1.0 INTRODUCTION

Southeast Missouri experiences relatively small magnitude earthquakes on a regular basis, and is the site of several of the largest magnitude (estimated 8.0 - 8.3) earthquake events to strike North America in recorded history (1811 - 1812). Experts agree that similar or greater magnitude earthquakes will strike this region again.

Geologic conditions in southeast Missouri make this region one of the most seismically susceptible in the country, based on its damage potential from intrinsically susceptible soil, high ground water levels and vast expanses of flood sensitive ground. If a high magnitude earthquake struck southeast Missouri today, infrastructure in the area would be devastated. Levees and dams could be breached. Bridges across the Mississippi and Missouri rivers could collapse or be rendered unusable. Landslides, floods, soil liquefaction, and the failure of roadway bridges and overpasses would close extended sections of highway. The network of lifeline facilities and services required for commerce and public health in St. Louis, Sikeston, Cape Girardeau and surrounding communities would be devastated. Utilities, including electrical power, communications, oil and gas distribution, sewage disposal and water distribution, would be disabled until emergency repair crews were able to access these communities. Southeast Missouri would be effectively cutoff from the rest of the world and individual towns and communities isolated.

In the event of a major earthquake, the reopening of emergency vehicle priority access routes into St. Louis, Sikeston and Cape Girardeau would be a top priority. To facilitate the rapid, cost-effective reopening of roadways and expedite the transport of aid into affected communities, a study of the earthquake susceptibility of roadways, bridges and overpasses in southeast Missouri is required. The Missouri Department of Transportation has designated the most viable of these routes as emergency vehicle priority access. In order to insure that the designated access routes will remain open post earthquake, the Missouri Department of Transportation needs to confirm that these re-entry routes will not sustain major unacceptable earthquake related damage, and, hence, could be reopened quickly following an earthquake event. In order to determine if the routes are viable, the Missouri Department of Transportation must assess the earthquake susceptibility of existing overpasses, bridges, dams, levees, canals and foundation soils along these routes. Ultimately, the Missouri Department of Transportation may elect to reinforce these features where necessary, thereby minimizing earthquake damage, repair time and costs.

The earthquake hazards assessment of designated emergency vehicle priority access routes, which are mainly National Highway System route in southeast Missouri, will produce tangible economic and humanitarian benefits. The benefits will be realized if the Missouri Department of Transportation is able to reopen designated highways in a timely and cost-effective manner. This effort fully supports the Federal Highway Administration (FHWA) National Strategic Plan's mobility goal related to the strategic objective of returning highways to full service following disasters. The expertise, methodologies and technologies developed during the course of this study will be made available to adjacent states through presentations in appropriate venues and by publications of the study in appropriate journals. In this way, similar site-specific studies of priority access routes in other Midwestern States will have the benefit of the protocols and procedures developed in this work.

2.0 STATEMENT OF PROBLEM/SCOPE OF WORK

The designated emergency vehicle priority access route into southeast Missouri includes portions of US 60. This route traverses varied geologic settings and includes or crosses many critical roadway features such as bridges, slopes, box culverts, and retaining walls. The extent of damage and survivability of these critical roadway features in the event of a major earthquake event is not fully known and would impact the ability to use these designated routes to provide emergency vehicular access in a timely manner.

This study involves the assessment of four critical bridges at two sites along US 60 (Figure 2.1) and the development of an initial geotechnical database that will be part of a future regional geotechnical GIS database. The methodologies developed in this study will be used to establish an assessment protocol. The output-interpreted geotechnical data will be used for future prioritization and retrofit of deficiencies noted at the bridge sites studied.

3.0 OBJECTIVES

There were two primary objectives for this study. Objective 1 was to establish a geotechnical database for earthquake design and future use in a geographic information system (GIS) for the portions of US 60 and MO 100 in the counties of Butler, Stoddard, New Madrid, Franklin and St. Louis. Objective 2 was to conduct detailed earthquake assessments at two sites along designated emergency vehicle priority access route US 60.

3.1 Geotechnical Databases

Databases have been established for current subsurface and earthquake design data for the US 60 corridor in Butler, Stoddard and New Madrid Counties and for the MO 100 corridor in Franklin and St. Louis Counties. The database includes appropriate geotechnical data from Missouri Department of Transportation files. These databases will be integrated into the existing Missouri Department of Transportation GIS system for future access, and serve as the beginning of a larger regional or statewide database for future development and use by the Missouri Department of Transportation.

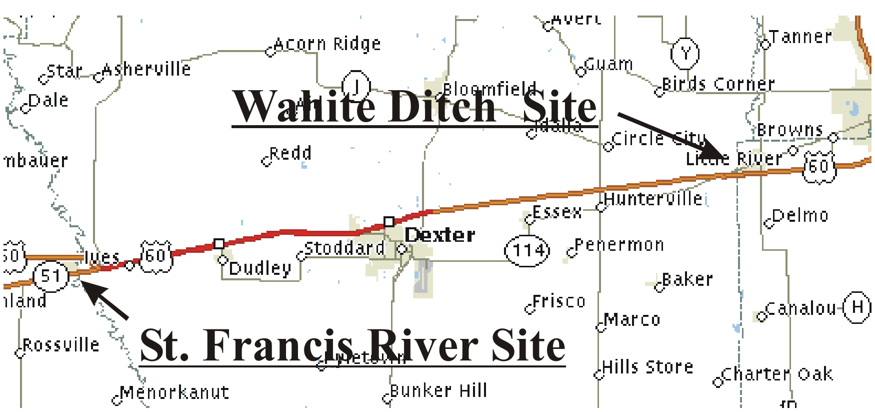


Figure 2.1 Study Site Locations

3.2 Site Specific Earthquake Hazards Assessments

Members of the Missouri Department of Transportation, University of Missouri-Rolla Departments of Geology and Geophysics, Civil Engineering and Geological Engineering and the Missouri Department of Natural Resources research team visited US 60 in Butler, Stoddard and New Madrid Counties. Sites with critical roadway features were visually evaluated and ranked based upon geologic, structural and perceived criticality/risk factors. The top two sites with differing geologic settings and potential high-risk earthquake hazards were selected for detailed site-specific earthquake assessments as part of this study. The sites selected are located in Stoddard County where US 60 crosses Wahite Ditch Number 1 and in Butler County and Stoddard County where US 60 crosses the St. Francis River.

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

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

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

2. Evaluate the susceptibility of each site to earthquake-induced slope instability and liquefaction.

3. Estimate shaking effects on the two types of existing bridge structures at each site.

4. Compare ground motion and structural response parameters from the site-specific earthquake analysis with those from the American Association of State Highway and Transportation Officials (AASHTO) response spectrum analysis method and provide preliminary guidance regarding selection of the analysis method at future sites.

5. Evaluate modified site assessment techniques and establish a basis for using these modified techniques at other sites along designated emergency access routes.

4.0 MISSOURI DEPARTMENT OF TRANSPORTATION GEOTECHNICAL DATABASE

4.1 Design

The primary goal for this database was to develop a repository of usable geotechnical data for the Missouri Department of Transportation. This section of the report outlines the philosophies behind both the development of the database and the design approach.

4.1.1 Design Approach

The approach to the development of the database revolved around the overall goal of designing a Missouri Department of Transportation statewide geotechnical database, customized to the needs of this project. There are two classical approaches to data management design: "top-down" or "bottom-up." A top-down design approach consists of conceptualizing the problem, breaking it into manageable sub-problems, identifying appropriate methodologies and processes, and manipulating the data to achieve a result that will impact the "real" world. This approach is idealistic and generally applicable only when there is no existing data and/or database.

On the other hand, when there are abundant data and information, or an existing database, the development of a system requires the use of a bottom-up approach. This requires the analysis of data format and structure prior to the identification of methodologies and processes. Once methodologies/processes have been identified, a final model can be developed. Figure 4.1 shows a hierarchical schematic of these two alternative system design approaches. The two classical approaches described above represent the extremes of how systems are designed.

For this project, an initial step was to model geologic and geotechnical data using a top-down approach. Topics related to construction of transportation systems and their subsurface characterizations were included in this phase. Data were categorized into different classes (Figure 4.2).

Missouri Department of Transportation investigators provided borehole logs and associated soil-testing data. This necessitated modification to the database design approach. When data became available, the database design shifted to a bottom-up approach. The categories dimmed in Figure 4.2 were not pursued any further. The scope of the database was focused to include only data at highway structure locations provided by Missouri Department of Transportation investigations.

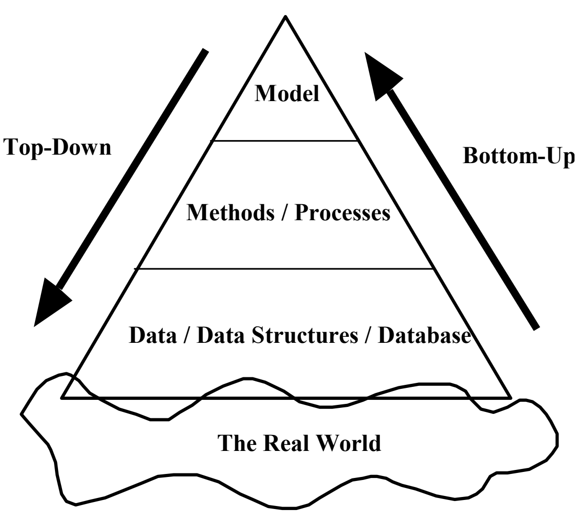


Figure 4.1 System Design: Top-Down vs. Bottom-Up

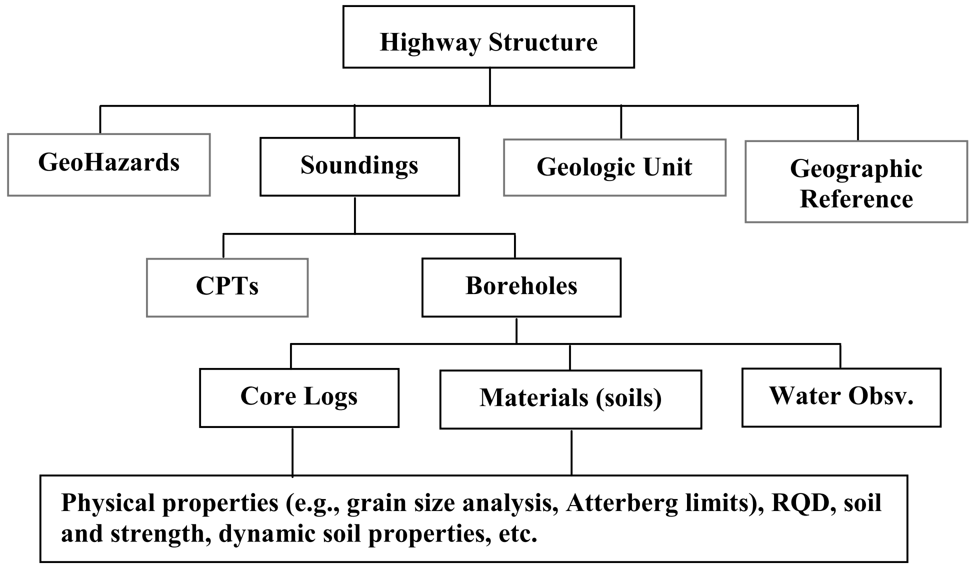


Figure 4.2 Organization of Missouri Department of Transportation Subsurface Data

Ultimately, a combination of the top-down and bottom-up approaches was used to design the database. The existing geotechnical data dictated the uniqueness of the application and the model developed. However, the design of the different tables was organized from a hierarchical point of view. In other words, the design was an iterative process of studying the data definitions/form/structure and developing the conceptual model and methods.

4.1.2 A Geotechnical Generic Example

A traditional geotechnical engineering project typically focuses on the subsurface characterization of a specific site and the interaction of man-made structures with the earth mass. However, multi-disciplinary projects usually require the expansion of the focus into other related fields (e.g., bridges, environmental, geology, etc.). In such instances, the engineer may be required to collect a broad range of available information to help solve a problem. The sources of information are the subsurface data recovered by invasive (e.g., boreholes, soundings) and non-invasive (e.g., geophysical, remotely sensed) explorations, the existing surface features, and the future surface and subsurface features planned for the site, if any. The multiple types of information are available in different physical forms and the engineer's expertise and judgment are used to synthesize this information and make decisions and recommendations about how to proceed with the project. When the amount of information that can be effectively collected and manipulated is abundant, the use of an information and database management system can aid in the problem solving process.

Data introduced into a database can serve not only a specific project, but for a continued period of time and for other projects. However, problems involving the legacy and integrity of the data may become an issue. For example, when data is retrieved and used, it may incur changes that alter the database, depending on the read/write permissions allocated to a user. Spatial information uses coordinate systems and map projections that may be modified during the life of the data and a record of these transformations needs to be stored. The date and the units of a value stored in a field need to be documented. Therefore, a record that keeps track of the data transformation and contents should be used and is usually referred as "data about the data" or metadata. Since a database may be intended to serve information for a continued period of time it is important to identify the data sources, the data requirements, and the data structures (Luna and Frost, 1995).

The general principles of object-oriented modeling and design were followed. However, the diagram included in this section does not necessarily follow a standard notation for reasons of clarity (also not common language in civil engineering). The three models used in the Object Modeling Technique (OMT) are the object model, the dynamic model, and the functional model. They each represent a different aspect of the system: object model - static, structural, "data"; dynamic model - temporal, behavioral, "control"; functional model - transformational, "function" (Rumbaugh et al., 1991). For this Missouri Department of Transportation database, the object model has been adopted to represent subsurface geotechnical data and a generic example is shown in Figure 4.3. These three kinds of models separate a system into three orthogonal views and are not completely independent, but each model can be examined and understood by itself to a large extent. The final architecture of the database was a product of the data structures and the module integration and will be discussed in more detail later.

4.1.3 Analysis and Data Structure

Probably the most time consuming task in the database aspect of the project was the analysis and definition of the data structure for the database. Missouri Department of Transportation's customized needs were met by focusing on data from borehole explorations and by retaining the terminology consistent with the analog data (paper form) provided, such as soil descriptions, stratigraphy, and testing nomenclature. No digital data were available for inclusion in this database. An extensive reference was developed by The Missouri Department of Natural Resources to perform data entry into database (refer to accompanying User's Manual). Soil descriptions and soil test names were standardized with the assistance of Missouri Department of Transportation.

