FIFTH SHORT COURSE
SOIL DYNAMICS IN ENGINEERING PRACTICE
WHEELING, IL APRIL 29-30, 2013

STABILITY OF ABUTMENTS OF TWO BRIDGES IN MISSOURI

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(updated March 2013)

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INTRODUCTION

The damaging earthquakes of 1964 in Niigata, Japan and Alaska and the growth of nuclear industry in 1960s and 1970s and the associated stringent safety requirements for the nuclear power plants lead to rapid growth of the field of Geotechnical Earthquake Engineering.

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Other factors that contribute to the growth of this field are the industrial advancements.

1. Design of foundations for power generation equipment and other machinery;
2. Design and construction of offshore structures and
3. Defense requirements.

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Infrastructure Systems

1. Transport facilities such as bridge for highways, railways, subways, airports, tunnels;
2. Residential facilities such as single storey to multi-storey buildings;
3. Industrial units, dams, nuclear power plants and offshore platforms.

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Infrastructure Systems Failure

It may be mentioned that the transportation facilities are essential for movements of goods and people, for economic progress and also have a post-earthquake importance in providing relief and quick movement of medical facilities. Their failure may result in losses several times the cost of their repair and reconstruction.

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The Goals of the Site Assessments at These Locations:

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

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

3. Estmiate shaking effects on the various types of existing bridge structures at each site.

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The Goals of the Site Assessments at These Locations:

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

5. Evaluate the modified site assessment techniques identified in the US60 study and establish a basis for using these modified techniques at other sites along designated emergency access routes.

6.Finally, a qualitative assessment of slope stability along the MP100/I-44/US50 corridor from Manchester to Gerald will be completed, as well as an assessment of evidence of previous earthquake activity (in the form of sand blows, prehistoric slope movement etc).

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Geotechnical Engineering is Useful in:

1. Design of new infrastructures;
2. Ascertaining safety and stability of existing infrastructures which might have been designed and built in the past , when the seismic analysis and design considerations were not as stringent and the behavior of soils under dynamic loads and method of analysis were not as advanced;

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Geotechnical Engineering is Useful in (Cont.):

a) For example, non-linear behavior of soil and strain dependence of shear modulus and damping have been studied extensively only since late sixties;

b) Similarly, the liquefaction phenomena in soil and methods to predict liquefaction of sand have undergone significant changes in the last 40 years and

c) some aspects of liquefaction of silts and clay are still in the preliminary stage of development, although many sites with silts have liquefied in Turkey earthquake of 1999.

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Problem Statement and Objective

The Missouri Department of Transportation has designated specific routes for vehicular access of emergency personnel, equipment and supplies in the event of a major earthquake in southeast Missouri. This routes have varied geologic settings and include or cross many critical roadway features such as

1. Bridges and Box culverts
2. Retaining Walls and Abutments
3. Steep Slopes of Abutment fills and
4. Flooded area

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.

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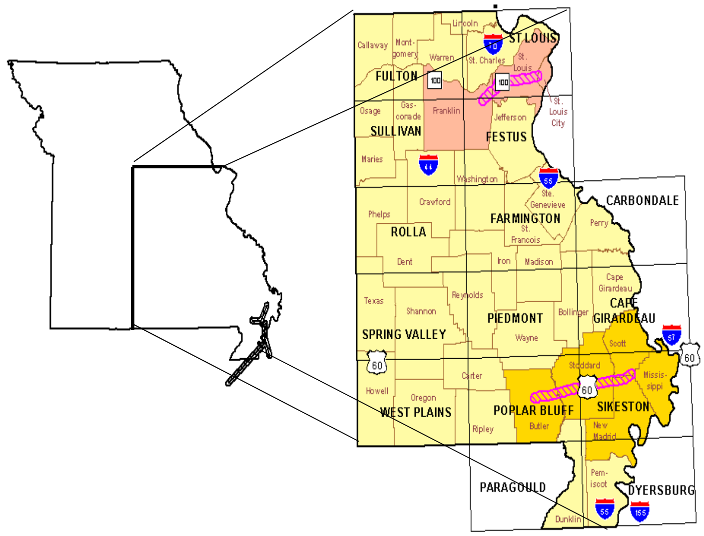
VICINITY MAP AND PROJECT LOCATION MAY 2000
Study Area Index Maps



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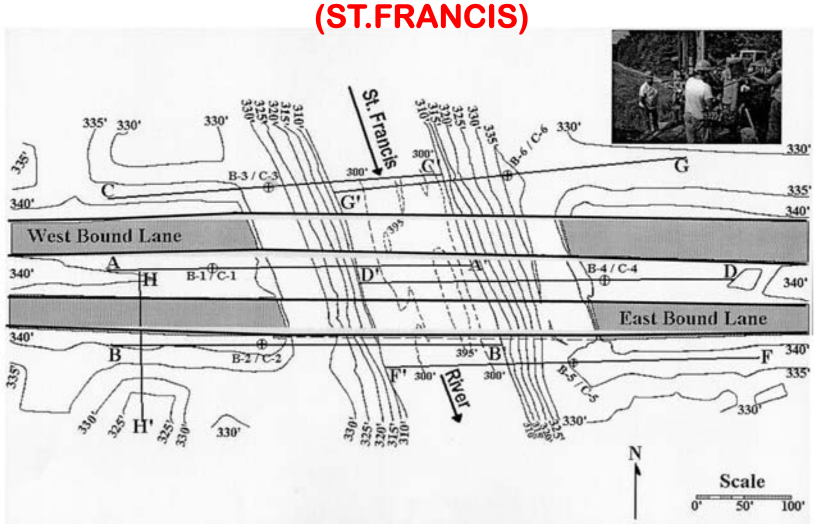
SUB SURFACE EXPLORATION SITE PLAN
(ST.FRANCIS)



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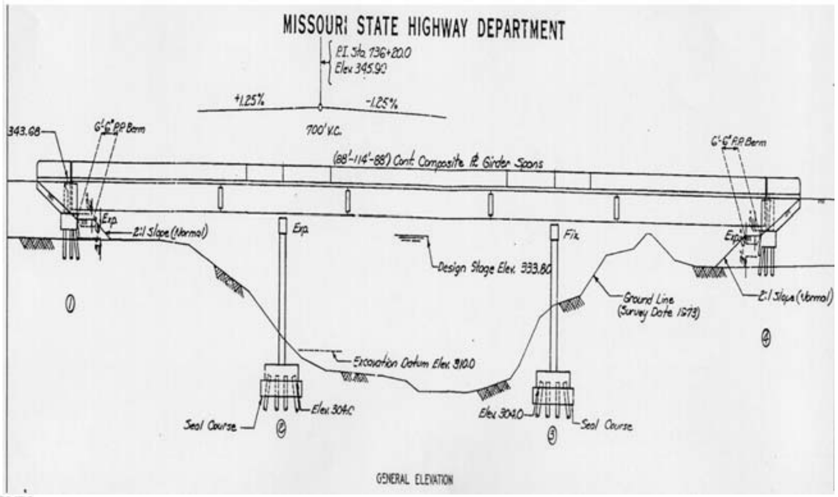
GENERAL ELEVATION OF BRIDGE OVER ST. FRANCIS RIVER



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SEISMIC HAZARD ANALYSIS

1. Selection of Credible Synthetic Ground Motion
2. Shake Analysis
3. Liquefaction Analysis
4. Abutment Analysis
5. Slope Stability Analysis

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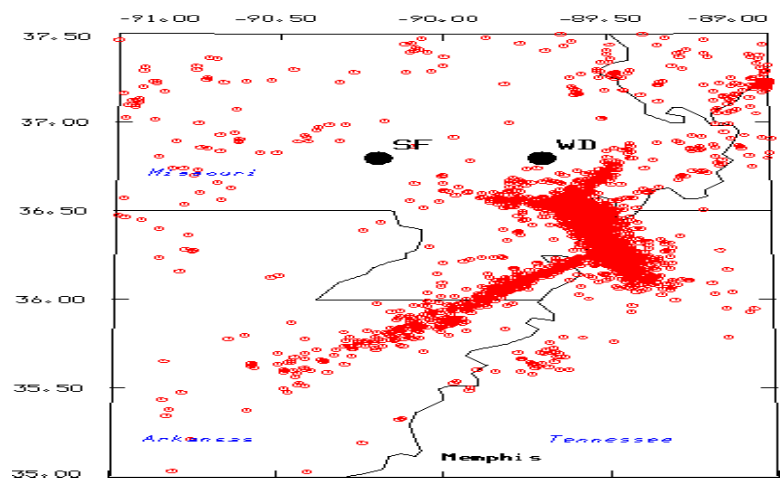

