65	Muddy Fk Crk	9.4				
Z		CH	68/80	10.76	N Fk Little Rvr Drdg	9.4				
Z		CH	68/80	11.4	Pennyrile Brdg	9.4				
Z		CH	68/80	18.18	Brdg S Fk Little Rvr	9.4				
Z		CH	68/80	3.56	Bridge	9.4				
Z		CH	68/80	4.75	Fill	9.4				
Z		CH	68/80	11.4	Fill	9.4				
Carlisle County										
B	1	CL	121	1.2	Fill	26.2	18	0.413	1.1	
B	2	CL	121	4.8	Fill	26.2	17	0.413	1.1	
B	3	CL	121	0.05	Fill	26.2	18	0.420	1.0	
B	4	CL	121	0.4	Fill	26.2	17	0.438	0.9	
B	5	CL	121	4.2	Fill	26.2	16	0.452	0.8	
B	6	CL	62	3.88	Fill Bridge Fill E	26.2	16	0.460	0.8	
B	7	CL	121	2.85	Fill	26.2	16	0.470	0.7	
B	8	CL	121	4.9	Fill	26.2	15	0.473	0.7	
B	9	CL	121	5.1	Fill	26.2	15	0.480	0.7	
B	10	CL	121	0.2	Fill	26.2	15	0.491	0.6	
B	11	CL	121	2.51	Fill	26.2	14	0.520	0.5	

Ranking, along with estimated displacement or factor of safety for **50 year event**.										
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	50 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated displace., cm)	C/D (Factor of Safety)
B	12	CL	62	3.88	Fill Bridge Fill W	26.2	12	0.570	0.3	
B	13	CL	121	9.38	Brdge South	26.2	11	0.593	0.3	
B	14	CL	121	9.38	Brdge North	26.2	12	0.597	0.3	
B	15	CL	121	2.5	Fill	26.2	10	0.617	0.2	
C	1	CL	121	4.5	Fill	26.2	10			1.0
C	2	CL	121	9.1	Brdge South	26.2	10			1.0
C	3	CL	121	9.1	Brdge North	26.2	9			1.0
C	4	CL	62	6.04	Bridge	26.2	6			1.0
Calloway County										
B	1	CW	641	14.45	Fill	8.3	39	0.171	7.0	
B	2	CW	121	21.57	App. Fills for W. Fk. of Clarks River	8.3	11	0.765	0.1	
B	3	CW	94	24	Fill	8.3	43	0.915	0.0	
C	1	CW	94	17.1	Jonathan Creek W	8.3	20			1.2
C	2	CW	94	11.07	Fill	8.3	21			1.2
C	3	CW	94	11.44	Clarks River Bridge	8.3	27			1.2
C	4	CW	641	15.7	Fill	8.3	18			1.2
C	5	CW	94	17.1	Jonathan Creek E	8.3	17			1.3
C	6	CW	641	5.5	Fill	8.3	15			1.3
C	7	CW	94	11.3	Clarks River Bridge	8.3	23			1.4
C	8	CW	641	15.85	Fill	8.3	16			1.4
C	9	CW	641	5.65	Fill	8.3	15			1.4
C	10	CW	641	1.1	Fill	8.3	15			1.4
C	11	CW	94	16.5	Elm Grove Bridge(Jonathan Crk) E	8.3	15			1.4
C	12	CW	121	23.9	0.91 mi S. of CW-GR county line	8.3	13			1.4
C	13	CW	641	8.92	Fill	8.3	15			1.5
C	14	CW	641	8.9	Fill	8.3	11			1.6
C	15	CW	94	23.03	Fill E	8.3	13			1.6
C	16	CW	94	1.77	Williams Creek E	8.3	11			1.6
C	17	CW	641	15.6	Fill	8.3	12			1.7
C	18	CW	641	8.95	Fill	8.3	12			1.7
C	19	CW	121	23.5	1.31 mi S. of CW-GR county line	8.3	10			1.7
C	20	CW	94	11.4	Fill	8.3	23			1.7
C	21	CW	94	23.03	Fill W	8.3	13			1.8
C	22	CW	94	11.1	Clarks River Bridge	8.3	21			1.9
C	23	CW	94	16.5	Elm Grove Bridge(Jonathan Crk) W	8.3	15			2.0
C	24	CW	94	1.77	Williams Creek W	8.3	11			2.1
C	25	CW	641	1.2	Fill	8.3	15			2.1
C	26	CW	641	5.75	Fill	8.3	17			2.4
C	27	CW	641	5.6	Fill	8.3	15			2.5
C	28	CW	94	6.44	Haynes Creek E	8.3	15			3.9
C	29	CW	121	20.6	0.97 mi S. of W. Fk. of ClarksRiver	8.3	12			5.1
C	30	CW	94	6.44	Haynes Creek W	8.3	14			5.5

Ranking, along with estimated displacement or factor of safety for **50 year event**.										