Additionally, one item that required several iterations was related to the definition of fields and records. To overcome this problem, the Missouri Department of Transportation and project investigators were asked to submit the nomenclature of geotechnical parameters, and to provide typical value units, and maximum and minimum values for each. For example, Table 4.1 shows an example of a more extensive list of geotechnical earthquake engineering parameters with the required information to define the structure of the data. The complete list is provided in User Instructions for Data Entry and Editing Database of Borehole and Other Geotechnical Data for Missouri Highway Structures.

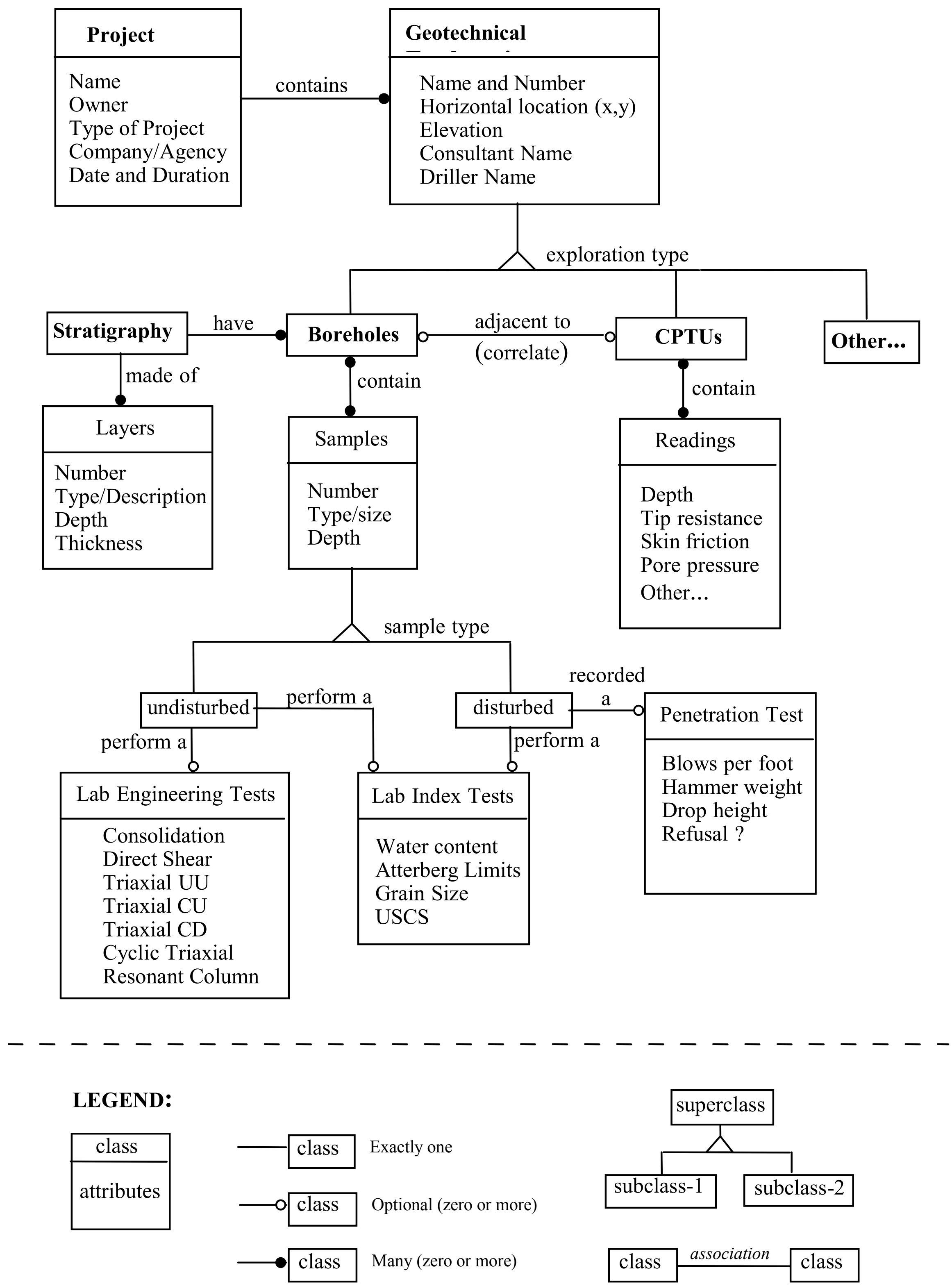


Figure 4.3 Example of An Object-Oriented Geotechnical Database Model (Luna and Frost, 1995)

Table 4.1 Example of Data Structure Input to Database

Field Name	Field Description	Field Type(1)	Field Size	Decimal Places	Min. Value	Max. Value	Units
Elev	Top of layer elevation	N	6	2	0.00	100.00	m
Soil Type	USCS Soil Classification	A	10				
γ	Dry Unit Weight	N	5	2	90.00	140.00	Lb/ft^3
NSPT	Measured Standard Penetration Value	N	3	0	1	50	Blow/ft
Nspt-CPT Corr	Correlation SPT_CPT data (qc/N, qc in kN/m^2; N=Nspt)	N	6	2	400.0	1000.00	
Less than 0.075	Percent passing 0.075 mm sieve	N	5	2	0	100.00	%
Vs	Shear wave velocity	N	6	2	110.0	260.00	M/s
G	Shear modulus	N	7	2	50.00	3200.00	Kg/cm^2

Notes: (1) N= numeric; A = alphanumeric

4.2 Implementation

The database design was implemented using the Microsoft Access software package. It is currently operational on a Pentium-based computer using the Windows NT operating system. The database is being populated by means of an interface designed specifically for this project. Refer to the companion document, User Instructions for Data Entry and Editing Database of Borehole and Other Geotechnical Data for Missouri Highway Structures for details about the rationale and usage of these “forms” for data entry into the tables.

4.3 Link to a Spatial Database (GIS)

The database was designed to link to a Geographic Information System (GIS). In principle, it can be referred as a spatial database. The data fields with geographic coordinates and referenced coordinate system are field identified as key items in the databases. However, at this time the entries to each borehole are not available. It is essential to link the boreholes to a common geographic reference so they can be related to the other spatial themes.

5.0 SITE CHARACTERIZATION: PROCEDURES

5.1 Field Investigations

The two study sites were investigated using both surface and subsurface exploration and mapping techniques. The details of the investigation program are discussed below.

5.1.1 Drilling and Sampling

Exploratory boreholes were advanced at each bridge site using mud-rotary techniques with a drill rig and personnel provided by Missouri Department of Transportation. Three 50-foot boreholes, two 100-foot boreholes, and one 200-foot borehole were made at both bridge sites.

The sampling interval varied with depth. The planned intervals were to sample continuously from the ground surface to a depth of 30 feet, to sample at 5-foot intervals from 30 to 80 feet, and to sample at 10-foot intervals thereafter. Several types of samples were collected, depending upon the soil type and depth. Shelby tube and SPT samples were alternated for cohesive soils, and only SPT samples were taken for non-cohesive soils. Shelby tube diameters were varied between 3-inch and 5-inch tubes so samples could be used for a variety of tests. Missouri Department of Transportation personnel logged the borings, retrieved and field-tested the samples. The samples were wrapped and sealed in paraffin for later testing. Field-testing consisted of torvane and pocket penetrometer testing of the cohesive samples. SPT values were recorded for both cohesive and non-cohesive soils.

5.1.2 Test Pits

Shallow exploratory test pits were dug to provide information on fill depths, lateral stratigraphy, and homogeneity of site soils. The local Missouri Department of Transportation maintenance shed near each bridge site provided a backhoe and operator to dig and backfill pits. An engineer from the University of Missouri-Rolla selected trench locations and prepared a log of soil units visible in the test pit walls.

5.1.3 Cone Penetrometer

Six seismic cone penetrometer soundings were completed at the St. Francis River Site and five at the Wahite Ditch Site, with a goal of advancing the soundings as deep as feasible (estimated to be 80 ft or 25 m). Recorded parameters included tip resistance, local friction, pore pressure, and inclination at 0.15 ft (0.05 m) intervals, as well as seismic velocity at 3.3 ft (1 m) intervals. Parameters calculated or correlated from recorded parameters included friction ratio, soil type, and SPT. In many cases, soil resistance exceeded the available capacity of the cone rig (CME 850) and soundings were halted before the target depth was reached.

5.1.4 Surface Mapping

A University of Missouri-Rolla engineer developed site engineering geologic maps showing estimated limits and types of fill material, geometry of site slopes, and other geologic features potentially impacting the roadway, bridges, or abutments. Because both bridge sites proved to be rather simple and homogeneous geologically, information from this field mapping was incorporated into the slope stability cross-sections and the site geology discussion and is not presented separately.

5.1.5 Interviews with Local Personnel

Mr. Dan Camden of the Wappapello Lake Control office, U.S. Army Corps of Engineers was contacted to obtain information regarding potential impacts to the US 60 roadway following catastrophic failure of the Wappapello Dam, on the St. Francis River.

Mr. Lonnie Blasingame, Missouri Department of Transportation Regional Superintendent, provided information on historic maintenance on the portions of US 60 near the St. Francis River Site and Wahite Ditch Site.

5.2 Laboratory Investigations

Soil samples taken from the borings at the two sites were transported initially to the Missouri Department of Transportation Geotechnical Laboratory. The field boring logs were reviewed and soil samples selected for testing. Missouri Department of Transportation personnel conducted basic index tests at the Missouri Department of Transportation Geotechnical Laboratory. Static and cyclic triaxial tests were conducted at the University of Missouri-Rolla Civil Engineering Geotechnical Laboratories.

5.2.1 Missouri Department of Transportation Laboratory Testing

Soil tests conducted by Missouri Department of Transportation personnel include:

- Pocket penetrometer
- Torvane
- Natural water content
- Liquid limit
- Plastic limit
- Unconfined compression
- Sieve and hydrometer

All tests were conducted in general accord with the applicable AASHTO Standards. The results of their testing are given in Appendix B.

5.2.2 University of Missouri-Rolla Laboratory Soil Testing

Upon completion of the classification testing, the soils were transported to the University of Missouri-Rolla for testing in their Geotechnical Engineering Laboratory. The soils were categorized into three general soil types for testing (high plastic clay, low plastic clay and silt). The University of Missouri-Rolla conducted static consolidated-undrained (staged and multi-sample CU) tests and strain controlled cyclic triaxial tests on samples of the cohesive soils. The silt soils were disturbed such that they could not be tested in the laboratory.

5.2.2.1 Consolidated-Undrained (CU) Triaxial Tests

Consolidated-undrained triaxial compression tests were conducted on selected samples of cohesive soils from the borings. All tests were conducted in general agreement with Test Method for Consolidated-Undrained Triaxial Compression Test on Cohesive Soils, ASTM D5567. When there were only limited volumes of soil sample suitable for testing, a staged CU test was conducted. These tests were conducted in general agreement with the procedure developed by Sridharan and Rao (1972). Effective stress and total stress cohesion intercept and friction angles were determined. The results are presented in Appendix B.

5.2.2.2 Cyclic Triaxial Tests

Selected cohesive soil samples were tested for their strain dependent shear modulus and damping. The tests were conducted in general accordance with Test Methods for the Determination of the Modulus and Damping Properties of Soils Using the Cyclic Triaxial Apparatus, ASTM D 3999. All tests were conducted at a frequency of 10 cycles per second (Hz). The tests were staged in that a single sample was tested at three different deformation levels. The resulting moduli and damping were then plotted as a function of the imposed strain. The results are presented in tabular form in Appendix B.

5.3 Base Rock Motion Determination

In a traditional earthquake hazard assessment project, the first step is to select a rock base ground motion at the site. This usually requires a site-specific seismic hazard analysis taking into consideration the characteristics of all the known earthquake sources (faults, zones, epicentral distance, geological condition, and background) that could affect the site. However, in the central United States there is a lack of recorded strong ground motion in the New Madrid area that can be used for such purposes. Therefore investigators in the research community have resorted to procedures that develop synthetic base rock motions at a site.

Dr. Robert Herrmann, Professor of Geophysics at St. Louis University, was requested to provide credible synthetic ground motions at both the St. Francis River Site and the Wahite Ditch Site.

5.3.1 Current Peak Ground Acceleration

The locations of both the bridge sites (St. Francis River Site (SF) and the Wahite Ditch Site (WD)) are shown in Figure 5.1 together with neighboring earthquake locations for the time period 1974-1995. The St. Francis River Site is about 37 - 150 km from possible earthquakes in the active part of the current seismicity zone, while the Wahite Ditch Site is about 15 - 150 km from active seismicity (Herrmann, 2000).

In the preparation of the 1996 National Earthquake Hazards Reduction Program (NEHRP) maps, the United States Geological Survey (USGS) considered other possible locations obtained by moving the 'Z' seismicity pattern westward slightly to the edge of the ancient right and eastward to the eastern boundary. They then assigned weights of 1/3 to each of the three patterns.

The USGS 1996 maps equally weighted two ground motion magnitude – distance relations: one based of the Toro and McGuire model for EPRI and the other a purely USGS model. The 1996 maps were generated for a nationwide NEHRPB-C soil condition boundary so that one could use the methodology in Federal Emergency Management Agency (FEMA) -273, for example, to adjust the mapped values to sites with other than the B-C soil condition in the upper 30 meters.

By entering a latitude and longitude at the USGS - National Seismic Hazard Mapping Project, the peak ground acceleration can be obtained (Table 5.1 Peak Ground Acceleration, Herrmann, 2000). However, Hermann, (2000) has not used these values in the final recommendations of base rock motions.

5.3.2 Magnitudes and Distances for the Recommended Acceleration Values

The magnitudes and distances for the study sites were selected from a table of ground motion parameters as a function of magnitude and distance (the USGS ground motion model enters into the hazard analysis code by a table lookup). The following acceptable combinations are shown in Table 5.2 (Herrmann, 2000).

5.3.3 Time Histories

Using the band-limited Gaussian white noise technique of Boore (1922), the program *DORVT180* and *TD_DRVR* were used together with auxiliary programs for display.

The Central United States (CUS) deep soil ground motion model with F96 (USGS96 source scaling) given on the CUS ground motion web page, with a soil thickness of 0 meters was used. Because the CUS model includes 1 km of Paleozoic layers, there is a slight frequency dependent site amplification. The model uses recently determined, CUS specific, crustal wave propagation from the source to the site (Appendix D).

The recommend rock base accelerations for a probability of exceedance (PE) of 10 % in 50 years and a PE of 2 % in 50 years for each site was obtained, i.e. 40-rock base synthetic ground motions (time histories) are available. Half of these were for PE of 10% in 50 years and other half were for PE of 2 % in 50 years.