Seismicity in the 1974 - 1995 Time Period in the Vicinity of the St. Francis River Site (SF) and the Wahite Ditch Site (WD).(Herrmann, (2000))

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SITE SPECIFIC GROUND MOTION

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

Peak Ground Acceleration (USGS 1996 Seismic Hazard Maps)

Probability of Exceedance	Magnitude	Distance, R
	Mw	(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

a. Wahite Ditch Site

Probability of Exceedance	Magnitude	Distance, R
	Mw	(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. St. Francis River Site

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

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

a. St. Francis River Bridge Site

Probability of Exceedance	Magnitude	Distance, R
	Mw	(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	Distance, R
	Mw	(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

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Table 8.1: Detail of Synthetic Ground Motion at the Rock Base of Wahite Ditch Site with Corresponding Maximum Peak Horizontal Ground Acceleration
a. PE 10% In 50 Years

Name (1)	Mw (2)	R (km) (3)	Max acc. at rock-base(g) (4)	Max acc. at soil-surface(g) (5)
WD100101*	6.4	40	0.126	0.153
WD100102*	6.4	40	0.119	0.152
WD100103	6.4	40	0.136	0.127
WD100104	6.4	40	0.121	0.144
WD100105*	6.4	40	0.13	0.151
WD100201*	7.0	65	0.124	0.185
WD100202*	7.0	65	0.142	0.171
WD100203	7.0	65	0.173	0.171
WD100204	7.0	65	0.144	0.147
WD100205*	7.0	65	0.166	0.180
Mw = Magnitude R = Epicentral distance * Used in further analysis				

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Table 8.1 Cont. : b. PE 2% In 50 Years

Name (1)	Mw (2)	R (km) (3)	Max acc. at rock-base(g) (4)	Max acc. at soil-surface(g) (5)
WD020101*	7.8	16	1.549	0.437
WD020102*	7.8	16	1.769	0.478
WD020103*	7.8	16	2.129	0.512
WD020104	7.8	16	1.996	0.415
WD020105	7.8	16	1.822	0.423
WD020201	8.0	20	1.442	0.440
WD020202	8.0	20	1.589	0.440
WD020203*	8.0	20	1.855	0.525
WD020204*	8.0	20	1.720	0.406
WD020205*	8.0	20	1.559	0.447
Mw = Magnitude R = Epicentral distance * Used in further analysis				

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Table 8.2: Detail of Peak Ground Motion Used at the Wahite Ditch Site Rock Base, Ground Surface, Bridge Abutment and Pier

a. PE 10% in 50 years

File Name	Max. acc. at rock-base EL. 106.0 (g)	Max acc. at soil-surface EL 307.2 (g)	Max acc. at bridge abutment EL 301.2 (g)	Max acc. at bridge pier EL 269.9 (g)
WD100101*	0.126	0.153	0.153	0.139
WD100102*	0.119	0.152	0.151	0.127
WD100105*	0.13	0.151	0.151	0.120
WD100201*	0.124	0.185	0.185	0.169
WD100202*	0.142	0.171	0.170	0.146
WD100205*	0.166	0.18	0.180	0.157

b. PE 2% in 50 years

File Name	Max. acc. At rock-base EL. 106.0(g)	Max acc. at soil-surface EL 307.2 (g)	Max acc. at bridge abutment EL 301.2 (g)	Max acc. at bridge pier EL 269.9 (g)
WD020101*	1.549	0.437	0.440	0.430
WD020102*	1.769	0.478	0.482	0.512
WD020103*	2.129	0.512	0.514	0.522
WD020202*	1.589	0.44	0.446	0.466
WD020203*	1.855	0.525	0.527	0.538
WD020205*	1.559	0.447	0.449	0.444

Table 8.2: Detail of Peak Ground Motion Used at the St.Francis River Site Rock Base, Ground Surface, Bridge Abutment and Pier

a) PE 10% in 50 years

Name	Max. acc. at rock-base EL. 149.8. (g)	Max. acc. at soil-surface EL. 341.8. (g)	Max. acc. at bridge abutment EL341.8 (g)	Max. acc. at bridge-pier EL 301.4 (g)
SF100103*	0.106	0.146	0.160	0.126
SF100104*	0.100	0.146	0.160	0.134
SF100105*	0.107	0.151	0.155	0.154
SF100201*	0.113	0.203	0.206	0.214
SF100202*	0.136	0.196	0.200	0.204
SF100205*	0.153	0.187	0.190	0.204

b) PE 2% in 50 years

Name	Max. acc. at rock-base EL. 149.8. (g)	Max. acc. at soil-surface EL. 341.8. (g)	Max. acc. at bridge abutment EL341.8 (g)	Max. acc. at bridge-pier EL 301.4 (g)
SF020101*	1.069	0.497	0.514	0.655
SF020103*	0.845	0.428	0.437	0.560
SF020105*	1.089	0.473	0.490	0.602
SF020201*	0.604	0.447	0.457	0.571
SF020203*	0.693	0.453	0.465	0.544
SF020205*	0.596	0.391	0.400	0.452

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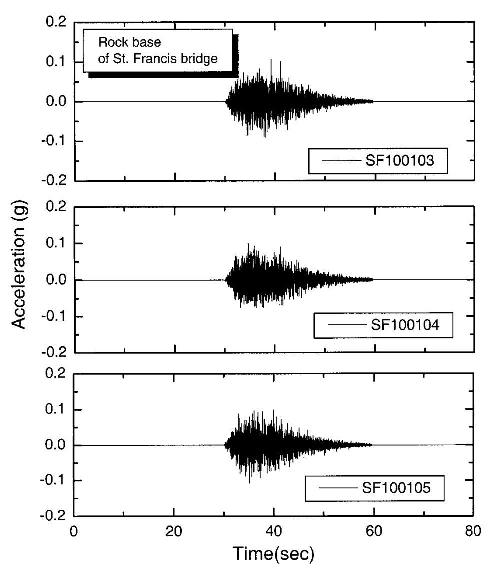

a. PE 10 % in 50 years, Magnitude = 6.2

Figure 8.4a Acceleration Time Histories for St. Francis River Bridge Site PE 10% in 50 Years, Magnitude = 6.2

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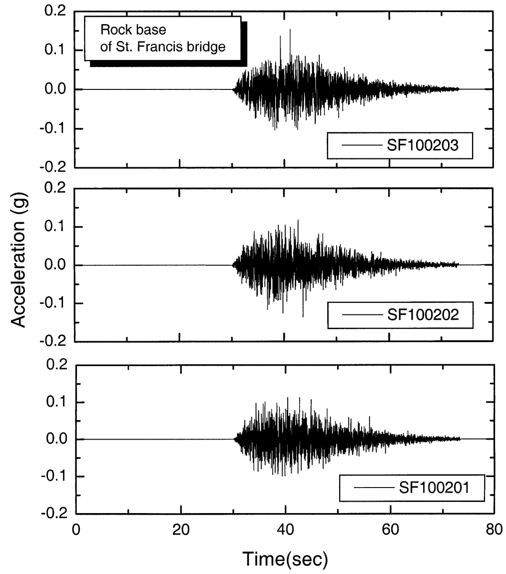

b. PE 10 % in 50 years, Magnitude = 7.2

Figure 8.4b Acceleration Time Histories for St. Francis River Bridge Site PE 10% in 50 Years, Magnitude = 7.2

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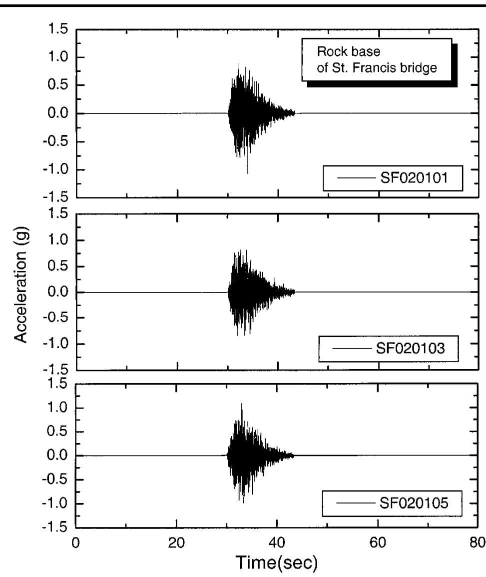

c. PE 2 % in 50 years, Magnitude = 6.4

Figure 8.4c Acceleration Time Histories for St. Francis River Bridge Site PE 2% in 50 Years, Magnitude = 6.4