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	50 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated displace., cm)	C/D (Factor of Safety)
Z		CW	641	5.49	Bridge	8.3	15			
Z		CW	641	1.15	Bridge	8.3	15			
Z		CW	641	15.81	Bridge	8.3				
Z		CW	641	15.65	Bridge	8.3				
Z		CW	641	5.66	Bridge	8.3				
Fulton County										
A	1	FU	94	29.15	Fill	26.8	29	0.084	20.1	
B	1	FU	94	29.2	Fill	26.8	28	0.203	5.2	
B	2	FU	94	17.85	Bridge	26.8	18	0.481	0.7	
B	3	FU	94	17.22	Bridge E	26.8	17	0.522	0.5	
B	4	FU	94	17.7	Fill	26.8	18	0.526	0.5	
B	5	FU	94	17.22	Bridge W	26.8	17	0.528	0.5	
B	6	FU	166	8.9	Fill	26.8	14	0.545	0.4	
B	7	FU	94	25.52	Bridge	26.8	14	0.5573	0.4	
B	8	FU	166	9.03	Bridge (Jnct 1125)	26.8	14	0.624	0.2	
Z		FU	94	24.04	Bridge @ Flood Stage	26.8	13			
Z		FU	166	2.09	Culvert	26.8	15			
Z		FU	94	24.22	Bridge	26.8	10			
Graves County										
A	1	GR	58/80	6.68	Mayfield Crk Bridge 3 E	14.5	24	0.123	11.6	
A	2	GR	58/80	6.68	Mayfield Crk Bridge 3 W	14.5	24	0.137	9.9	
B	1	GR	121	20.19	ICRR App. Fills South	14.5	29	0.156	8.1	
B	2	GR	121	20.19	ICRR App. Fills North	14.5	29	0.175	6.7	
B	3	GR	121	11.73	Mayfield Bypass App. Fill South	14.5	29	0.248	3.6	
B	4	GR	121	7.96	Mayfield Crk Bridge North	14.5	24	0.283	2.7	
B	5	GR	94	2.1	Fill	14.5	24	0.346	1.7	
B	6	GR	58	5.3	Fill	14.5	25	0.402	1.2	
B	7	GR	58	5.1	Fill	14.5	25	0.510	0.5	
B	8	GR	94	4.8	Fill	14.5	20	0.548	0.4	
B	9	GR	121	8.27	Mayfield Crk Branch Brdg North	14.5	11	0.921	0.0	
C	1	GR	121	22.3	0.3 mi S. of Co. Line	14.5	21			1.0
C	2	GR	121	21.7	0.9 mi S. of Co. Line	14.5	21			1.0
C	3	GR	121	7.96	Mayfield Crk Bridge South	14.5	17			1.0
C	4	GR	121	19.2	N. of Jntn of 1213/121	14.5	19			1.0
C	5	GR	121	11.73	Mayfield Bypass App. Fill North	14.5	19			1.0
C	6	GR	58/80	6.68	Mayfield Crk Bridge 2 W	14.5	12			1.0
C	7	GR	58/80	6.68	Mayfield Crk Bridge 1 E	14.5	12			1.0
C	8	GR	121	6.68	Mayfield Crk Bridge 1 W	14.5	12			1.0
C	9	GR	58/80	6.68	Mayfield Crk Bridge 2 E	14.5	12			1.0
C	10	GR	58/80	6.68	Mayfield Crk Bridge 4 W	14.5	12			1.0
C	11	GR	121	22	0.6 mi S. of Co. Line	14.5	17			1.1
C	12	GR	45	1.8	Jackson Creek	14.5	15			1.1
C	13	GR	94	4.3	Fill	14.5	21			1.1
C	14	GR	94	1.68	Fill	14.5	15			1.1

Ranking, along with estimated displacement or factor of safety for **50 year event**.										