The details of these rock base motions are shown in Appendix D. Selected sets of time histories of rock base acceleration-time histories are shown in Figures 5.2a, b, c, d and 5.3a, b, c, d. These will be used in all subsequent analysis as explained further.

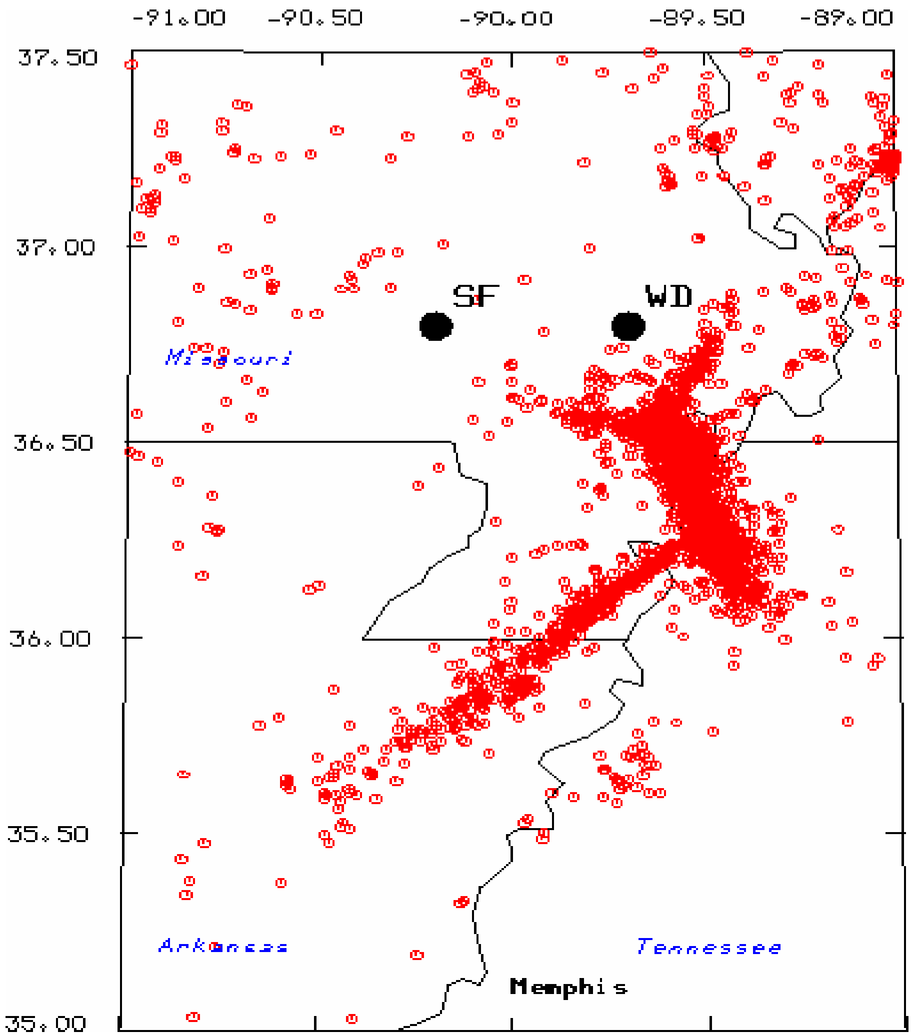


Figure. 5.1 Seismicity in the 1974 - 1995 Time Period in the Vicinity of the St. Francis River Site (SF) and the Wahite Ditch Site (WD).(Herrmann, (2000))

Table 5.1 Peak Ground Acceleration (Herrmann, 2000)
(Source; USGS 1996 Seismic Hazard Maps)

Site location	**Peak Ground Acceleration (g)**	
	10 % PE in 50 years	2% PE in 50 years
St. Francis River Site (36.8°N, 90.2°W)	0.158	0.643
Wahite Ditch Site (36.8°N, 89.7°W)	0.196	1.343

Table 5.2 Magnitudes and Distances for Selected Earthquakes, (Herrmann, 2000)

a. St. Francis River Site

Probability of Exceedance	Magnitude Mw	Distance, R (km)
10 % in 50 years	6.2	40
10 % in 50 years	7.2	100
2 % in 50 years	6.4	10
2 % in 50 years	8.0	40

b. Wahite Ditch Site

Probability of Exceedance	Magnitude Mw	Distance, R (km)
10 % in 50 years	6.4	40
10 % in 50 years	7	65
2 % in 50 years	7.8	16
2 % in 50 years	8.0	20

For the purpose of establishing procedures for the remainder of this earthquake engineering study, the following rationale was adopted to select what ground motion time history be used in the subsequent analysis.

All of the 20 ground motions were used for one-dimensional wave propagation analysis (using the *SHAKE* program, Schnabel, 1972) for each bridge site. This resulted in a profile of peak accelerations for each soil layer for both bridge sites. The three ground motions with highest peak horizontal ground acceleration (maximum PGA) at the surface were selected for the subsequence analyses.

5.4 Seismic Response of Soil

This section describes the procedures used to evaluate the response of the various site features to the selected simulated earthquakes.

5.4.1 Wave Propagation Analysis

Several methods for evaluating the effect of local soil conditions on ground response during earthquake are presently available. Most of these assume that the soil response is caused by the upward propagation of shear wave from the rock base. Analytical procedures, considering nonlinear soil behavior, have been used in the *SHAKE* (Schnabel, et. al. 1972) and *SHAKE91* (Idriss and Sun, 1991) computer programs.

The *SHAKE91* procedure generally involves the following steps:

1. Determination of the ground motion at the base rock for use in the analysis. This ground motion is a function of the maximum acceleration, effective duration, magnitude and epi-central-distance.

2. Determination of the dynamic properties of the soil deposit (shear modulus, mass density, shear wave velocity, etc.). Non-linear properties of several soils have been established for use in this and other analyses (Seed and Idriss, 1971, Vucetic and Dobry, 1991).
3. Computation of the response of the soil deposit to the rock-base motions using SHAKE91.

SHAKE91, with its pre- and post-processor *SHAKEDIT,* were used to propagate the horizontal rock-base motion to the soil layers, and were also used to transfer P-waves from the rock base to the above layers. Brief descriptions of these programs are presented in Appendix C.

5.4.2 Liquefaction Analysis

A universally accepted procedure of liquefaction analysis (Seed and Idriss, 1971 and Youd and Idriss, 1997) is as follows:

1. At a point in the soil mass, compute τ_{av} shear stress caused by the earthquake (base rock motion in Figures 5.2 and 5.3) using equation 5.1:

$$\tau_{av} = 0.65 \bullet \left(\frac{a_{\max}}{g}\right) \bullet \sigma_o \bullet r_d \qquad (5.1)$$

τ_{av} may be expressed as the Cyclic Stress Ratio (CSR) (equation 5.2),

$$CSR = \frac{\tau_{av}}{\sigma_o} = 0.65 \bullet \left(\frac{a_{\max}}{g}\right) \bullet \left(\frac{\sigma_o}{\sigma_o'}\right) \bullet r_d \qquad (5.2)$$

where,

a_{max} = peak horizontal ground acceleration at that surface. a_{max} is considered constant throughout the entire depth.
g = acceleration due to gravity
σ_o = total vertical overburden stress
σ_o’ = effective vertical overburden stress
r_d = stress reduction coefficient

r_d has been expressed as a function of depth below the ground level z, as (Youd and Idriss, 1997):

$$r_d = \frac{\left[1 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5}\right]}{\left[1 - 0.4117z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.00121z^{2}\right]} \qquad (5.3)$$

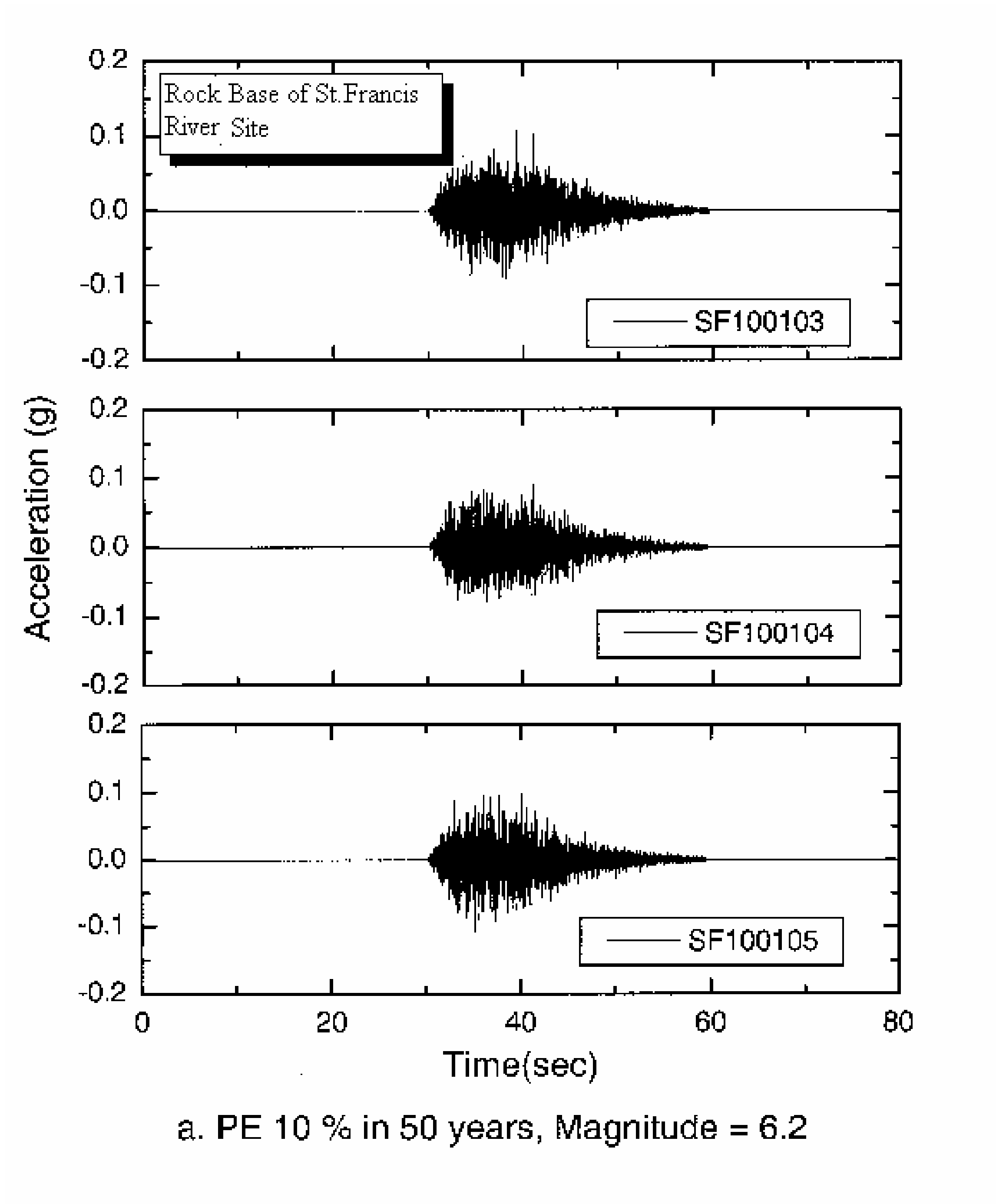


Figure 5.2.a The Selected Base Rock Motion for the St. Francis River Site, PE 10 % in 50 Years, Magnitude 6.2

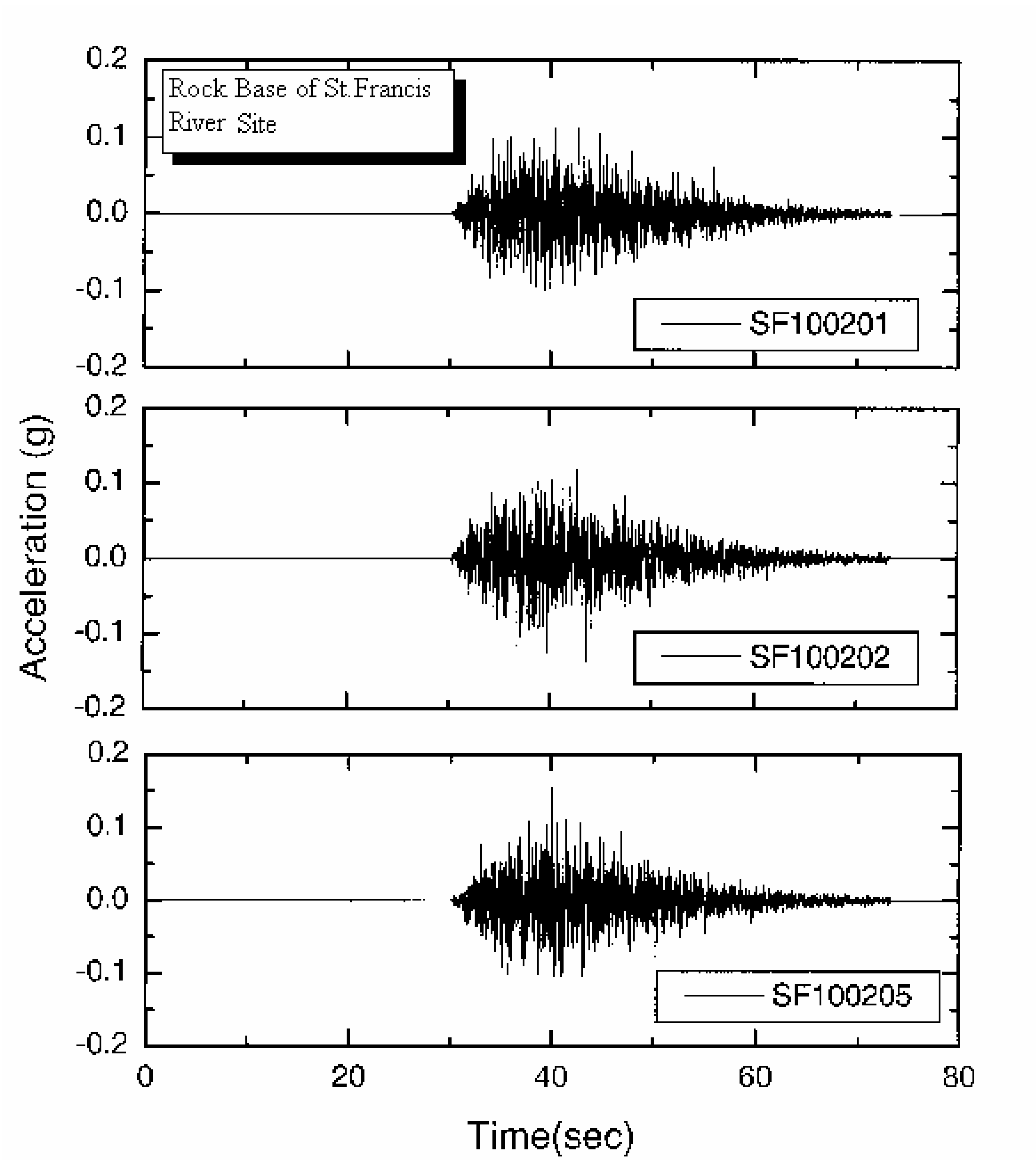


Figure 5.2.b The Selected Base Rock Motion for the St. Francis River Site, PE 10 % in 50 years, Magnitude 7.2