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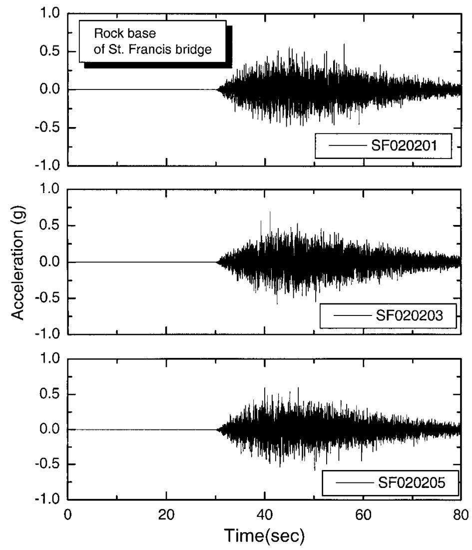
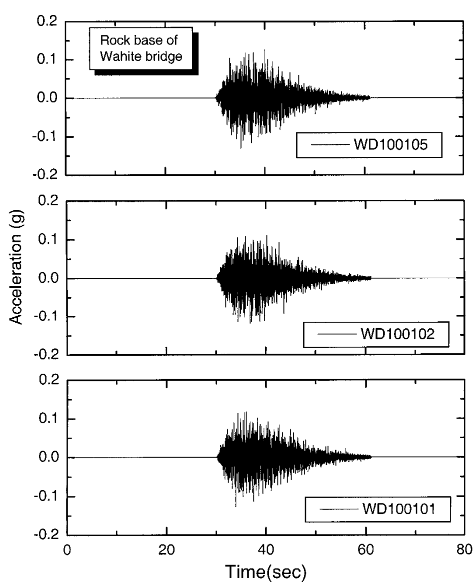


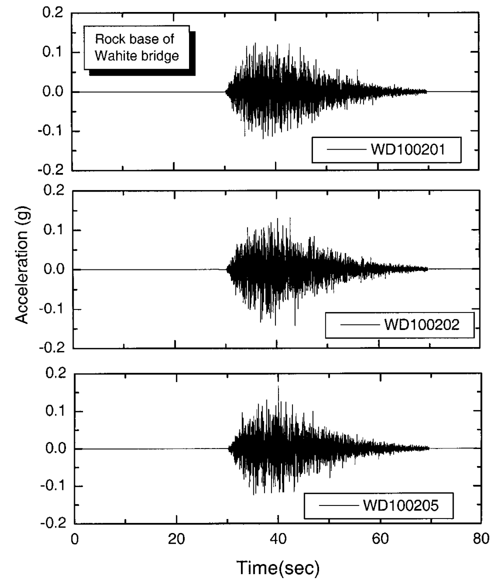
d. PE 2 % in 50 years, Magnitude = 8.0

Figure 8.4d Acceleration Time Histories for St. Francis River Bridge Site PE 2% in 50 Years, Magnitude = 8.0



a. PE 2 % in 50 years, Magnitude = 6.4

Figure 8.39a Acceleration Time Histories for the Wahite Ditch Bridge Site PE 2% in 50 Years Magnitude = 6.4



b. PE 10 % in 50 years, Magnitude = 7.0

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Figure 8.39b Acceleration Time Histories for the Wahite Ditch Bridge Site PE 10% in 50 Years Magnitude = 7.0

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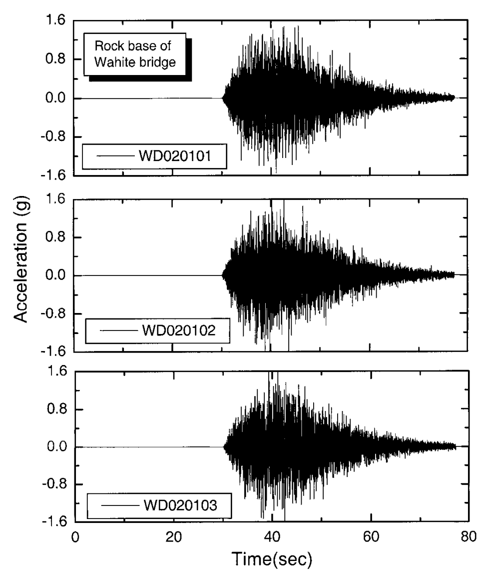

c. PE 2 % in 50 years, Magnitude = 7.8

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Figure 39c Acceleration Time Histories for the Wahite Ditch Bridge Site PE 2% in 50 Years, Magnitude = 7.8

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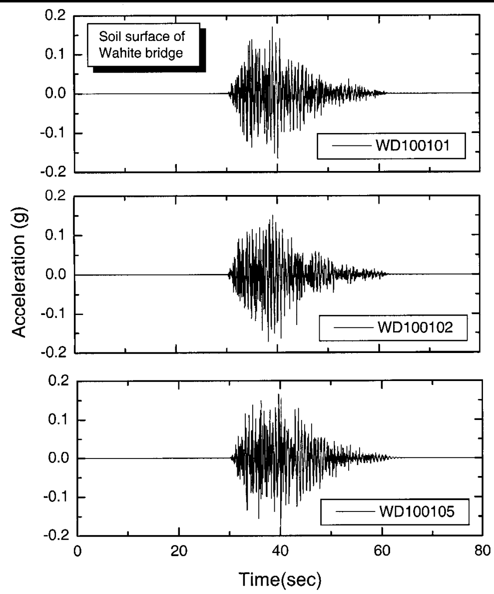

a. PE 10 % in 50 years, Magnitude = 6.4

Figure 8.44a Surface Ground Acceleration at the Wahite Ditch Bridge Site

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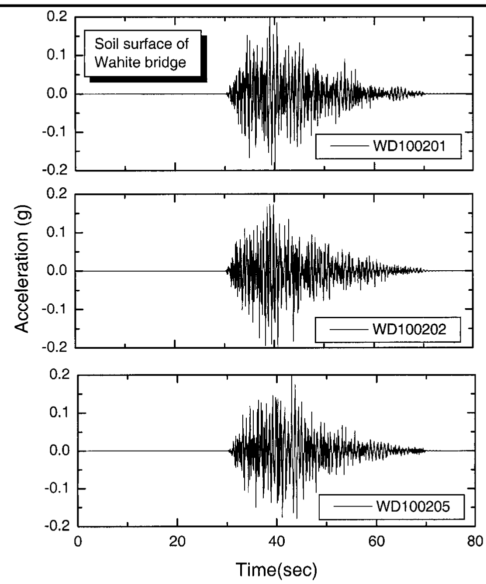

b. PE 10 % in 50 years, Magnitude = 7.0

Figure 8.44b Surface Ground Acceleration at the Wahite Ditch Bridge Site

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SEISMIC HAZARD ANALYSIS

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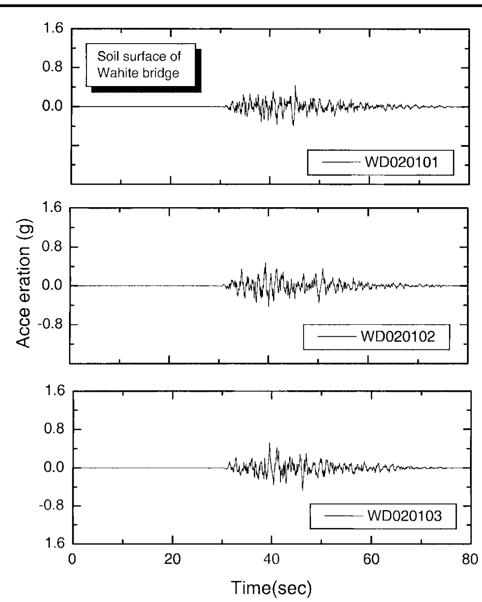

c. PE 2 % in 50 years, Magnitude = 7.8

Figure 8.44c Surface Ground Acceleration at the Wahite Ditch Bridge Site

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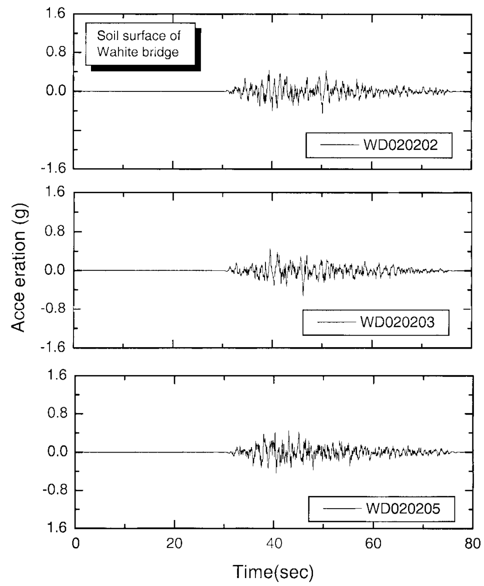