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	50 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated displace., cm)	C/D (Factor of Safety)
C	15	GR	94	2.9	Bridge	14.5	15			1.1
C	16	GR	94	2.96	Fill	14.5	15			1.1
C	17	GR	121	16.1	0.8 mi N. of 440/121	14.5	15			1.2
C	18	GR	121	18.8	0.3 mi S. of Jntn of 1213/121	14.5	15			1.2
C	19	GR	58/80	12.25	Panther Crk App. Fills W	14.5	13			1.2
C	20	GR	94	4.9	Fill	14.5	13			1.2
C	21	GR	121	8.27	Mayfield Crk Branch Brdg South	14.5	11			1.2
C	22	GR	94	1.85	Fill	14.5	24			1.2
C	23	GR	58/80	12.25	Panther Crk App. Fills E	14.5	13			1.2
C	24	GR	45	10.54	Bridge W	14.5	23			1.3
C	25	GR	45	1.68	Bayou Chien Creek	14.5	19			1.3
C	26	GR	45	6.09	Bridge W	14.5	19			1.3
C	27	GR	121	5.2	N. of Ky 1890/121	14.5	11			1.3
C	28	GR	58/80	6.68	Mayfield Crk Bridge 4 E	14.5	12			1.3
C	29	GR	45	7.86	Bridge E	14.5	16			1.3
C	30	GR	45	13.1	Richard Creek Bridge	14.5	14			1.3
C	31	GR	58/80	12.44	Panther Crk Fork App. Fills W	14.5	10			1.3
C	32	GR	121	8.14	Mayfield Crk Overfl. Brdg North	14.5	11			1.3
C	33	GR	121	8.75	Kess Crk Bridge South	14.5	9			1.3
C	34	GR	58	3.9	Fill	14.5	10			1.3
C	35	GR	58	0.51	Culvert	14.5	14			1.3
C	36	GR	45	6.09	Bridge E	14.5	19			1.4
C	37	GR	94	2	Bridge E	14.5	15			1.4
C	38	GR	121	8.14	Mayfield Crk Overfl. Brdg South	14.5	10			1.4
C	39	GR	45	7.86	Bridge EE	14.5	16			1.4
C	40	GR	58/80	12.44	Panther Crk Fork App. Fills E	14.5	10			1.4
C	41	GR	45	7.8	Bridge W	14.5	16			1.4
C	42	GR	45	10.54	Bridge E	14.5	23			1.4
C	43	GR	58/80	9.9	1.7 mi. E. of Juntn 131 & 58/80	14.5	9			1.4
C	44	GR	58	0.51	Culvert	14.5	14			1.5
C	45	GR	58	2.83	Culvert	14.5	14			1.5
C	46	GR	58	2.83	Culvert	14.5	14			1.5
C	47	GR	45	12.2	Opossum Creek Bridge	14.5	14			1.5
C	48	GR	94	2	Bridge W	14.5	15			1.5
C	49	GR	121	8.75	Kess Crk Bridge North	14.5	5			1.6
C	50	GR	45	12.2	Opossum Creek Bridge	14.5	14			1.7
C	51	GR	45	1.8	Jackson Creek	14.5	14			1.7
C	52	GR	58/80	13.7	W. of Juntn KY 564/KY 58/80	14.5	20			2.6
C	53	GR	121	17.7	2.4 mi N. of 440/121	14.5	18			2.9
C	54	GR	94	3.7	Fill	14.5	15			3.3
C	55	GR	94	15.4	Fill (near 83)	14.5	14			3.7
C	56	GR	58/80	13.8	0	14.5	13			4.8
C	57	GR	121	1.9	0.4 mi S. of Jntn 564/94	14.5	5			6.9
Z		GR	94	0.19	Bridges	14.5	13			

Ranking, along with estimated displacement or factor of safety for **50 year event**.

Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	50 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated displace., cm)	C/D (Factor of Safety)
Z		GR	94	0.19	Bridges	14.5	9			
Z		GR	94	0.19	Bridges	14.5	7			
Z		GR	94	2.01	Bridge	14.5				
Z		GR	94	0.24	RR Underpass Bridge W	14.5				
Z		GR	58	5.27	Bridge	14.5				
Hickman County										
B	1	HI	58	19.85	Fill	30.8	30	0.362	1.5	
B	2	HI	94	16	Fill	30.8	17	0.467	0.7	
B	3	HI	58	19.7	Fill	30.8	30	0.510	0.5	
B	4	HI	94	16	Fill	30.8	17	0.513	0.5	
Z		HI	94	15.87	Bridge	30.8	17			
Z		HI	45	1.2	Fill & Tied-In Piles	30.8				
Z		HI	58	19.82	RR Bridge	30.8				
Livingston County										
A	1	LI	62/641	0.31	KY Lake Bridge	12.5	25	0.123	11.7	
C	1	LI	62/641	2.75	Fill	12.5	21			1.0
C	2	LI	62/641	0.97	Quarry Rd Bridge W	12.5	30			2.1
C	3	LI	62/641	0.97	Quarry Rd Bridge E	12.5	30			2.1
C	4	LI	62/641	0.64	RR Bridge W	12.5	28			4.5
C	5	LI	62/641	0.64	RR Bridge E	12.5	28			4.5
Z		LI	62/641	2.78	Bridge	12.5				
Logan County										
C	1	LO	68/80	21.7	Fill	9.1	8			2.1
C	2	LO	68/80	9.6	RR Bridge	9.1	24			2.4
C	3	LO	68/80	9.64	Fill	9.1	24			2.6
C	4	LO	68/80	12.5	Fill	9.1	18			3.5
C	5	LO	68/80	15.1	Fill	9.1	16			3.8
C	6	LO	68/80	2.8	Fill Bridge Fill W	9.1	25			4.4
C	7	LO	68/80	20.94	Bridge W	9.1	17			4.6
C	8	LO	68/80	2.8	Fill Bridge Fill E	9.1	11			5.5
C	9	LO	68/81	25.75	Bridge E	9.1	17			5.8
C	10	LO	68/80	21.94	Bridge E	9.1	17			6.2
C	11	LO	68/80	24.75	Bridge W	9.1	17			6.2
Z		LO	68/80	10.33	Bridge	9.1	8			
Z		LO	68/80	2.8	Whip-will Crk Brdg	9.1				
Z		LO	68/80	9.64	L&N RR Brdg	9.1				
Z		LO	68/80	10.33	Town Brnch E Fk	9.1				
Z		LO	68/80	12.5	E Russ Point Brnch	9.1				
Z		LO	68/80	15.1	3.2m W 722 N Jctn	9.1				
Z		LO	68/80	20.94	Blk Lick Crk Brdg	9.1				
Z		LO	68/80	21.7	W L&N RR	9.1				
Z		LO	68/80	21.95	E L&N RR	9.1				
Z		LO	68/80	21.91	Bridge	9.1				
Z		LO	68/80	21.95	Fill (<5')	9.1				

Ranking, along with estimated displacement or factor of safety for **50 year event**.										