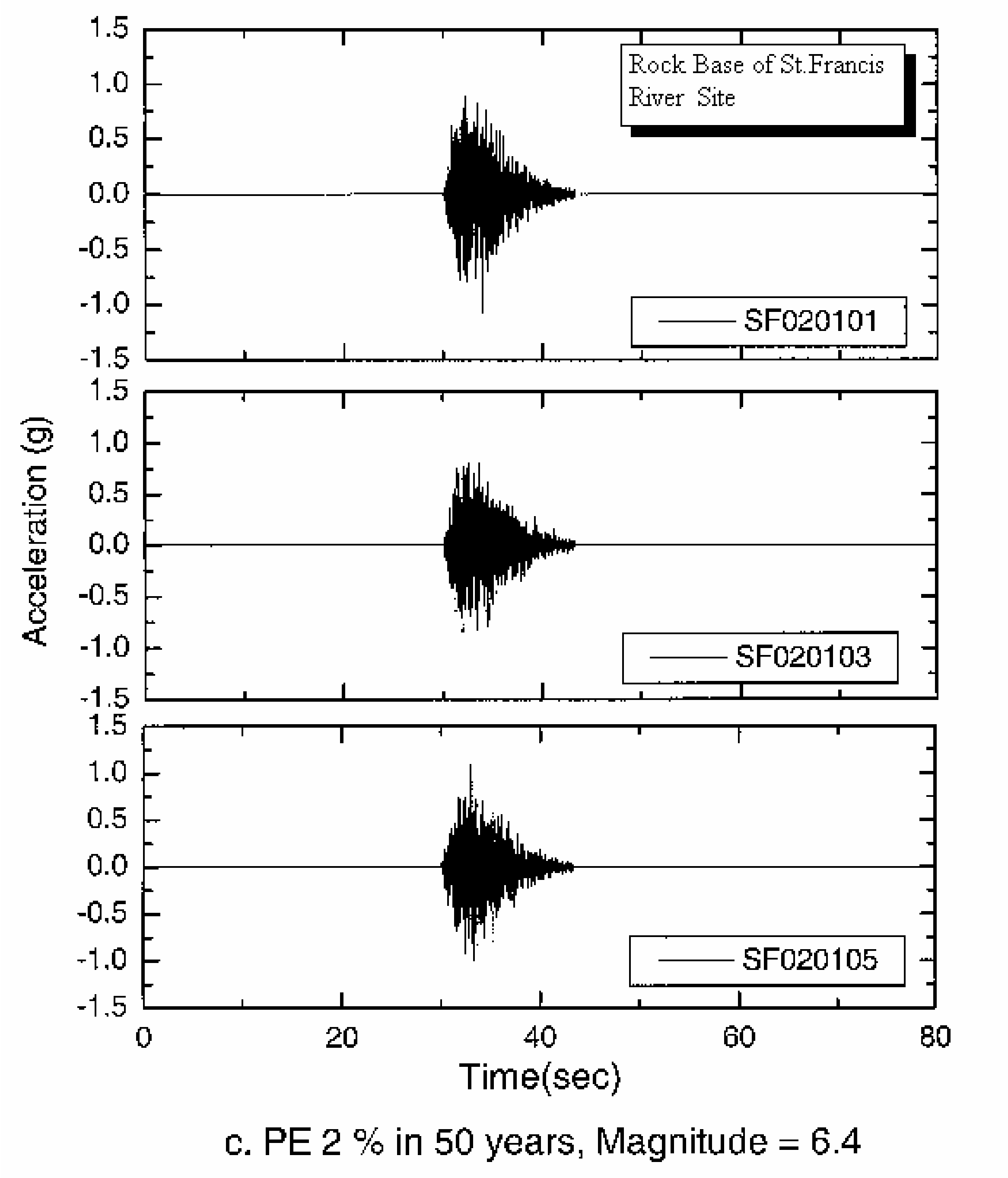
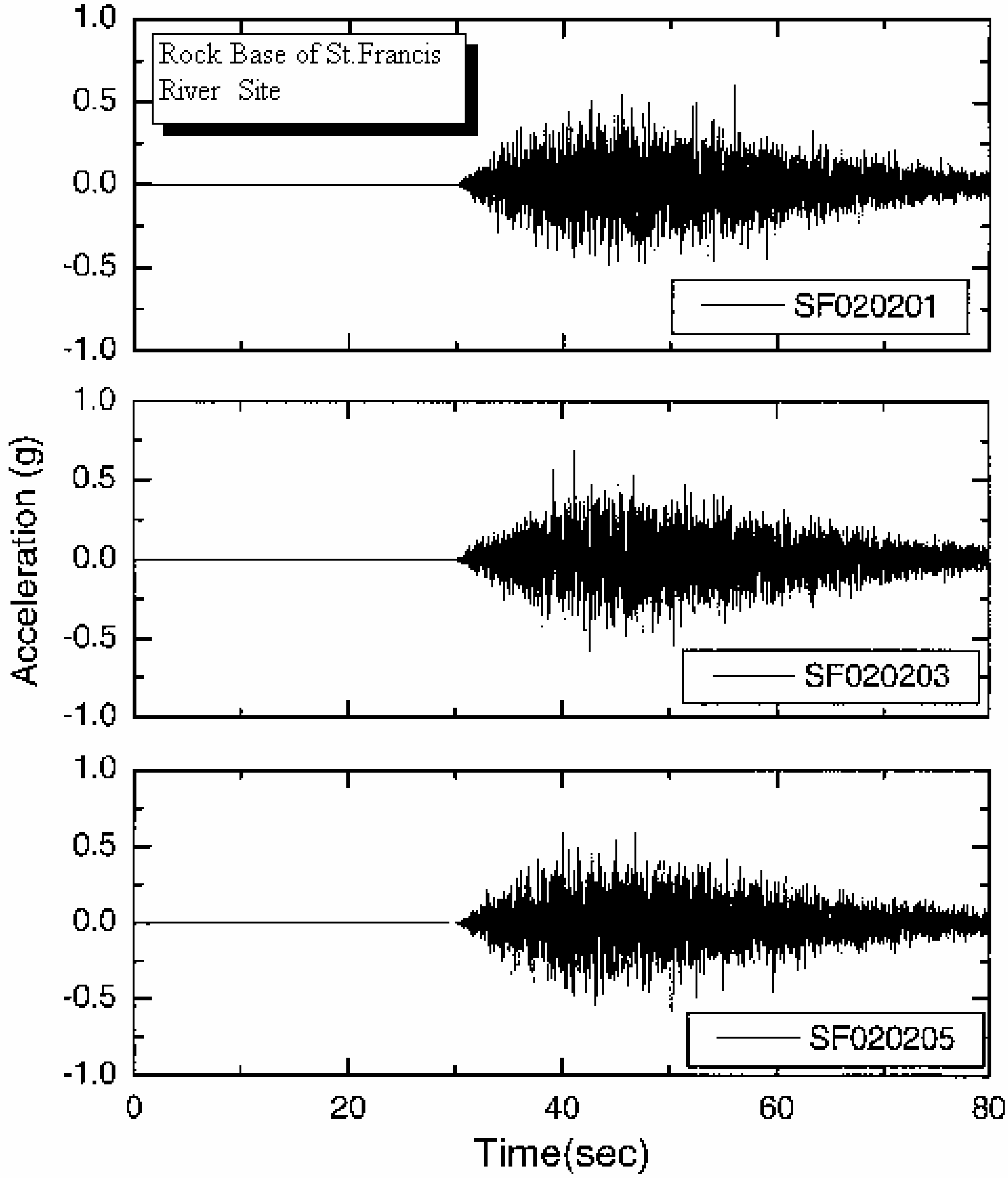


Figure 5.2.c The Selected Base Rock Motion for the St. Francis River Site, PE 2 % in 50 years, Magnitude 6.4



d. PE 2 % in 50 years, Magnitude = 8.0

Figure 5.2.d The Selected Base Rock Motion for the St. Francis River Site, PE 2 % in 50 years, Magnitude 8.0

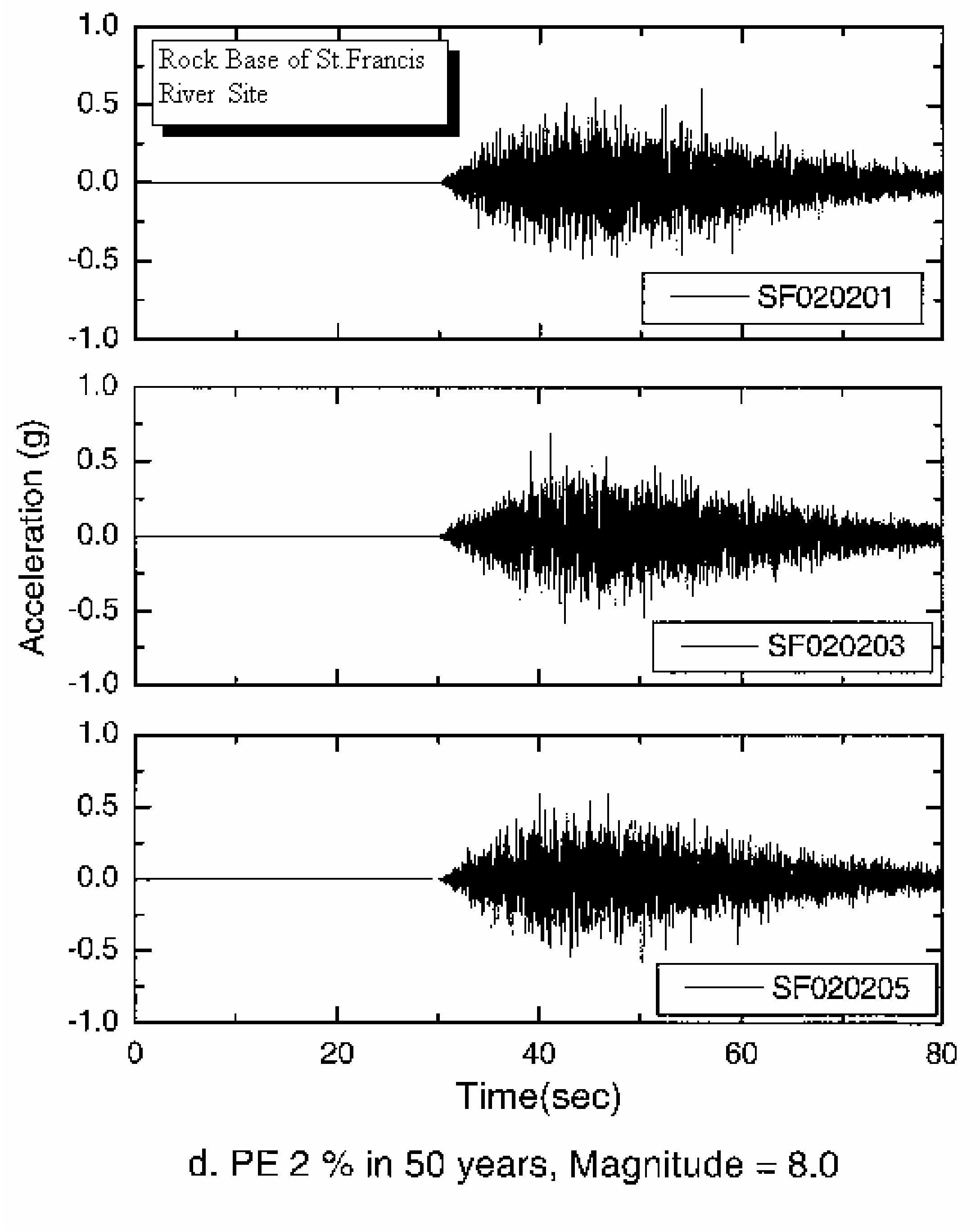


Figure 5.3.a The Selected Base Rock Motion for the Wahite Ditch Site, PE 10 % in 50 years, Magnitude 6.4