d. PE 2 % in 50 years, Magnitude = 8.0

Figure 8.44d Surface Ground Acceleration at the Wahite Ditch Bridge Site

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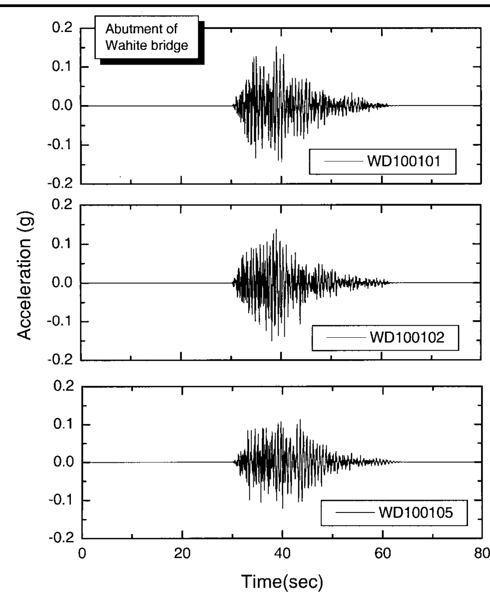

a. PE 10 % in 50 years, Magnitude = 6.4

Figure 8.45a Ground Acceleration at the Abutment Wahite Ditch Bridge

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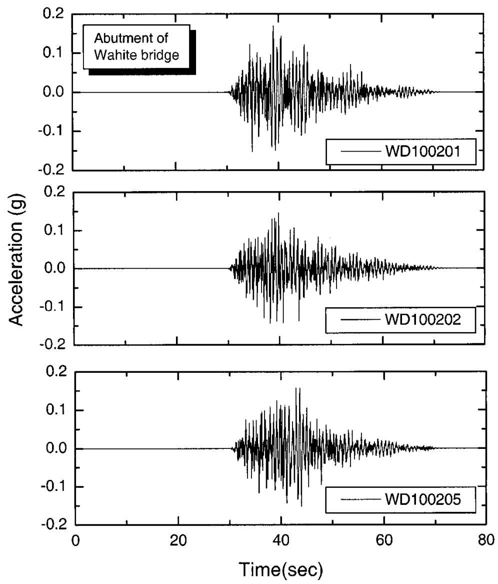

b. PE 10 % in 50 years, Magnitude = 7.0

Figure 8.45b Ground Acceleration at the Abutment Wahite Ditch Bridge

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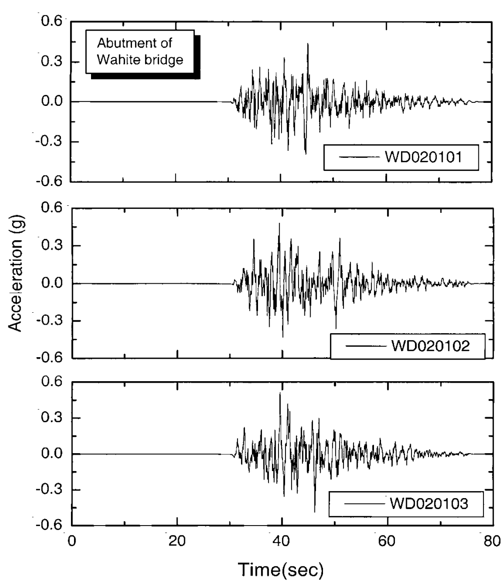

c. PE 2 % in 50 years, Magnitude = 7.8

Figure 8.45c Ground Acceleration at the Abutment Wahite Ditch Bridge

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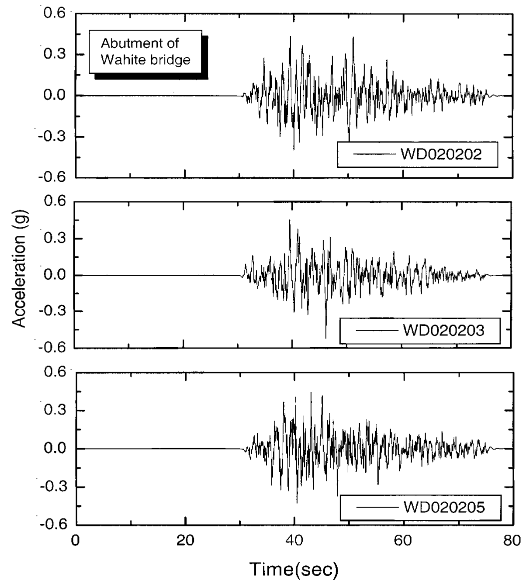

d. PE 2 % in 50 years, Magnitude = 8.0

Figure 8.45d Ground Acceleration at the Abutment Wahite Ditch Bridge

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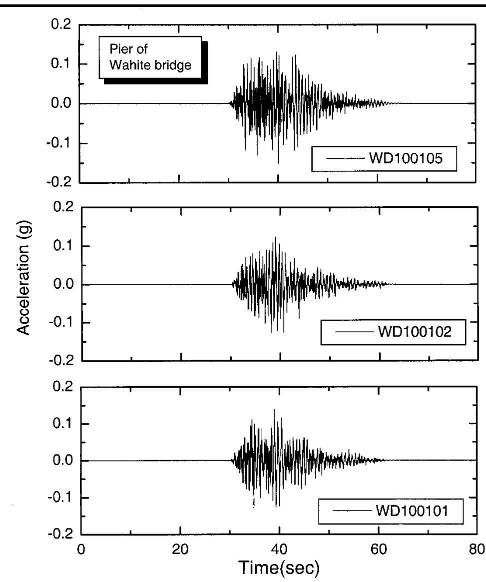

a. PE 10 % in 50 years, Magnitude = 6.4

Figure 8.45a Ground Acceleration at the Wahite Ditch Bridge Pier

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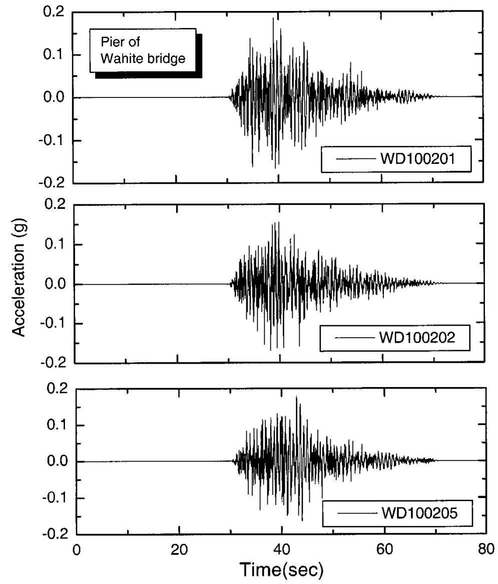

b. PE 10 % in 50 years, Magnitude = 7.0

Figure 8.46b Ground Acceleration at the Wahite Ditch Bridge Pier

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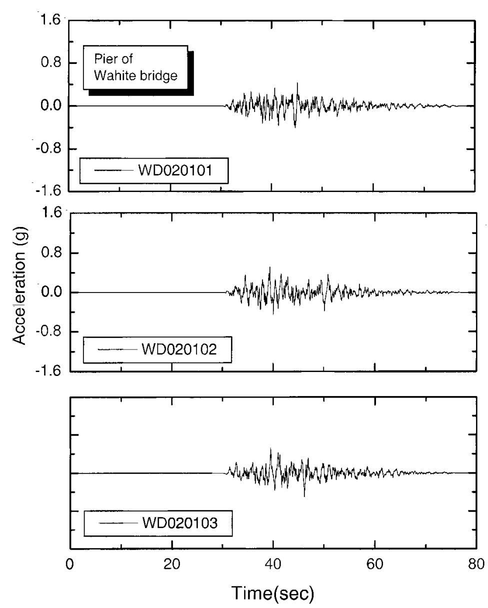

c. PE 2 % in 50 years, Magnitude = 7.8

Figure 8.46c Ground Acceleration at the Wahite Ditch Bridge Pier

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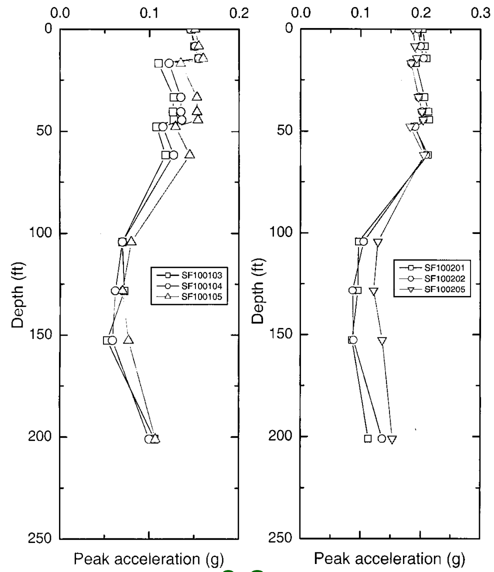
Figure 8.5 Peak Ground Acceleration vs. Depth for PE 10% in 50 Years