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	50 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated displace., cm)	C/D (Factor of Safety)
Lyon County										
B	1	LY	62/641	4.5	Fill	8.6	49	0.506	0.6	
B	2	LY	62/641	4.8	Fill	8.6	43	0.289	2.6	
C	1	LY	62/641	7.3	Fill	8.6	26			1.0
C	2	LY	62/641	8.25	Fill	8.6	35			1.0
C	3	LY	62/641	7.5	Fill	8.6	27			1.2
C	4	LY	62/641	9.2	Fill	8.6	24			1.3
C	5	LY	62/641	7.8	Fill	8.6	23			1.4
C	6	LY	62/641	7.4	Fill	8.6	23			1.4
C	7	LY	62/641	7.6	Fill	8.6	23			1.4
C	8	LY	62/641	10.35	Fill	8.6	15			1.4
C	9	LY	62/641	9.65	Fill	8.6	16			1.4
C	10	LY	62/641	1.65	Fill	8.6	15			1.5
C	11	LY	62/641	1.1	Fill	8.6	15			1.5
C	12	LY	62/641	9.85	Fill	8.6	14			1.6
C	13	LY	62/641	9.95	Fill	8.6	13			1.6
C	14	LY	62	12.8	Bridge	8.6	12			1.6
C	15	LY	62/641	3.67	Fill	8.6	11			1.8
C	16	LY	62	13.4	Fill	8.6	7			2.3
C	17	LY	62	11.45	Fill	8.6	15			4.0
Z		LY	62/641	7.3	Fill	8.6				
Z		LY	62	11.6	Bridge	8.6				
Z		LY	62	12.2	Fill	8.6				
Marshall County										
A	1	MA	62	10.9	Fill	14.1	40	0.067	27.5	
B	1	MA	62	2.47	Culvert	14.1	27	0.160	7.8	
B	2	MA	62	2.55	Fill	14.1	27	0.163	7.5	
B	3	MA	80	9.9	App Fill Clark Rvr Brdg E	14.1	26	0.175	6.7	
B	4	MA	68	9.43	App Fill Jack Purch Pkwy Brdge W	14.1	25	0.397	1.2	
B	5	MA	58/80	1.12	App Fill W Fk Clarks Rvr Brdg E	14.1	24	0.422	1.0	
B	6	MA	58/80	1.12	App Fill W Fk Clarks Rvr Brdg W	14.1	24	0.422	1.0	
B	7	MA	94	0.4	Fill	14.1	27	0.424	1.0	
B	8	MA	68	9.43	App Fill Jack Purch Pkwy Brdge E	14.1	23	0.443	0.9	
B	9	MA	641	9.4	Bridge	14.1	23	0.479	0.7	
B	10	MA	641	9.45	Fill	14.1	23	0.479	0.7	
B	11	MA	68/80	27.8	App Fill Tenn Rvr Brdg	14.1	24	0.525	0.5	
B	12	MA	68	22.48	App Fill Jon Crk Brdg East	14.1	22	0.546	0.4	
B	13	MA	68	22.48	App Fill Jon Crk Brdg West	14.1	23	0.547	0.4	
B	14	MA	62	1.2	Fill	14.1	14	0.949	0.0	
C	1	MA	80	10.4	0.5 mi E Clark Rvr Brdg	14.1	19			1.0
C	2	MA	80	9.9	App Fill Clark Rvr Brdg W	14.1	26			1.0
C	3	MA	80	12.52	App Fill Jonathan Crk Brdg W	14.1	43			1.0
C	4	MA	62	9.45	Fill	14.1	20			1.0

Ranking, along with estimated displacement or factor of safety for **50 year event**.											
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	50 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated displace., cm)	C/D (Factor of Safety)
C	5	MA	62	9.55	Fill	14.1	20			1.0
C	6	MA	641	0.2	Fill	14.1	18			1.0
C	7	MA	80	12.52	App Fill Jonathan Crk Brdg E	14.1	46			1.0
C	8	MA	62	10.87	Fill	14.1	17			1.1
C	9	MA	80	10.9	1 mi E Clark Rvr Brdg	14.1	16			1.1
C	10	MA	62	10.87	Fill	14.1	17			1.1
C	11	MA	80	9.4	0.4 mi. west of Clark R. Brdg W	14.1	10			1.1
C	12	MA	80	9.4	0.4 mi. west of Clark R. Brdg E.	14.1	10			1.1
C	13	MA	62	3.6	Fill	14.1	14			1.2