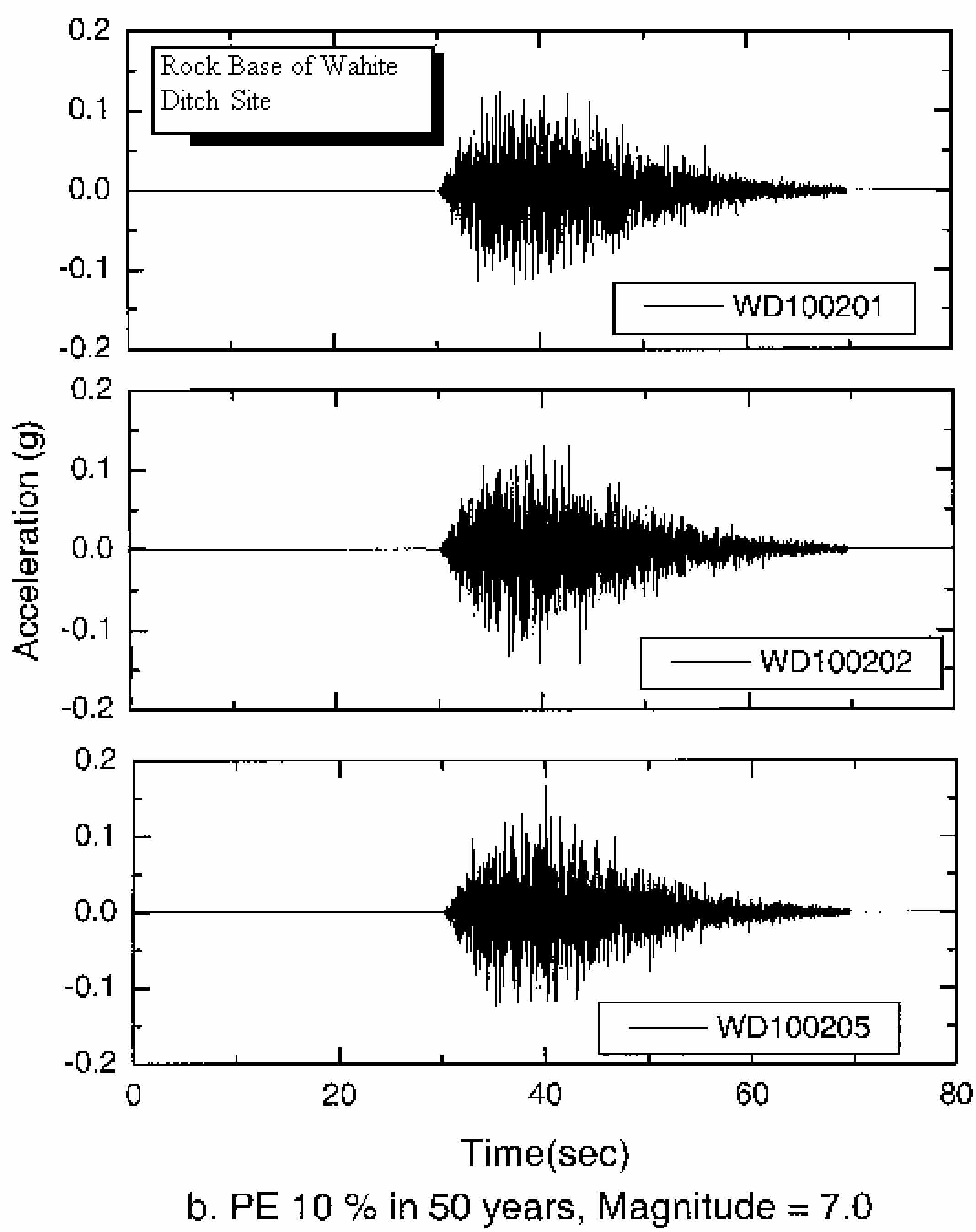


Figure 5.3.b The Selected Base Rock Motion for the Wahite Ditch Site, PE 10 % in 50 years, Magnitude 7.0

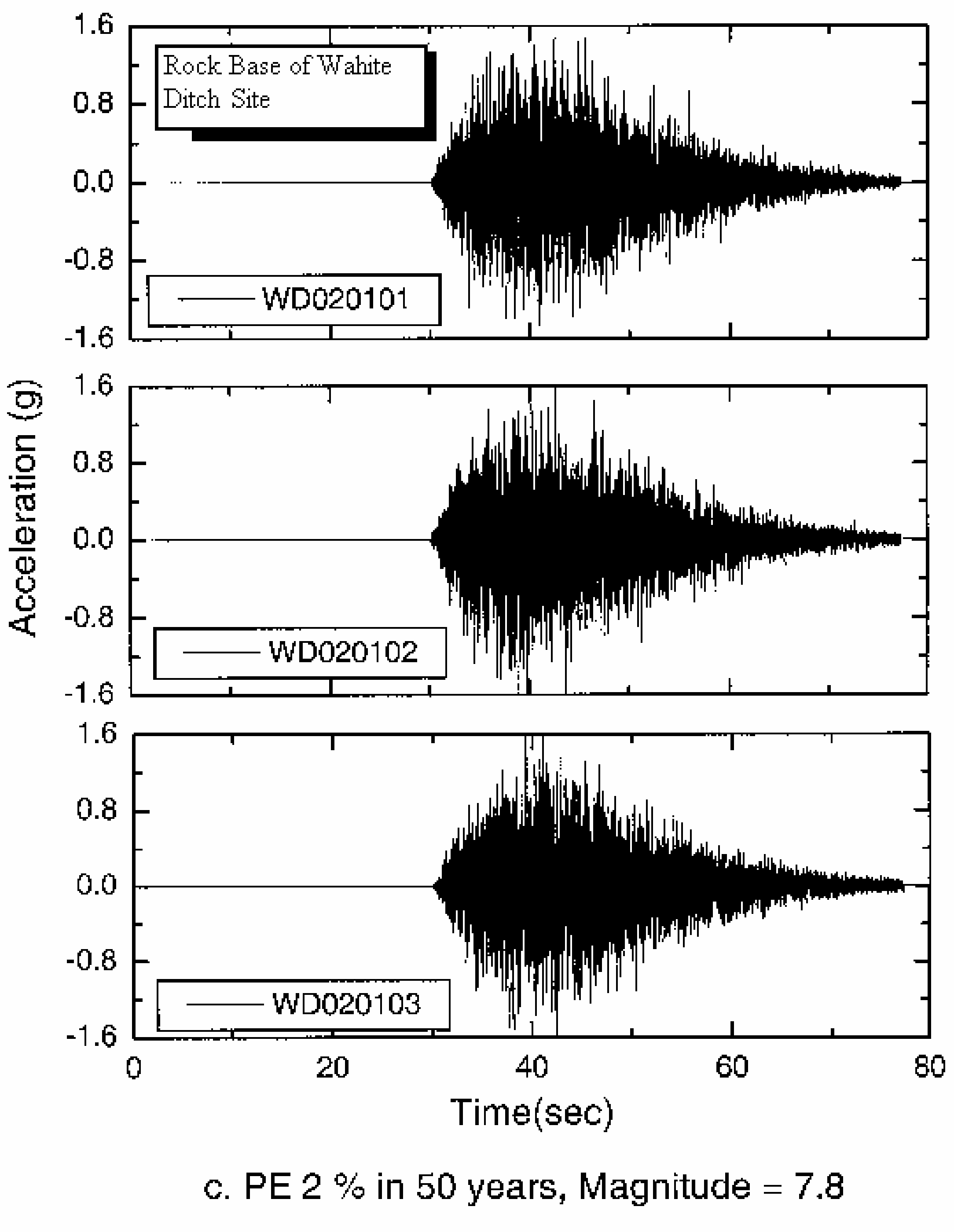


Figure 5.3.c The Selected Base Rock Motion for the Wahite Ditch Site, PE 2% in 50 years, Magnitude 7.8

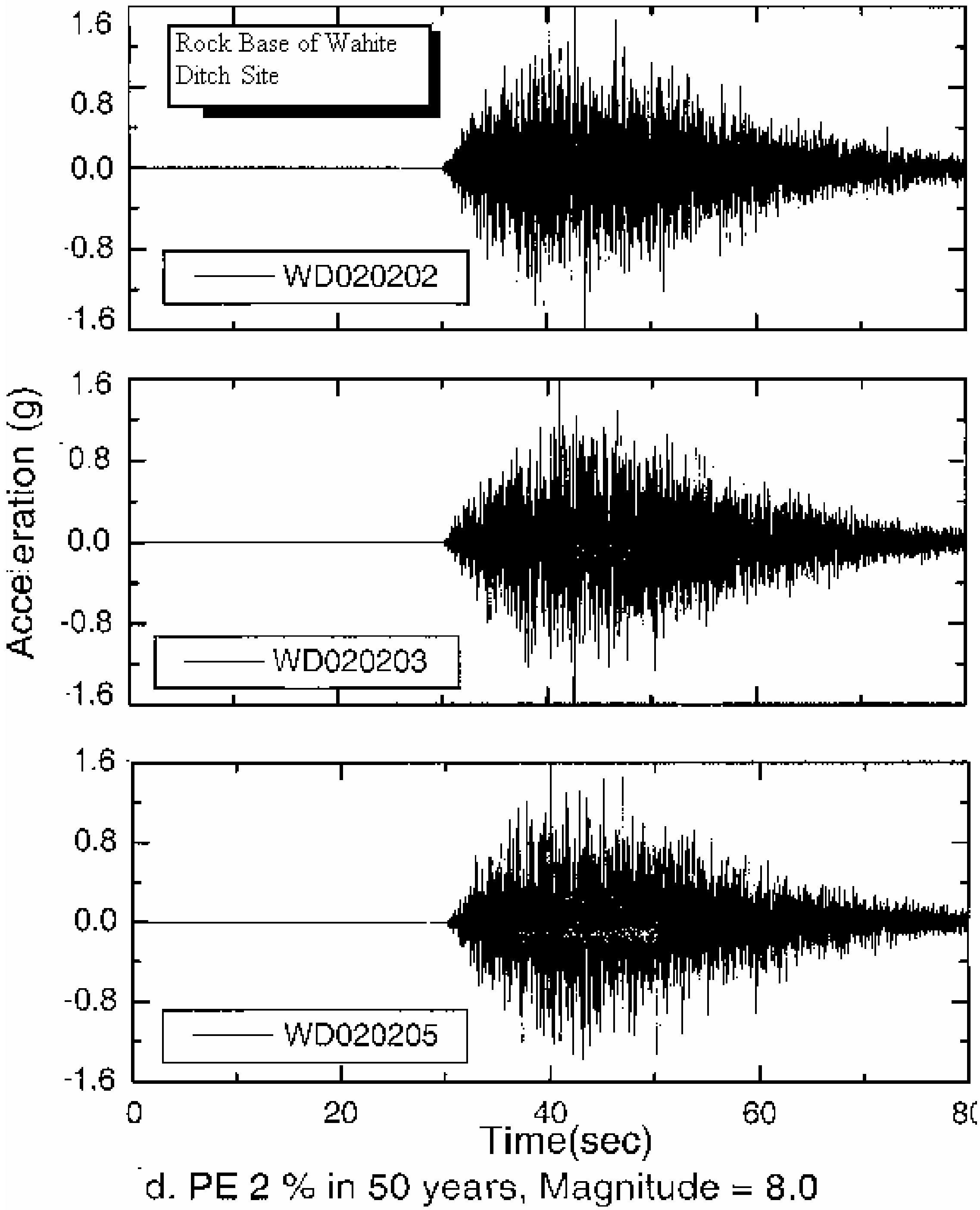


Figure 5.3.d The Selected Base Rock Motion for the Wahite Ditch Site, PE 2 % in 50 years, Magnitude 8.0

2. Estimate τ_{liq}, the shear strength to cause liquefaction at the above point under the ground motion (Figures 5.2 and 5.3).

   τ_{liq} is also expressed as cyclic resistant ratio (CRR) i.e. τ_{liq}/σ_o' at the above point. A relationship with τ_{liq}/σ_o' and corrected $(N1)_{60}$ for earthquake magnitude 7.5 is in Figure 5.4.

   The standard penetration test values NM are converted to $(N1)_{60}$ by correcting for energy and other factors as below (equation 5.4).

$$(N1)_{60} = NM\ CN\ CE\ CB\ CR\ CS \tag{5.4}$$

   where,
   NM = Observed SPT value
   CN = factor to correct NM for overburden pressure
   CE = Correction for hammer energy ratio
   CB = Correction for borehole diameter
   CR = Correction for rod length
   CS = Correction for samplers with or without liners

3. The factor of safety (FOS) against liquefaction is computed as:

$$FOS = \tau_{liq}/\tau_{av} \tag{5.5a}$$

   or

$$FOS = CRR/CSR \tag{5.5b}$$

In this manner, τ_{av} (or CSR) and τ_{liq} (CRR) are computed along the depth of a profile at several points and the factors of safety of a deposit are evaluated.

Modifications to τ_{av} in the SHAKE Program

1. The SHAKE program is used to analyze the wave propagation from base rock up to surface layer.

2. The output of SHAKE program includes peak acceleration of each soil layer.

3. This peak acceleration (a_{max}) is used to compute τ_{av}. This may give slightly different value of τ_{av} as compared to their result using equation 5.1.

The Seed and Idriss simplified method (1971), as modified by Youd and Idriss (1997) was used in the liquefaction potential analysis of this project.

5.5 Slope Stability of Abutment Fills

Seven cross-sections from the St. Francis River Site were selected for slope stability analysis (Figure 5.5), as were seven from the Wahite Ditch Site (Figure 5.6). At both sites, the cross-sections represented the steepest site slopes. The cross-sections were developed from the topographic maps created by Missouri Department of Transportation and from the subsurface information obtained by drilling and cone penetrometer soundings. The cross-section data was then entered into the slope stability program *PCSTABL5* using the pre and post processor *STEDwin*. The slopes were analyzed under static and pseudostatic conditions using the Modified Bishop Method.

5.5.1 Soil Property Estimation

The soil properties needed for *PCSTABL5* analysis were estimated using a conservative approach. Wet unit weight, saturated unit weight, cohesion and internal angle of friction were estimated by correlation with SPT values, Cone Penetration Tests (CPT), Missouri Department of Transportation and University of Missouri-Rolla laboratory tests, and several technical references. Specifically, values were estimated as follows:

- The $(N1)_{60}$, CPT, and laboratory data values were matched at various depths and compared for consistency.
- The density condition of the soil was based on correlations with $(N_1)_{60}$ and CPT (McCarthy, 1998; Meigh, 1987). The relative density (D_r) was based on correlations with $(N_1)_{60}$ and CPT (Hunt, 1984; Meigh, 1987).
- The dry unit weight of granular soils was determined from relative density (McCarthy, 1998). Dry unit weight could not be measured directly in the field or laboratory for granular soils.
- The void ratio of granular soils was calculated from the minimum and maximum dry unit weight of silty sand (Lambe and Whitman, 1969).
- For clays the equation below was used to determine the void ratio:

$$e = \frac{\gamma_{water}}{\gamma_{dry}} - 1 \qquad (5.6)$$

  The Missouri Department of Transportation assumed G_s of 2.67, and the dry unit weight of the clay was obtained from Missouri Department of Transportation laboratory tests.

- The wet and saturated unit weight of the soil was determined by using equation 5.7:

$$\gamma = \frac{(G_s + S(e))\gamma_{water}}{(1+e)} \qquad (5.7)$$

  The degree of saturation was set equal to 50% (S = 0.5) for the wet unit weight of soil and equal to 100% (S=1) for the saturated unit weight of soil.

- Internal angle of friction for clays was found from cone resistance and friction ratio (Meigh, 1987).