Francis River Bridge Site

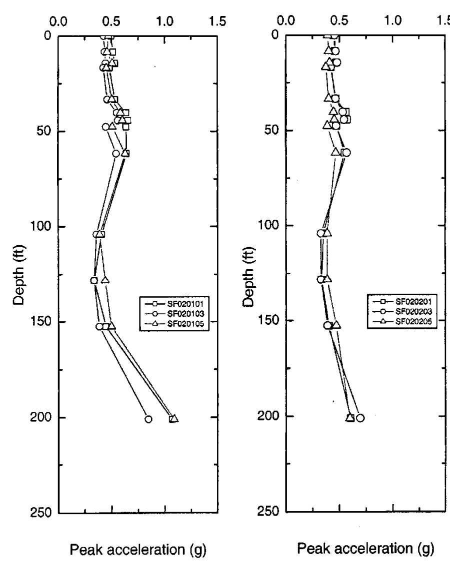


a. Magnitude = 6.2 b. Magnitude = 7.2

(Ground Motion Amplification)

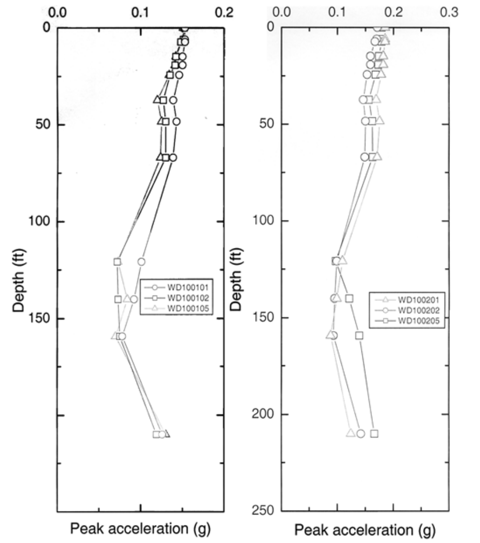
Figure 8.6 Peak Ground Acceleration vs. Depth for PE 2% in 50 Years

Francis River Bridge Site



a. Magnitude = 6.4 b. Magnitude = 8.0

(Ground Motion Attenuation)



a)PE 10 % in 50 years, M6.4 b)PE 10 % in 50 years, M7.0

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Figure 8.42 Peak Ground Acceleration vs. Depth for PE 10% in 50 Years Magnitudes 6.4 and 7.0 Wahite Ditch Bridge Site (Ground Motion Amplification)

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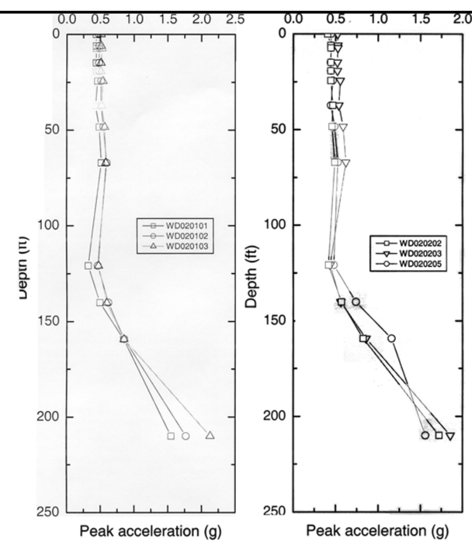

a)PE 2 % in 50 years, M7.8 b)PE 2 % in 50 years, M8.0

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Figure 8.43 Peak Ground Acceleration vs. Depth for PE 2% in 50 Years Magnitudes 7.8 and 8.0 Wahite Ditch Bridge Site (Ground Motion Attenuation)

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VERTICAL SEISMIC RESPONSE OF SOIL

Herrmann (2000) also recommended that vertical rock motion is of the same order as the horizontal rock motion. SHAKE91 is used to transmit the horizontal rock motion to the soil surface and/or any other depth. No such solution was available for transmission of vertical motion (2001). Therefore, the following procedure was adopted to transfer vertical rock motion to desired elevation.

1. Use SHAKE to transfer the P-wave.
2. Adjust peak vertical ground motion to be 2/3 of the peak horizontal ground motion.
3. Adjust the time history to reflect adjustment in (2) above.

The calculated vertical time histories of acceleration at the soil surface, the base of bridge abutment and at the bridge pier were also modified as above.

It appears that for the horizontal and vertical time histories of any one event:

4. (k_v) max and (k_h) max do not occur at the same instant of time.
5. Frequency contents of these two-motions are quite different.

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SEISMIC HAZARD ANALYSIS

1. Selection of Credible Synthetic Ground Motion
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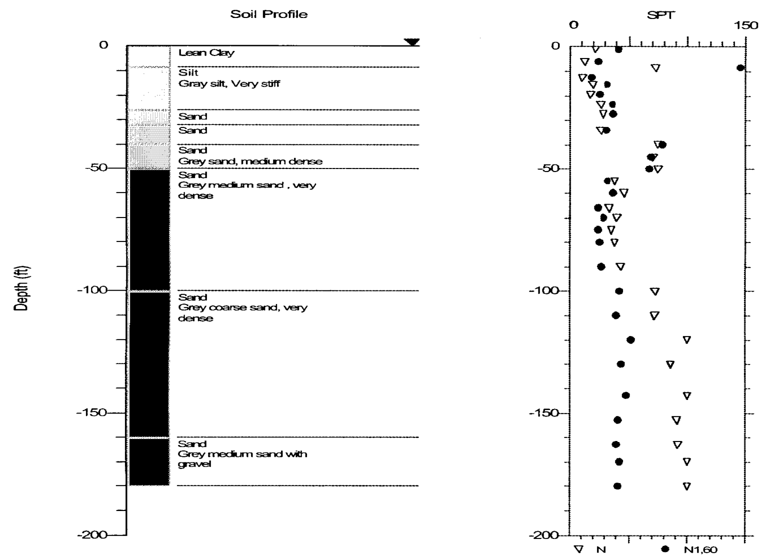
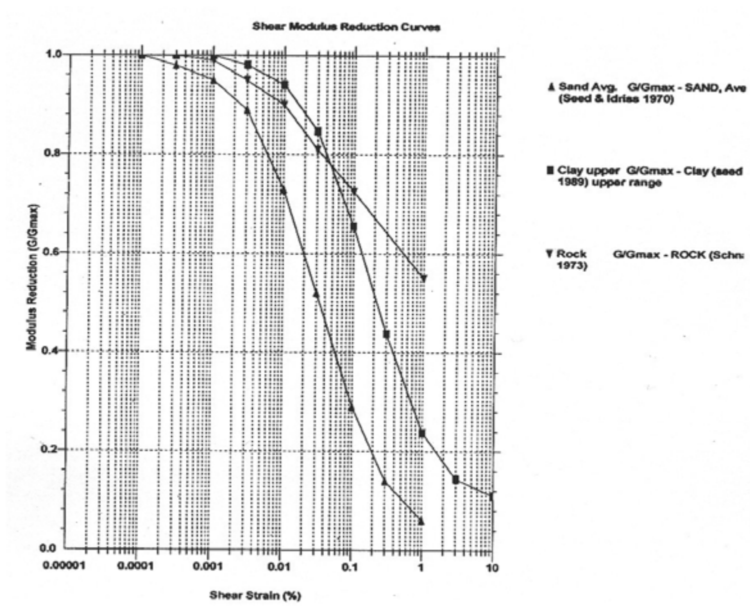
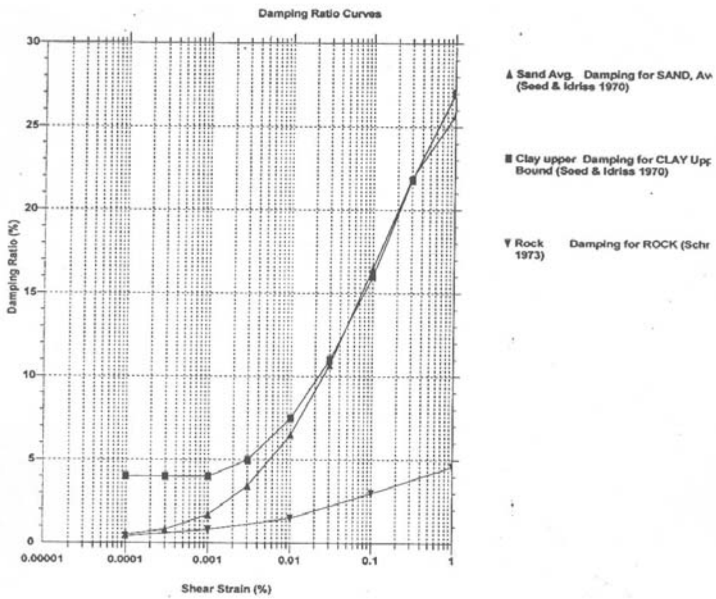