C	14	MA	62	0.7	Fill	14.1	14			1.2
C	15	MA	94	1.6	Terrapin Creek Fill	14.1	15			1.2
C	16	MA	641	5.85	Culvert @ 1518	14.1	16			1.2
C	17	MA	68	24.85	0.95mi W 68/80 Jnct	14.1	12			1.2
C	18	MA	80	8.4	Bridge Approach Fill	14.1	7			1.2
C	19	MA	80	15.06	App Fill Clear Crk Brdg E	14.1	12			1.3
C	20	MA	80	15.06	App Fill Clear Crk Brdg W	14.1	12			1.3
C	21	MA	62	1.7	Fill	14.1	10			1.4
C	22	MA	80	8.8	0	14.1	8			1.4
C	23	MA	80	8.8	0	14.1	8			1.4
C	24	MA	641	9.94	Fill	14.1	9			1.5
C	25	MA	641	9.8	Fill	14.1	11			1.5
C	26	MA	80	12.1	2.2mi E Clrk Rvr Brdg E	14.1	9			1.5
C	27	MA	80	12.1	2.2mi E Clrk Rvr Brdg W	14.1	9			1.5
C	28	MA	80	13	0.48mi E Jon Crk Brkg	14.1	8			1.5
C	29	MA	641	9.85	Fill	14.1	11			1.6
C	30	MA	62	6.7	Fill	14.1	27			2.0
C	31	MA	62	6	Fill	14.1	23			2.3
C	32	MA	62	6.3	Fill	14.1	23			2.5
C	33	MA	62	5.8	Fill	14.1	21			2.5
C	34	MA	62	7.35	Fill	14.1	20			2.6
C	35	MA	62	5.7	Fill	14.1	21			2.6
C	36	MA	68/80	26.71	0.95mi E 68/80 Jnct	14.1	19			2.7
C	37	MA	62	6.5	Fill	14.1	21			2.7
C	38	MA	62	4.5	Fill	14.1	19			2.9
C	39	MA	62	5.9	Fill	14.1	18			3.0
C	40	MA	62	5.4	Fill	14.1	17			3.1
C	41	MA	62	10.19	Fill	14.1	17			3.1
C	42	MA	641	0.3	Fill	14.1	18			3.2
C	43	MA	80	15.8	0.74mi E Clr Crk Brdg	14.1	17			3.3
C	44	MA	80	13.7	1.18 mi e Jon Crk Brdg	14.1	15			3.5
C	45	MA	62	0.9	Fill	14.1	14			3.6
C	46	MA	62	1.4	Fill	14.1	12			4.1
C	47	MA	62	1.9	Culvert	14.1	11			4.5
C	48	MA	641	10.4	Fill	14.1	11			4.8

Ranking, along with estimated displacement or factor of safety for **50 year event**.										
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	50 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated displace., cm)	C/D (Factor of Safety)
C	49	MA	80	6.6	App Fill Martin Crk Brdg	14.1	8			5.5
C	50	MA	80	6.2	App Fill Martin Crk Brdg	14.1	8			6.0
C	51	MA	641	11.9	Fill	14.1	6			6.3
C	52	MA	641	12.1	Culvert	14.1	6			6.6
C	53	MA	641	12.1	Culvert	14.1	6			6.7
C	54	MA	80	6.46	App Fill Martin Crk Brdg	14.1	6			7.0
Z		MA	641	9.83	Bridge	14.1	11			
Z		MA	641	9.87	Bridge	14.1	8			
Z		MA	641	0.24	Bridge	14.1				
Z		MA	62	11.94	Kentucky Dam/Tennessee River	14.1				
Z		MA	62	9.48	Cypress Crk Drain Bridge	14.1				
McCracken County										
B	1	MC	60	19.86	Clark Mem. Bridge	13.4	26	0.183	6.2	
B	2	MC	60	11.76	RR overpass	13.4	34	0.305	2.3	
B	3	MC	60	19.86	Clark Mem. Bridge	13.4	29	0.368	1.5	
B	4	MC	62	13.45	(West) Fill	13.4	42	0.382	1.3	
B	5	MC	60	11.65	Fill	13.4	32	0.536	0.4	
B	6	MC	60	19.86	Clark Mem. Bridge	13.4	31			0.8
C	1	MC	60	6.71	Fill	13.4	27			1.0
C	2	MC	60	19.64	60 West Overpass	13.4	20			1.0
C	3	MC	60	19.86	Clark Mem. Bridge	13.4	35			1.0
C	4	MC	60	19.86	Clark Mem. Bridge	13.4	20			1.0
C	5	MC	60	6.6	Fill	13.4	27			1.0