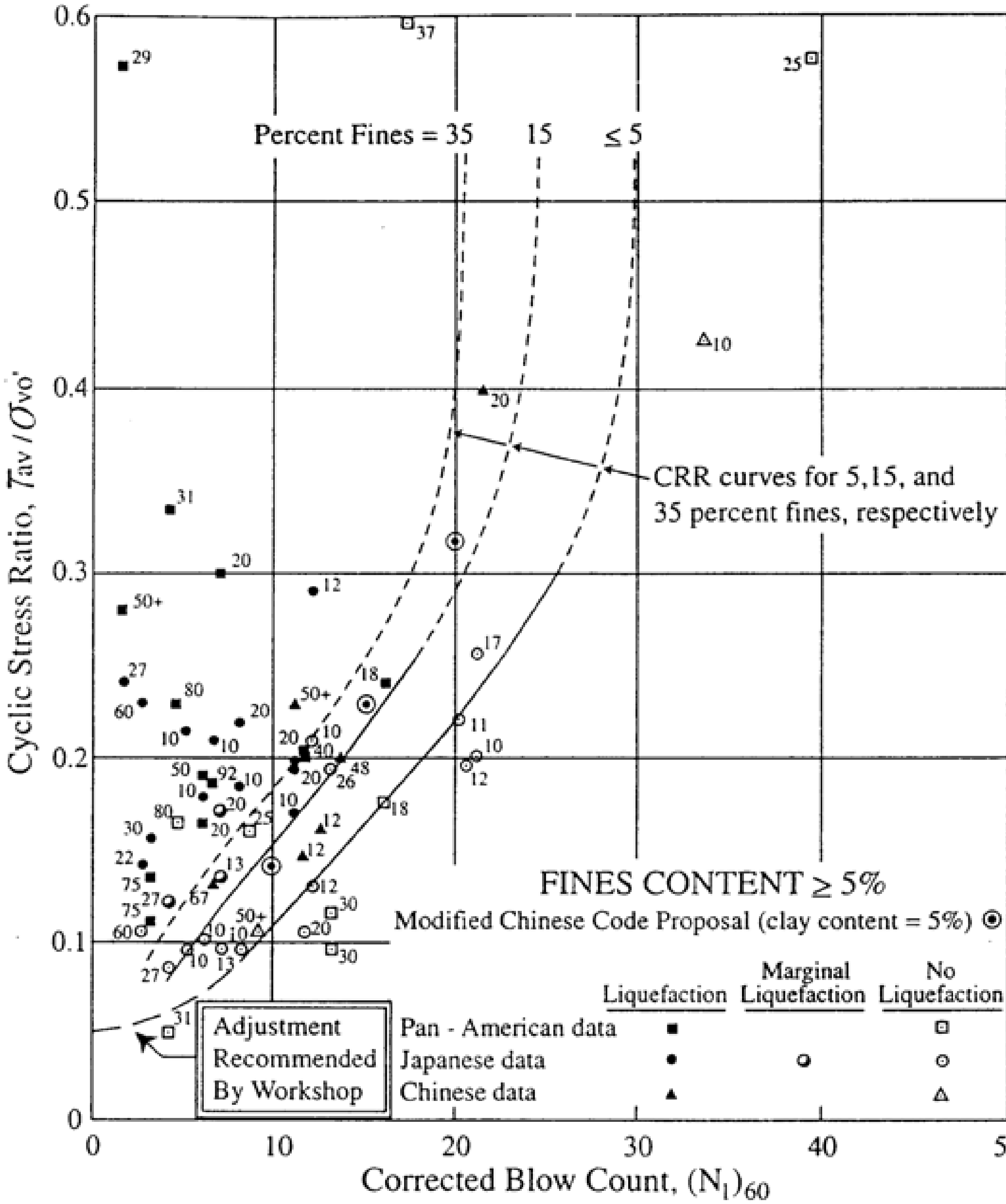


Figure. 5.4 Simplified Base Curve Recommended for Calculation of CRR From SPT $(N_1)_{60}$ Data Along With Empirical Liquefaction Data for M=7.5 (Seed et. al., 1971, modified by Youd and Idriss, 1997)

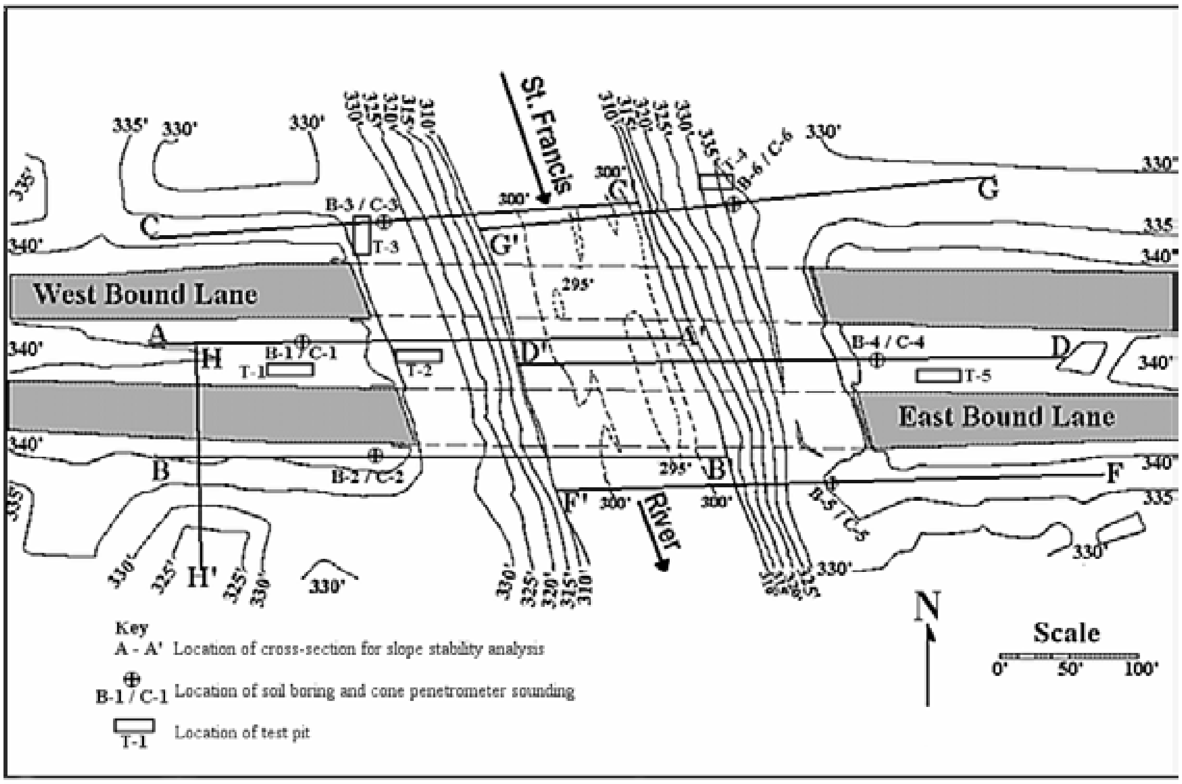


Figure 5.5 St. Francis River Site Topography, Cross-Sections and Boring Locations

- For cohesionless soils, $(N_1)_{60}$ values were used to determine the internal angle of friction (McCarthy, 1998).
- Cohesion was determined from the torvane and laboratory tests conducted by the Missouri Department of Transportation.

The soil properties obtained through this procedure were then averaged for each stratigraphic unit at the St. Francis River Site and the Wahite Ditch Site

5.5.2 Groundwater Elevation Selection

Two groundwater elevations were selected for the stability analysis at each slope: a low water condition and a high water condition. The low water condition was based on the water elevations measured by Missouri Department of Transportation and University of Missouri-Rolla in September and October of 1999. These elevations were anticipated to be lowest reasonable levels due to the lack of rain for the preceding 6-8 weeks. The high water condition was estimated from the height of the water staining on the bridge piers and was expected to be the highest reasonable groundwater elevation, representing levels during a prolonged wet season and flooding event. The two groundwater elevations were then used to conduct the static and dynamic slope stability analysis at the study sites.

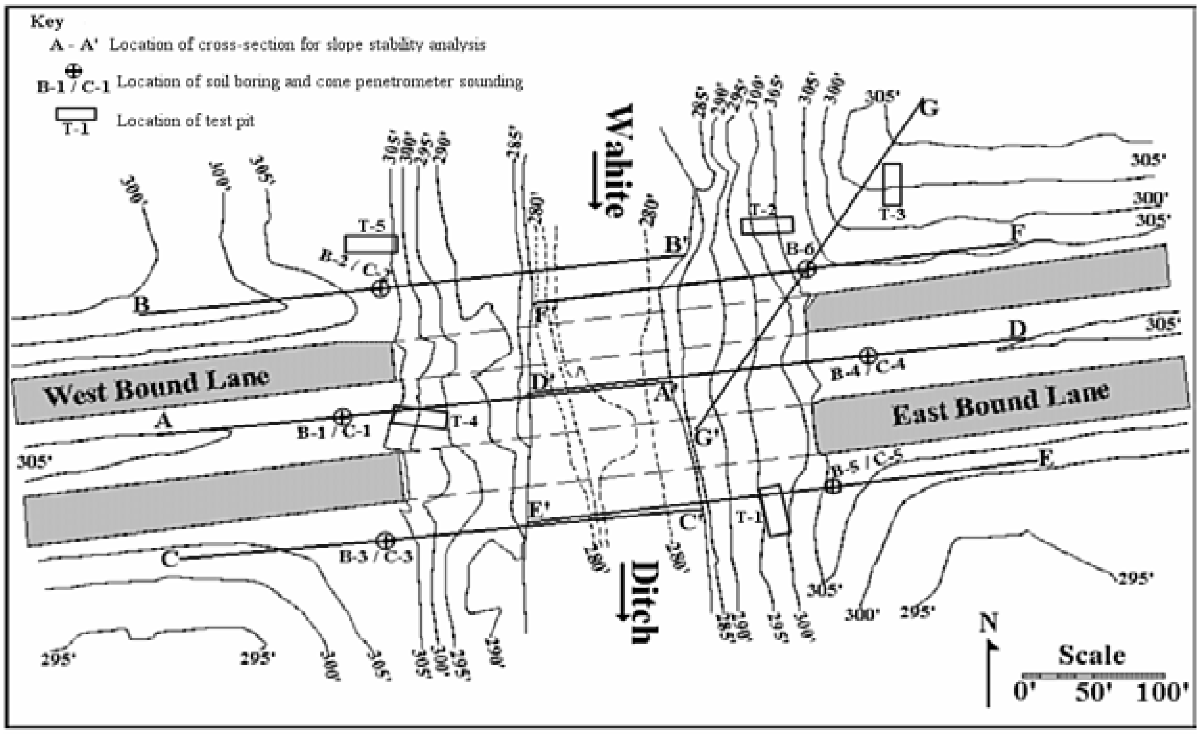


Figure 5.6 Wahite Ditch Site Topography, Cross-Sections and Boring Locations

5.5.3 Design Horizontal and Vertical Earthquake Accelerations

Three sets of ground accelerations were selected for the St. Francis River Site and the Wahite Ditch Site based on the *SHAKE91* analysis. The acceleration sets covered the following conditions:

1. Adjusted Peak Horizontal Ground Acceleration (PHGA). PHGA was selected for each bridge site as the maximum horizontal acceleration from the ten synthetic time records. PHGA was adjusted to a level of 66% of the original value. While Kramer (1996) and other researchers recommend using lower values (on the order of 50%), it is prudent to conduct a slightly more conservative analysis, since the effects of transient pore-water pressures are not accounted for in the analysis, and the strength and density values used in the analysis were obtained, in part, from published correlations.

2. Adjusted PHGA with corresponding Adjusted Peak Vertical Ground Acceleration (PVGA). For this set of analyses, the Adjusted PHGA values from set 1 were used along with the corresponding PVGA occurring at the same time as the PHGA in the synthetic time records. The PVGA values were adjusted using the modification factors described in Section 8.1.3.3 and 8.2.3.3 (C = 66% * PHGA/PVGA), and then adjusted to 66% of the modified value. Both positive and negative vertical ground accelerations were analyzed.

3. Adjusted PVGA with corresponding Adjusted PHGA. For this set of analyses, the PVGA was selected as the maximum vertical acceleration from the same time record as set 1, used with the corresponding PHGA occurring at the same time. The values were adjusted as described for set 2.

Each set above used acceleration values for earthquakes with 2% and 10% exceedance probabilities in 50 years. The selected design horizontal accelerations were used in *PCSTABL5* to represent pseudo-static earthquake conditions, for both low and high ground water. The adjusted accelerations are summarized below in Table 5.3.

5.6 Flood Hazard Analysis

Flood hazards were estimated assuming that an earthquake caused catastrophic failure of waterway levees in the vicinity of US 60 or failure of the Wappapello Dam, located approximately eight miles north of US 60.

Eleven 7.5-minute topographic maps were collected to map the areas that would be affected by flooding if local levees failed during an earthquake. The flooding analysis consisted of the following procedures:

- River, creek, and drainage ditch locations, approximate elevations of water levels, and approximate elevations of both natural and man-made levees flanking the waterways were identified.
- Zones on the topographic maps were subdivided along the roadway by 5-foot contour intervals.
- Areas where the land was below water levels in waterways were marked as zones of potential flooding. Each area was field checked to visually assess the elevation of the roadway compared to surrounding land.

An evaluation of the effects of catastrophic failure of the Wappapello Dam was completed by U.S. Army Corps of Engineers (1985). Flood maps presented in the U.S. Army Corps of Engineers report are summarized in Section 8.1.6

Table 5.3 Design Horizontal and Vertical Earthquake Accelerations

a. St. Francis River Site

	Set 1		**Set 2**		**Set 3**	
Earthquake	**HGA**	**VGA**	**HGA**	**VGA**	**HGA**	**VGA**
10% PE	0.135	0	0.135	±0.048	0.012	±0.090
2% PE	0.331	0	0.331	±0.170	0.014	±0221

b. Wahite Ditch Site

	Set 1		**Set 2**		**Set 3**	
Earthquake	**HGA**	**VGA**	**HGA**	**VGA**	**HGA**	**VGA**
10% PE	0.123	0	0.123	±0.006	0.008	±0.082
2% PE	0.350	0	0.350	±0.007	0.060	±0.233

6.0 PROCEDURES FOR SEISMIC CONDITION ASSESSMENT OF BRIDGES AND ABUTMENTS

This section describes the complete procedure used to assess the condition of highway bridges based on the site-specific seismicity. The work delineated in this study includes the setup of computer models of the study bridges assuming rigid abutments and foundations, seismic analysis of the bridges under site specific ground motions, preliminary evaluation of various structural components, and comparison between the AASHTO spectrum and site-specific ground motions.

A set of performance goals, relating to the responses of the bridge to earthquakes of specified hazard level, are established for condition assessment. The performance goals are achieved through computer modeling and analyses using engineering criteria concerning the evaluation procedures of structural components. These criteria include the specification of acceptable levels of damage that meet the global performance goals as defined by AASHTO.