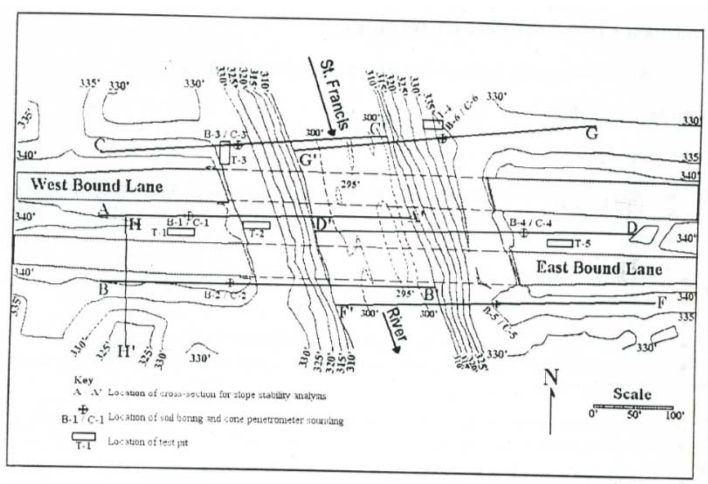
a) St. Francis Site



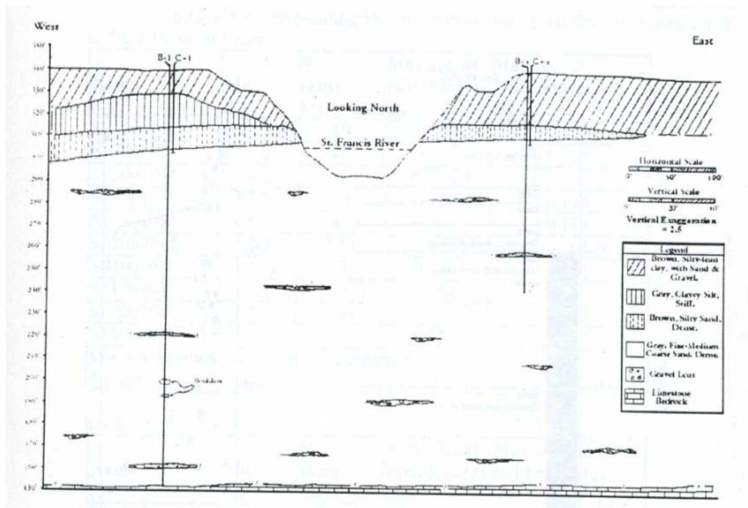
Shear Modulus Reduction Non-Linear material



b) Damping Ratio



Figure… St. Francis River Site Topography, Cross-Section and Boring Locations



Figure…Cross-Section of St. Francis River Site Geology

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) using equation 1:

$$\tau_{av} = 0.65 \bullet \left(\frac{a_{max}}{g} \right) \bullet \sigma_0 \bullet r_d \qquad (1)$$

τ_{av} may be expressed as the Cyclic Stress ratio (CSR) (equation 2)

$$CSR = \frac{\tau_{av}}{\sigma_0} = 0.65 \bullet \left(\frac{a_{max}}{g} \right) \bullet \left(\frac{\sigma_0}{\sigma_0'} \right) \bullet r_d \qquad (2)$$

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_0 = total vertical overburden stress**
- **σ_0' = 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) :

$$r_d = \frac{\left[1 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5}\right]}{\left[1 - 0.4117z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.00121z^2\right]} \quad (3)$$

2. Estimate τ_{liq} , the shear strength to cause liquefaction at the above point under the ground motion.

τ_{liq} is also expressed as cyclic resistant ratio (CRR) i.e., τ_{liq} / σ_0' at the above point. A relationship with τ_{liq} / σ_0' and corrected $(N_l)_{60}$ for earthquake magnitude 7.5 is in figure. The standard presentation test values NM are converted to $(N_l)_{60}$ by correcting for energy and other factors as below (equation 4)

$$(N_l)_{60} = NM\ CN\ CE\ CB\ CR\ CS \quad (4)$$

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

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

$$FOS = \tau_{liq} / \tau_{av} \quad (5a)$$

or

$$FOS = CRR/CSR \quad (5b)$$

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 τav 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 (amax) is used to compute τav. This may give slightly different values of τav as compared to their result using equation 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.

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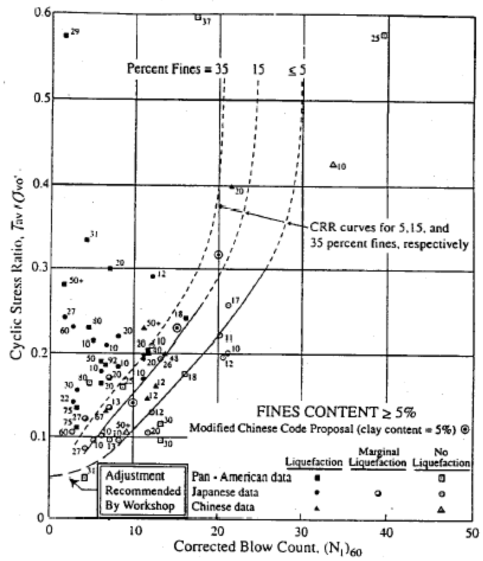

Figure 5.4 Simplified Base Curve Recommended for Calculation of CRR from SPT $(N_I)_{60}$ Data along with Empirical Liquefaction Data for M=7.5 (Seed et. al., 1971, modified by Youd and Idriss, 1997)

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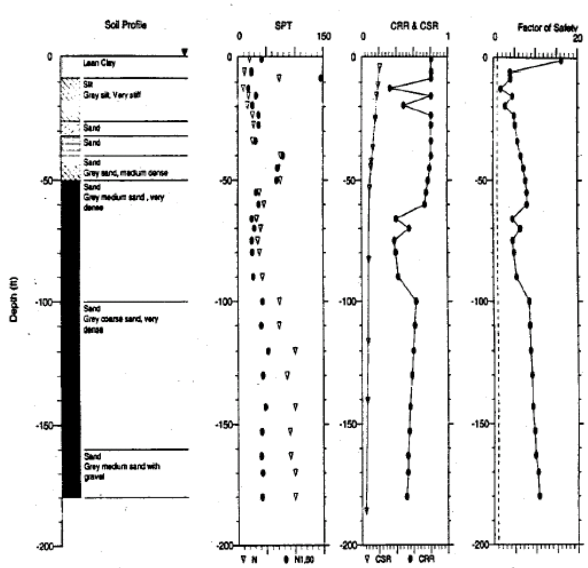

Figure…Soil Profile, CSR, CRR and Factor of Safety against liquefaction at the St.Francis River Site for PE 10% in 50 years and Magnitude = 6.2