C	6	MC	60	8.31	Fill	13.4	17			1.1
C	7	MC	60	8.29	Fill	13.4	16			1.1
C	8	MC	60	11.09	Perkins Creek Bridge	13.4	16			1.1
C	9	MC	62	14.85	Garrison Creek Culvert/Fill E	13.4	16			1.1
C	10	MC	62	14.85	Garrison Creek Culvert/Fill W	13.4	16			1.1
C	11	MC	60	4.1	Massac Creek Fork Bridge	13.4	11			1.4
C	12	MC	60	4.05	Fill	13.4	11			1.4
C	13	MC	62	14.3	Fill near Fisher Park	13.4	23			2.4
C	14	MC	62	16.25	Buzzard Creek	13.4	22			2.4
C	15	MC	62	13.85	(West) Fill	13.4	16			3.1
C	16	MC	62	15.9	Buzzard Creek	13.4	17			3.1
Z		MC	60	4.15	Fill	13.4	11			
Z		MC	60	1.3	Culvert	13.4	9			
Z		MC	68	1.01	Two I-24 Brdgs over US 68	13.4				
Z		MC	60	8.3	Bridge	13.4				
Z		MC	60	6.69	ICRR Bridge	13.4				
Z		MC	60	4.96	Fill	13.4				
Z		MC	60	4.95	Massac Creek Bridge	13.4				
Z		MC	60	4.85	Fill	13.4				
Z		MC	62	13.91	(East) Fill	13.4				
Todd										

Ranking, along with estimated displacement or factor of safety for **50 year event**.										
Class	Rank in Class	County Symbol	Hwy	Milepoint	Description	50 yr event (%g)	Maximum Slope Height (ft)	Khf/Kmax	U (estimated displace., cm)	C/D (Factor of Safety)
County										
C	1	TO	68/80	1.55	Bridge W	9.1	14			1.9
C	2	TO	68/80	1.55	Bridge E	9.1	14			2.5
C	3	TO	68/80	9.05	Fill	9.1	11			5.8
C	4	TO	68/80	9.15	Fill	9.1	10			6.5
C	5	TO	68/80	3.15	Bridge	9.1	12			6.5
C	6	TO	68/80	3.15	Bridge	9.1	12			7.3
C	7	TO	68/80	9.1	Bridge	9.1	18			8.0
Z		TO	68/80	1.55	West Fork of Red River Bridge	9.1				
Z		TO	68/80	1.7	0.15 mi E. of Red River Bridge	9.1				
Z		TO	68/80	1.8	0.25 mi E. of Red River Bridge	9.1				
Z		TO	68/80	3.15	Branch of W. Fk. of Red River	9.1				
Z		TO	68/80	9.05	App. fills for Elk Fork Bridge	9.1				
Trigg County										
B	1	TR	68/80	19.2	Little River Bridge	8.9	62	0.304	2.3	
B	2	TR	68/80	7	Fill at Elbow Bay	8.9	22	0.433	0.9	
C	1	TR	68/80	0.6	App. fill to Tenn. River Bridge	8.9	23			1.1
C	2	TR	68/80	18.6	Fill on Cadiz Bypass	8.9	50			1.3
C	3	TR	68/80	17.9	Bridge on Cadiz Bypass W	8.9	47			1.4
C	4	TR	68/80	8.2	W App Fill Cumb Riv Brdg	8.9	22			1.4
C	5	TR	68/80	2.1	Fill	8.9	37			1.7
C	6	TR	68/80	10.8	E App Fill Hopson Crk Brdg	8.9	18			1.8
C	7	TR	68/80	10.8	W App Fill Hopson Crk Brdg	8.9	18			1.8
C	8	TR	68/80	2	Fill	8.9	30			2.1
C	9	TR	68/80	18.2	Fill on Cadiz Bypass(1 sided)	8.9	34			2.2
C	10	TR	68/80	10.05	W App Fill Cumb Riv Brdg	8.9	24			2.7
C	11	TR	68/80	7.8	W App Fill Cumb Riv Brdg	8.9	22			2.9
C	12	TR	68/80	3.11	E Approach to Trace Bridge	8.9	24			3.1
C	13	TR	68/80	3.11	W Approach to Trace Bridge	8.9	24			3.2
C	14	TR	68/80	9	E App Fill Cumb Riv Brdg	8.9	19			3.3
Z		TR	68/80	17.89	App Fill Little River Brdg	8.9				
Z		TR	68/80	24.5	I-24 (Proposed)	8.9				