6.1 Global Performance Goals

Global performance goals are generally defined to set the criteria for acceptable performance during an earthquake event. These goals are described below.

6.1.1 American Association Of State Highway and Transportation Officials Specifications

The performance goals in the current AASHTO Specifications Division I-A, Seismic Design, are as follows:

- Small to moderate earthquake should be resisted within the elastic range of the structural elements without significant damage.
- Severe earthquakes should not cause collapse of all or part of a bridge. Damage that does occur should be readily detectable and accessible for inspection and repair.

Based on the AASHTO Specifications, bridges are designed only for one level of seismic hazard representing a severe event (the 500-year return period event). For that hazard level, damage must be limited, but not necessarily eliminated. The implication of this methodology is that the structure will sustain minimal or no damage for small to moderate earthquakes if the design is performed for a severe earthquake.

6.1.2 Bridges Along US 60

Many studies have shown that the seismic hazard in the Midwest increases considerably for an earthquake with a return period of 2,500 years instead of 500 years as specified by AASHTO. Bridges along the designated emergency vehicles access routes are needed the most to allow rescue teams and necessary rescue equipment to pass through for saving the lives in the stricken areas in case of a devastating earthquake event.

In principle, it is possible to eliminate any damage occurring from earthquakes, if bridges are upgraded for the maximum credible earthquake. However, the unreasonably high costs of such a retrofit as well as budgetary constraints often dictate a more realistic approach that would significantly reduce cost, but as a tradeoff would permit some damage under certain conditions.

The recommended performance goals of a bridge that could satisfy the above requirements are:

1. For the low hazard level (10% probability of exceedance in 50 years or approximately 500-year return period), the bridge shall be capable to carry normal traffic almost immediately after the earthquake. Damage shall be minimal and mostly limited to secondary structural elements.

2. For the high hazard level (2% probability of exceedance in 50 years or approximately 2500-year return period), the bridge shall be able to carry limited traffic within days (reduced lanes, light emergency traffic). The goal is to avoid collapse.

3. It is clear that the above-stated goals involve policy issues concerning the desirable level of service after an earthquake as well as the financial expenditures necessary to achieve this service. These issues are of socioeconomic nature and need to be addressed by the Missouri Department of Transportation as a matter of public policy. The present condition assessment provides the background information in terms of vulnerability of the bridge in terms of the expected level of service.

6.2 Engineering Performance Criteria

Earthquakes in the Midwest are characterized as infrequent but high consequence events. Once it occurs, an earthquake often induces larger forces in structural members (especially vertical structural components) than other dead and live loads. Economic considerations dictate that structural components resist earthquake loads using the available capacity in the inelastic range of their response. Thus ductility, or the ability of structural members to deform inelastically without loss of strength, is an important consideration in the response of structural components to seismic loads.

The established engineering performance criteria address a range of issues including the procedures for performance assessment, seismic demand and capacity, and acceptable damage. They are based on the AASHTO Specifications and the FHWA Seismic Retrofitting Manual for Highway Bridges (1995).

6.2.1 Performance Assessment

The seismic performance of the bridge is determined on the basis of the performance of its components. For all critical structural components, a capacity over demand (C/D) ratio is determined for each potential mode of failure. The lowest C/D ratio value of each component indicates the controlling mode of failure. A C/D ratio less than one implies a vulnerable component. Potential damage of groups of components is also considered to reflect the performance of overall structural systems.

The C/D ratio is defined as:

$$C / D = \frac{R_c - \sum Q_i}{Q_{EQ}} \tag{6.1}$$

where R_c is the ultimate force or deformation capacity of the component for the mode of failure under consideration. ΣQ_i is the sum of the force or deformation demands for loads other than earthquake loads (dead loads, live loads, etc.). Q_{EQ} is the earthquake force or deformation demand.

When components such as columns and pile caps resist failure in a ductile mode, the C/D ratios are multiplied by ductility indicators (μ>1) to enable the use of the elastic analysis results. For non-ductile modes of failure (buckling of bracing members and shear of beams, etc.), the ductility indicators are taken equal to one. For bridge bents with multiple columns, a ductility indicator of 5.0 is used. The ductility indicators of other members can be found in the FHWA Manual (1995).

6.2.2 Seismic Demand

Structural analysis was performed using the computer program *SAP2000*. The method of analysis of the bridge is the time-history analysis procedure and response spectrum analysis with uniform support excitations. The soil-structure interaction was taken into account by including several springs, representing soil flexibility. Nonlinear soil properties were accounted for by selecting strain compatible constants of the springs.

The computation of seismic demand was based on three-dimensional computer models of the bridge. These models were developed to the extent that the essential characteristics of the structure were adequately represented and the response of the bridge sufficiently predicted. They describe the as-built condition of the bridge.

The seismic forces to be resisted by the pile caps were determined based on the smaller forces due to the column over-strength capacity and the elastic analysis results. For the purpose of determining the over-strength capacity of columns, the nominal strength of reinforced concrete was increased by a factor of 1.30.

The seismic demands were calculated at two hazard levels for the site specific evaluation. They were also determined under the AASHTO design spectrum for comparison with the site-specific assessment. The well-known Complete Quadratic Combination (CQC) combination rule was applied to combine the effect of all vibration modes. The effect of different earthquake components were combined using the “30 percent rule” specified in AASHTO Specifications for peak responses.

6.2.3 Seismic Capacity

The current FHWA Seismic Retrofit Manual for Highway Bridges Guidelines (1995) are used to determine the capacity of concrete structure components.

6.2.4 Acceptable Damage

Examples of acceptable damage at low hazard levels for the AASHTO earthquake include:

- Damage to non-structural components.
- Limited cracking and spalling of the concrete columns.
- Some yielding of columns.
- Sign of yielding of member connections.
- Some damage to expansion joints.

Examples of acceptable damage at high hazard levels include:

- Serious damage to non-structural components.
- Cracking and spalling of the concrete columns.
- Yielding of columns.
- Yielding of member connections.
- Damage to bracings.

6.3 Analysis Procedures

This study focused on the condition assessment of both the superstructure (deck, girder, bearing, etc.) and the substructure (abutment, cap beam, column, and pile cap) for the selected bridges.

6.3.1 Computer Modeling of Bridges

One computer model was prepared for the analysis of each bridge. The concrete deck is simulated using shell elements while the remaining structural components are modeled as frame elements. To match the elevation of the actual bridge (deck and bearing), rigid dummy elements were introduced. Soil flexibility was represented by a set of springs at the centroid of pile caps at both interior bents and abutments. More detailed information on the computer model is discussed in Section 8.1.7.

6.3.1.1 Design Ground Motions

The bedrock motions for the analysis of the bridge were obtained from the procedures described in Section 5.3. They were used to determine the time-history response at the centroid of the pile caps based on one-dimensional seismic wave propagation from the *SHAKE* program as discussed in Section 5.4. The ground motion at one pile cap is considered as input for the entire bridge model.

6.3.1.2 Analysis Procedure

After the computer model was checked for connectivity, the seismic demand of structural members was computed based on the following procedure:

Step 1. The stiffness constants of springs at the pile caps of interior bents and abutments were estimated.

Step 2. The bridge model was analyzed under a longitudinal, transverse, vertical earthquake excitation, respectively.

Step 3. The effects of longitudinal, transverse and vertical earthquake excitations were combined.

Step 4. The compatibility between the load and displacement of the springs at all pile caps were checked. If they were not compatible, the stiffness constants were revised and Steps 1-3 were repeated until compatibility was satisfied.

6.3.2 Computer Modeling of Abutments

The abutments of all four bridges are supported on piles. However, only the abutments of the Old St. Francis River Bridge and the Old Wahite Ditch Bridge support the deck in simple support. The decks and the abutments of the two new bridges are constructed integrally with the bridge deck.

Figure. 6.1 depicts a typical non-integral bridge abutment supported on piles. Choudhry (1999) and Wu (1999) have proposed methods to calculate displacements of bridge abutment and retaining wall due to earthquake. The bridge abutment in their model was considered as a two degree of freedom model (Figure. 6.2). Based on this model, displacements of bridge abutment may occur in translation and rotation. A modification is used in this analysis to predict response of a bridge abutment supported on piles. The stiffness and damping factors due to pile-soil interaction are calculated by Novak's (1974) model (Appendix F).

Figure 6.3 shows the forces acting on the bridge abutment. These forces consists of:

1. The vertical seismic force increment (V_1) is

$$V_1 = k_v W \tag{6.2a}$$

where:
k_v = vertical seismic coefficient
W = weight of the abutment.

The vertical force may act in the positive (+) or negative (-) direction. The case that gives maximum displacement was adopted.

The point of application of V_1 is the center of gravity of the abutment and the horizontal distance from this point to the heel of the abutment is expressed as x_1 Figure 6.3.

The horizontal force (H_1) due to weight (W) of the abutment is computed as:

$$H_1 = k_h W \tag{6.2b}$$

where:

k_h = horizontal seismic coefficient

The height of the line of action of H_1 is at the centroid of the abutment, z_1 from the bottom.

2. The vertical seismic force increment, V_2, applied to the abutment is

$$V_2 = k_v Q \tag{6.3a}$$

where:

Q = Weight of the girder and traffic load acting on the bearing

The vertical force may act in the positive (+) or negative (-) direction. The case that gives the maximum displacement was adopted. The point of application of V_2 is the center of the bearing and the horizontal distance from this point to the heel of the abutment is expressed as x_2.

The horizontal seismic force H_2 of the girder is:

$$H_2 = k_h Q \tag{6.3b}$$

The height of the line of action of H_2 is assumed to be coincident with the upper surface of the bearing and at a distance z_2 from the bottom of the abutment.

3. The seismic force due to the weight of earth (W_s) ABCE (Figure 6.3) is given below with the point of application at the centroid (x_3, z_3) of the earth mass:

$$V_3 = k_v W_s \tag{6.4a}$$
$$H_3 = k_h W_s \tag{6.4b}$$

The earth pressure acting on the abutment is the sum of the earth pressure acting on the vertical line DE and the weight of soil mass ABCE and the seismic force. The earth pressure increment acting on the vertical line DE is calculated by the Mononobe-Okabe method. Its point of application is at 1/2 of the height of the line ED and the direction is inclined δ (Section 6.3) to normal on ED.