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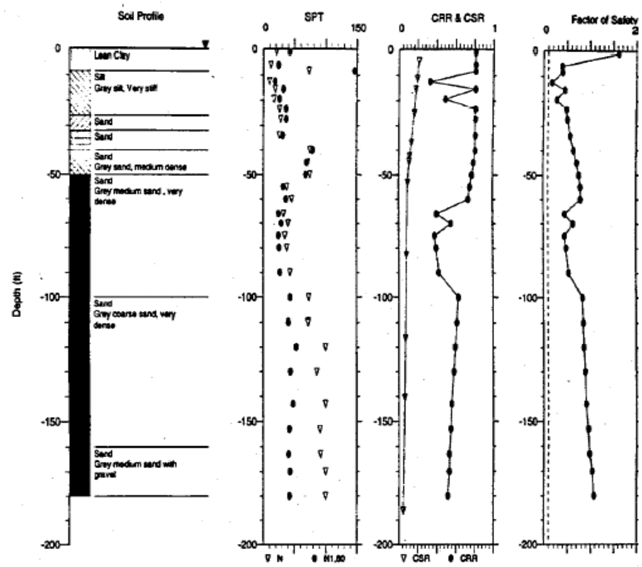

Figure 8.47 Soil Profile, CSR, CRR and Factor of Safety against liquefaction at the Wahite Ditch Bridge Site for PE 10% in 50 years and Magnitude = 6.4

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Table 8.3: The Different Zones of Soil Liquefaction for Different Factors of Safety, Francis River Bridge Site

Factor of Safety	Zones of Soil Liquefaction			
	PE10% in 50 years		PE 2% in 50 years	
	M6.2	M7.2	M6.4	M8.0
1.0	No	8.4 to 12.5	8.4 to 12.4 and 66 to 75	6 to 90
1.1	No	8.4 to 12.5	6.0 to 23.5 and 66 to 80	6 to 110
1.2	No	8.4 to 12.5	6.0 to 34.0 and 66 to 90	6 to 130
1.3	No	8.4 to 12. 5	6.0 to 40.0 and 66 to 90	6 to 153
1.4	No	8.4 to 12. 5 and75 to 80	6.0 to 50.0 and 66 to 90	6 to 180

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Figure…The Different Zones of Soil Liquefaction for Different Factors of Safety, Wahite Ditch Site

Factor of Safety	Depth of Soil Liquefy (ft)			
	PE 10% in 50 years		PE 2% in 50 years	
	M6.4	M7.0	M7.8	M8.0
1.0	No	120 to 130	20 to 201	20 to 201
1.1	No	120 to 130	20 to 201	20 to 201
1.2	No	120 to 130	20 to 201	20 to 201
1.3	No	120 to 130	20 to 201	20 to 201
1.4	No	120 to 130	20 to 201	20 to 201

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SEISMIC HAZARD ANALYSIS

1. Selection of Credible Synthetic Ground Motion
2. Shake Analysis
3. Liquefaction Analysis
4. **Abutment Analysis**
5. Slope Stability Analysis

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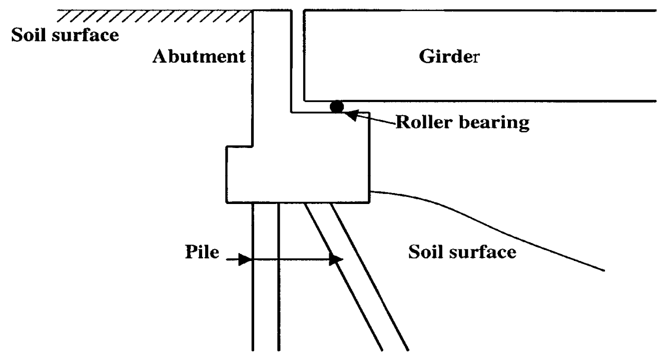
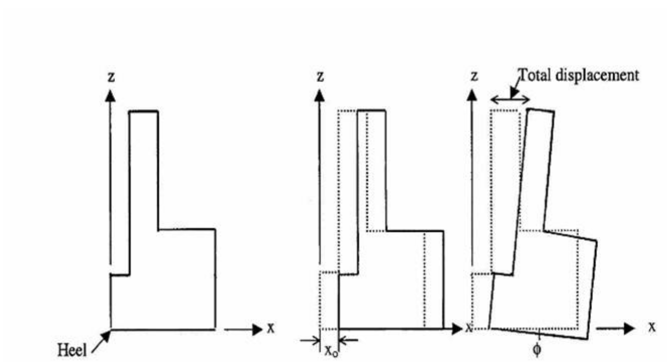
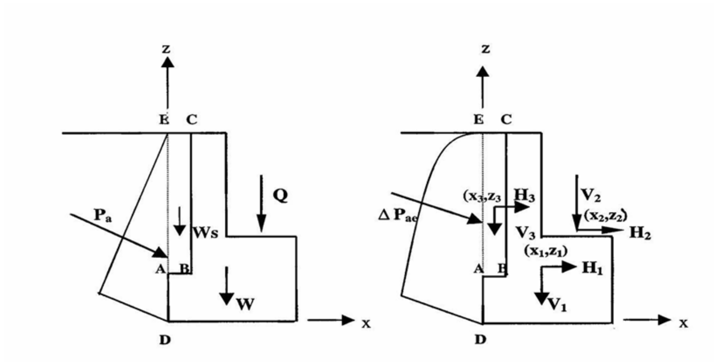


Typical Highway Bridge Abutment Supported on Piles



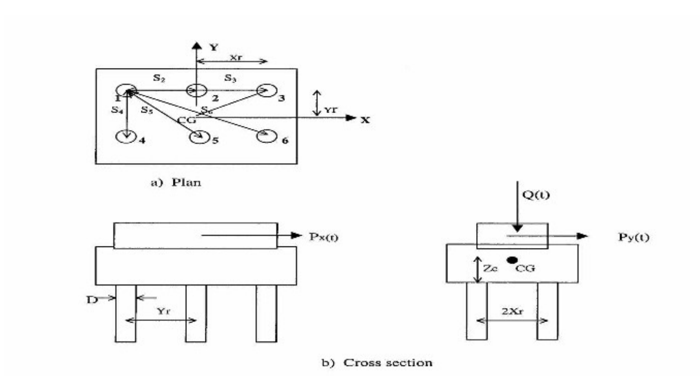
a. Initial Condition   b. Sliding   c. Sliding and Rotation

TRANSLATION AND ROTATION MOVEMENT OF ABUTMENT



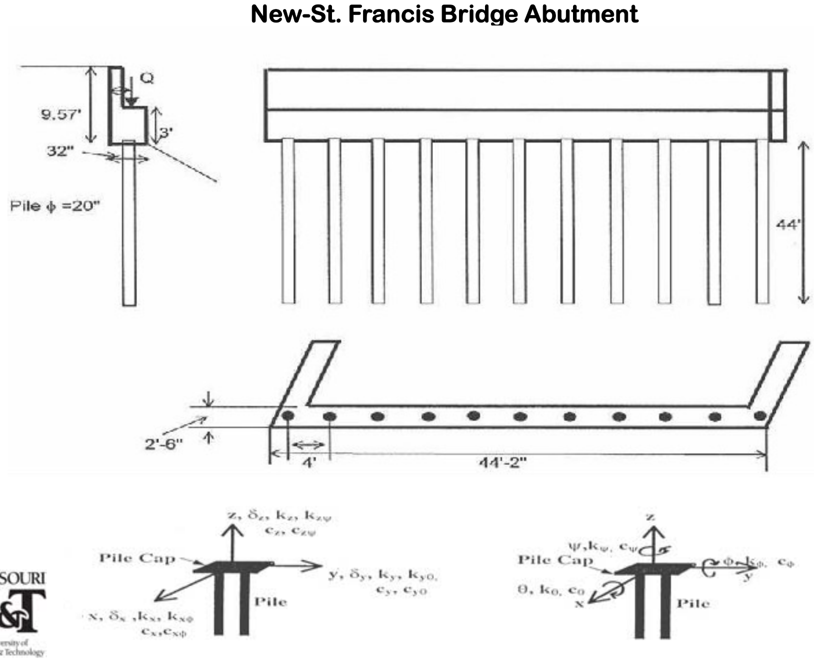
a) Static forces **b) Dynamic force increments**

FORCES ACTING ON THE BRIDGE ABUTMENT



Plan and Cross Section of Pile Group

New-St. Francis Bridge Abutment



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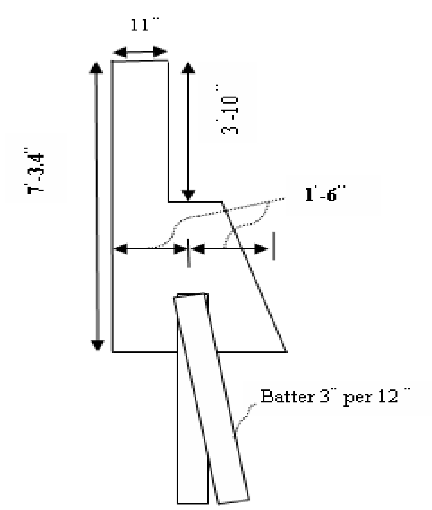

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Schematic Section of abutment of Old Wahite Bridge.