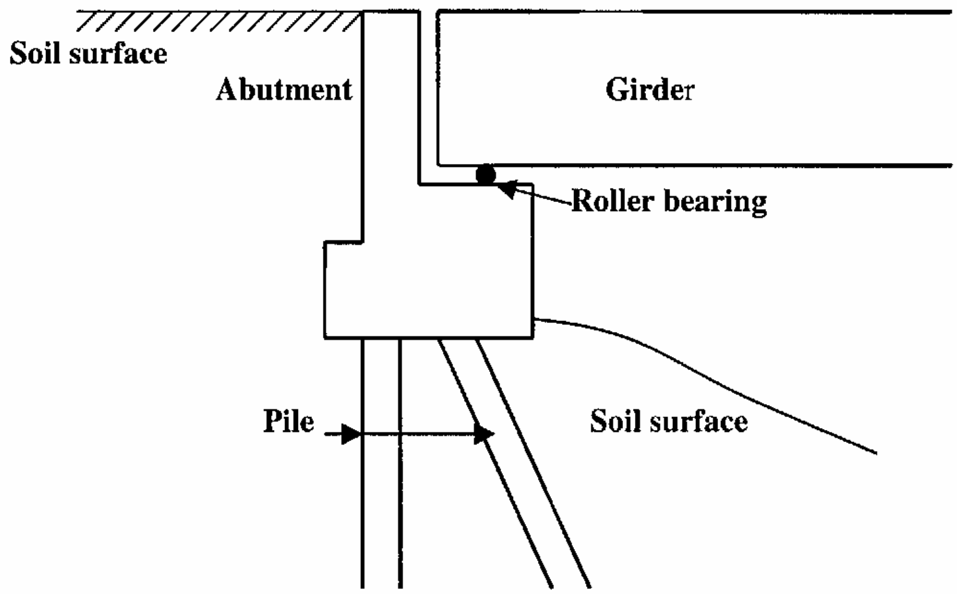


Figure. 6.1 Typical Non-Integral Bridge Abutment Supported on Piles

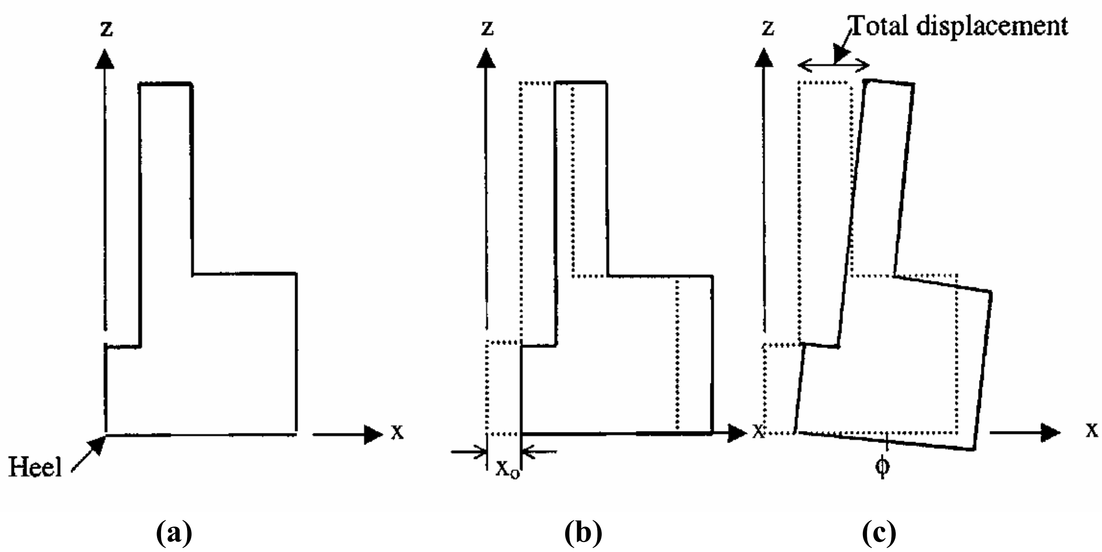
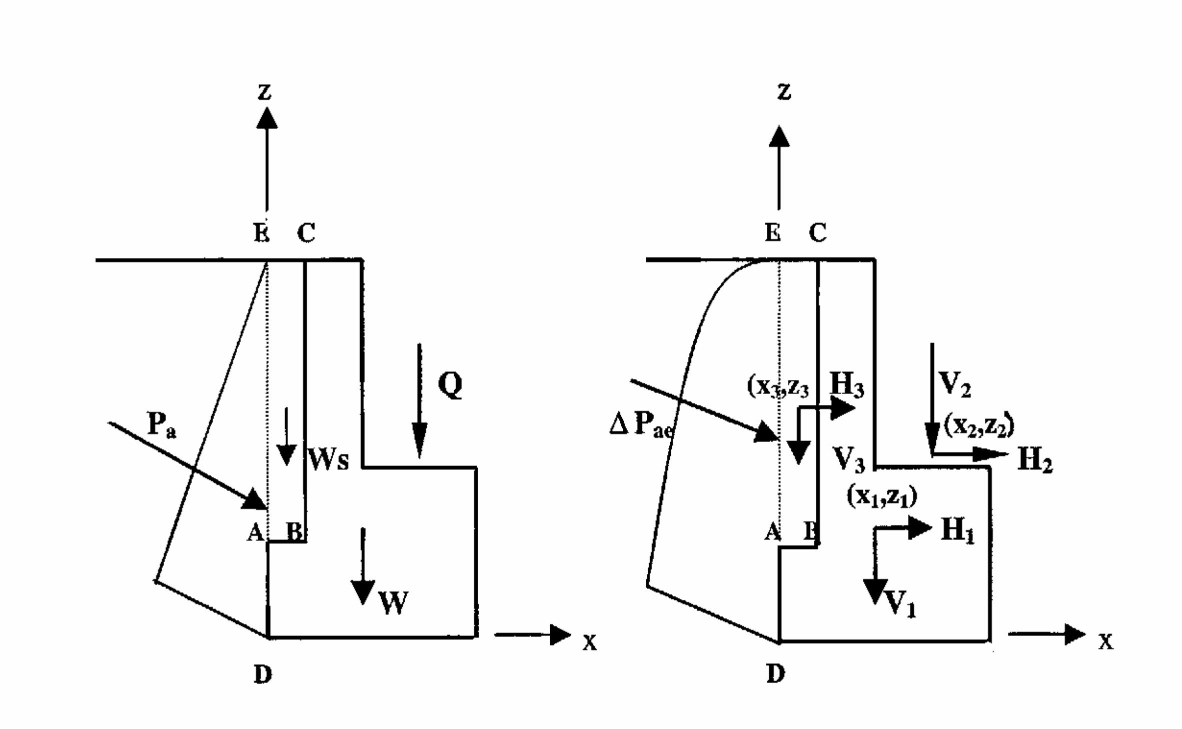


Figure 6.2 Translation and Rotation Movement of Bridge Abutment Forces Acting on the Non-Integral Bridge Abutment

a) Initial Condition
b) Sliding
c) Sliding and Rotation



a) Static Forces b) Dynamic Force Increments

Figure 6.3 Forces Acting on the Non-Integral Bridge Abutment

The horizontal force (P_x) and moment (M_ϕ) about the heel (D) due to seismic force are:

$$P_x = H_1 + H_2 + H_3 + \Delta P_{ae} \cos(\delta) \tag{6.5a}$$

$$M_\phi = V_1.x_1 + V_{2.}\, x_2 + V_3\, x_3 + \Delta P_{ae} \cos(\delta).\ 1/2H + H_{1.}\, z_1 + H_{2.}\, z_2 + H_{3.}\, z_3 \tag{6.5b}$$

The seismic displacement analysis procedure is presented as follow;

1. The bridge abutment supported on piles is shown (Figure 6.1).
2. Two degree of freedom of motion is used to obtain displacement of bridge abutment.
3. Point of rotation was assumed at the heel of bridge abutment (Figure 6.2, Wu, 1999).
4. Seismic response of bridge abutment was calculated based on the time history of acceleration acting on the base of bridge abutment.
5. The pile and soil interaction provided stiffness and damping, and the abutment provides the mass.
6. Stiffness and damping constants of soil-pile interaction were calculated using recommendation of Novak's (1974) and Novak and El-Sharnouby (1983) (Appendix H).
7. The seismic forces are presented based in Figure 6.3.
8. Non-linear soil properties were used to calculate strain-dependent stiffness and damping factors (Appendix B).
9. Displacements were calculated by solving the seismic force equilibrium for the active state condition. This means that the permanent displacement occurred if the acceleration acts towards the fill and the wall moves away from the fill.

10. Total displacements at the top of bridge abutment were calculated by adding the sliding and overturning displacement.

The solution technique to obtain the abutment displacement is presented in Appendix F.

7.0 REGIONAL GEOLOGY AND GEOTECHNICAL DATA

This section describes the regional geology of the study sites and summarizes the pertinent geotechnical data.

7.1 Regional Geology

The regional geology of the US 60 roadway between Poplar Bluff and Sikeston, Missouri is characterized by alluvial sands and gravels deposited by the ancestral Mississippi River, underlain by a dipping sedimentary sequence of limestones, dolomites, shales, and sandstones.

The study section of US 60 is located along the western margin of the Mississippi Embayment, as shown in Figure 7.1. This portion of the Embayment is bounded on the west by the Ozark Escarpment in Poplar Bluff, which consists of Paleozoic Dolomite and Sandstone overlain by thin residual soils. The Embayment extends in other directions beyond the study section. Shallow materials below the study section may be characterized as Wisconsin Braided Stream Terraces, from previous channels of the post-glacial Mississippi River. The terraces are broken into two groups, separated at Dexter by Crowley's Ridge (composed of Wilcox Group sand). The first groups of terrace deposits are older early Wisconsin terraces, located to the west of Crowley's Ridge, spanning from Poplar Bluff to Dexter. The second groups of terraces are younger late Wisconsin terraces, located to the east and extending from Dexter to Sikeston. The relationship of these terraces to Crowley's Ridge may be seen in the cross-section in Figure 7.2.

The early Wisconsin terraces to the west range in thickness up to 200 feet, based on maps by Saucier (1994), and consist of sand with some gravel (Grohskopf, 1955). Near Poplar Bluff, Ozark Escarpment Alluvial Fan deposits overlie the terraces. The terraces also contain the incised channels of the modern Black and St. Francis Rivers (near Poplar Bluff and Fisk, respectively) and an abandoned channel of the St. Francis River (approximately six miles east of Poplar Bluff). A more significant abandoned channel of an unnamed creek just west of Dexter has deposited undifferentiated Holocene Alluvium along approximately three miles of US 60. The slightly elevated Dudley Ridge represents a separate valley train deposit within the early Wisconsin sequence.

The late Wisconsin terraces to the east range in thickness up to 150 feet, an estimate also based Saucier (1994) maps, and are similar in composition to the early Wisconsin terraces.

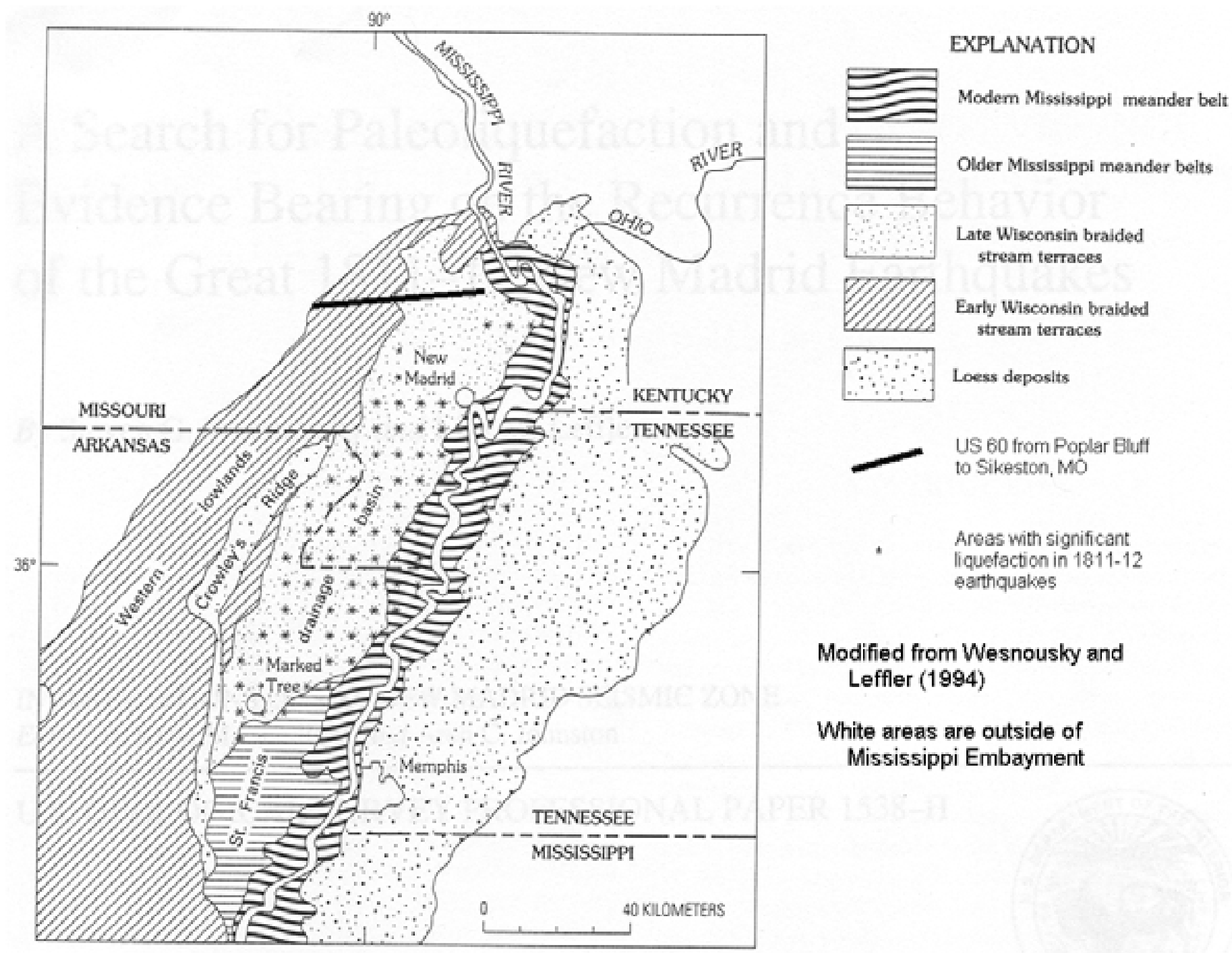


Figure 7.1 Extent of Mississippi Embayment

Sikeston Ridge is composed of early Wisconsin materials and is the remnant of a stream terrace deposited before the Mississippi occupied the Dexter-to-Sikeston floodplain. East of the Sikes-ton Ridge, late Wisconsin materials are overlain by sand and aeolian deposits (Saucier, 1994).

Crowley's Ridge is 40-mile long linear feature, trending northeast, which near Dexter is a cuesta of resistant Wilcox and Midway Group materials. The ridge is capped by residual soils and loess.

Bedrock beneath the US 60 alignment dips to the east, and the bedrock sequence from Poplar Bluff to Sikeston progresses from Paleozoic (Powell, Cotter / Jefferson City, Roubidoux and Gasconade Formations), to Cretaceous (McNairy and Owl Creek Formations), to Paleocene (Midway Group) and Eocene (Wilcox Group).

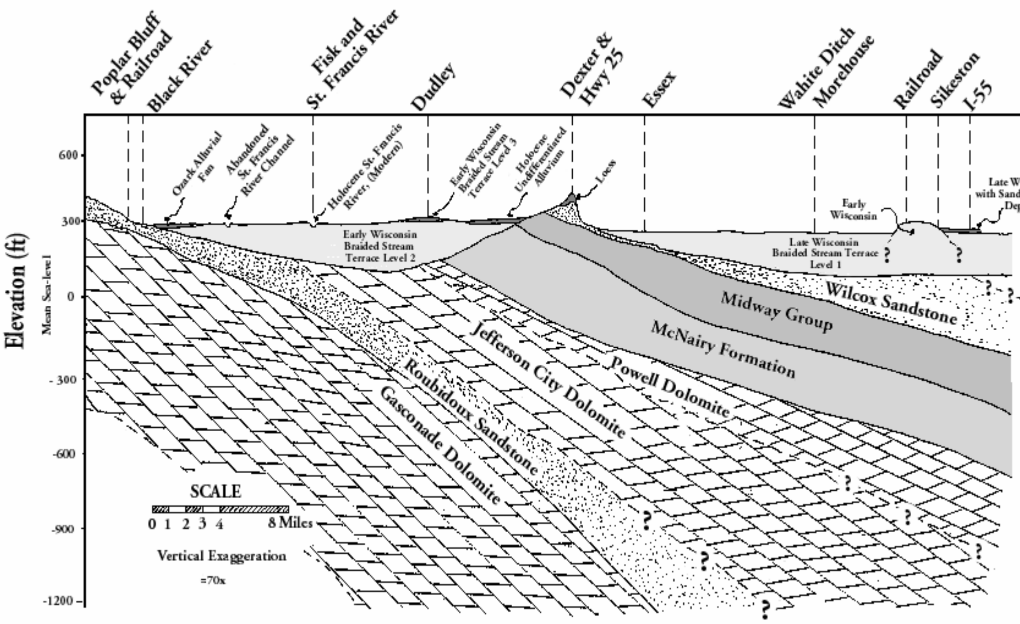


Figure 7.2 Cross-Section of Regional Geology

7.2 Summary of Field and Laboratory Data

The first task of the field investigation involved discussions with local Missouri Department of Transportation personnel regarding general road conditions. Mr. Lonnie Blasingame, the Missouri Department of Transportation Regional Superintendent for this portion of US 60, provided information regarding roadwork related to earthquake, slope stability, or flooding issues. In his experience in the area, which spans the last 14 years, no major roadwork has been required for slope stability, flooding, or seismicity-related damage. In general, work along the roadway has been to fill small potholes, and to cut expansion joints because the summer heat permanently expands the concrete, creating a potential for buckling of the pavement.

The majority of the field investigation involved drilling of exploratory boreholes, advancing CPT soundings, and digging shallow test pits. Boring and test pit logs are attached in Appendix A. The general stratigraphy shown by these logs was summarized in Section 7.2 above.

CPT data summaries are also attached in Appendix A. The stratigraphy indicated by the CPT soundings was used to confirm and enhance the cross-sections developed for slope stability analysis and to provide soil density and gradation information for liquefaction analysis.