EIGHT SPRING CONSTANTS

k_x, k_y, k_z TRANSLATION

k_θ, k_ϕ, k_ψ ROTATION

$k_{x\phi}, k_{y\theta}$ CROSS-COUPLING

EIGHT DAMPING CONSTANTS

C_x, C_y, C_z TRANSLATION

C_θ, C_ϕ, C_ψ ROTATION

$C_{x\theta}, C_{y\theta}$ CROSS-COUPLING

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Strain-displacement Relationship

- The shear strain and displacement relationship is not well defined in practical problems occuring in the field. However, the relationship has been recommended by Prakash and Puri (1981) as:

$$\gamma = \frac{\text{Amplitude of foundation vibration}}{\text{Average width of foundation}}$$

For vertical and horizontal vibration

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Strain-displacement Relationship (contd.)

- Kagawa and Kraft (1980) used following relationship for horizontal displacement in front of a pile:

$$\gamma_x = \frac{(1+\upsilon)X}{2.5D}$$

Where, n = poisson's ratio
X = horizontal displacement in x-direction
D = diameter of pile

- Rafnsson (1992) recommended that, the shear strain due to rocking can be reasonably determined as:

$$r_\phi = \phi / 3$$

Where, F = rotation of foundation about x or y axis

- Shear strain-displacement relationship for coupled sliding and rocking can be determined as:

$$\gamma_x = \frac{(1+v)X}{2.5D} + \frac{\phi}{3}$$

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Solution Technique for Displacement Dependent K's and C's

1.OBTAIN Unit weight, shear wave velocity, poisson's ratio, initial shear modulus; shear modulus degradation and damping curve as function of soil shear strain.

2.OBTAIN Pile length, pile diameter, elastic modulus of pile, shear wave velocity.

3.SELECT Relationship for half space stiffness and damping parameters as function of soil parameters, pile dimensions, and piles arrangement in the group.

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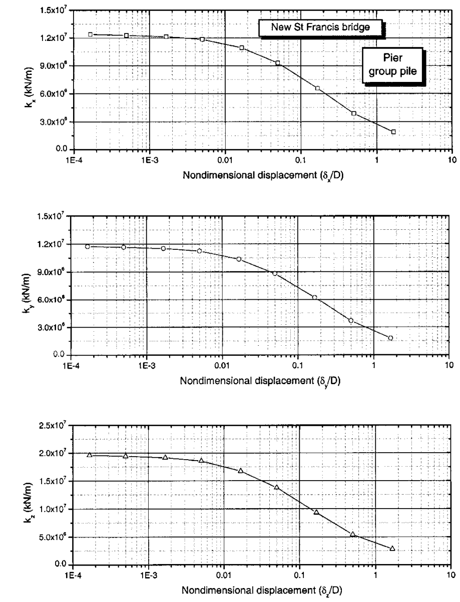

Spring stiffness for different modes of vibration

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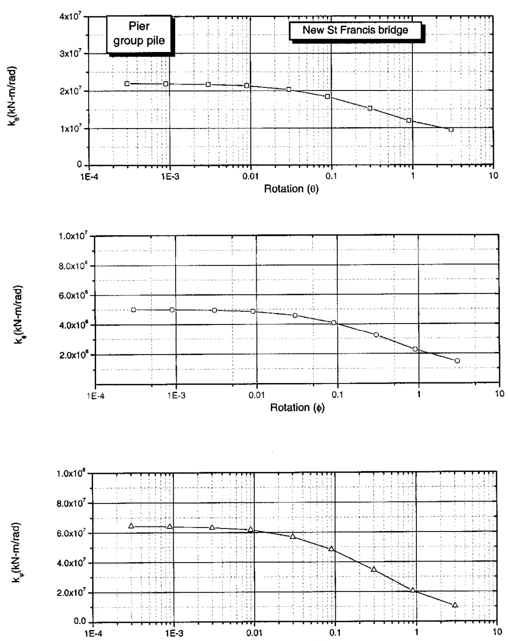

Spring stiffness for different modes of vibration

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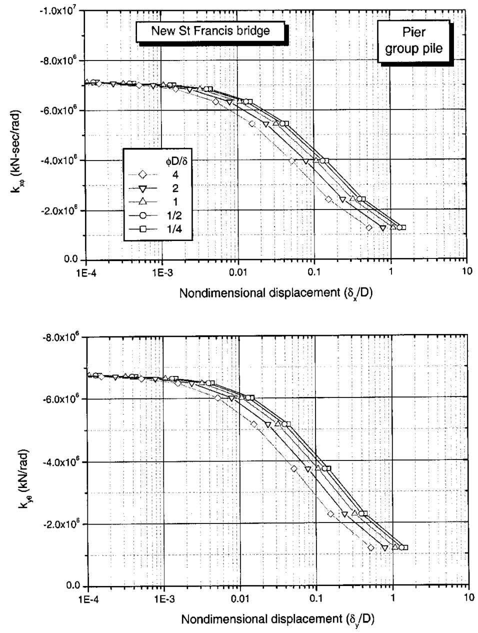
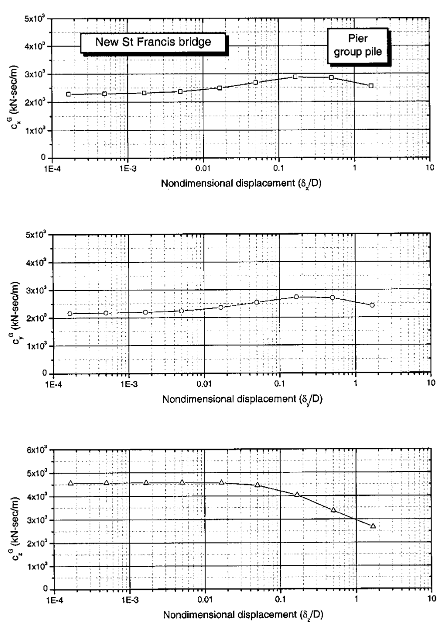
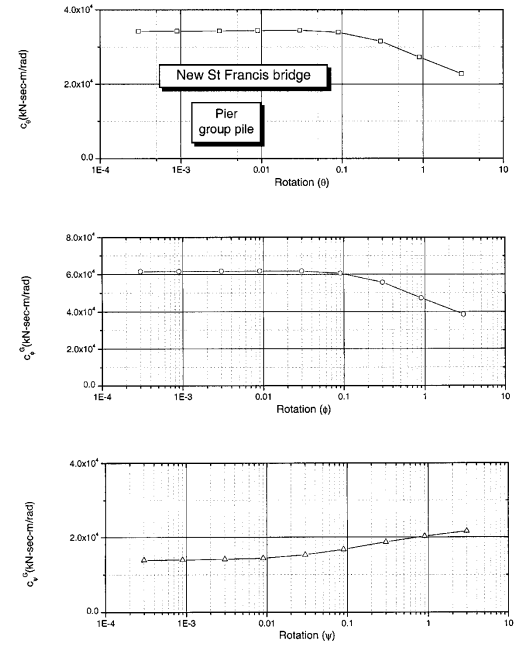
Spring stiffness for different modes of vibration





Damping for different modes of vibration



Damping for different modes of vibration

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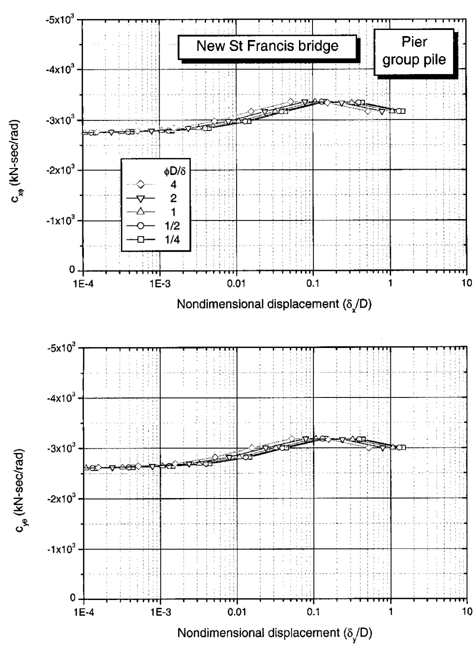

Damping for different modes of vibration

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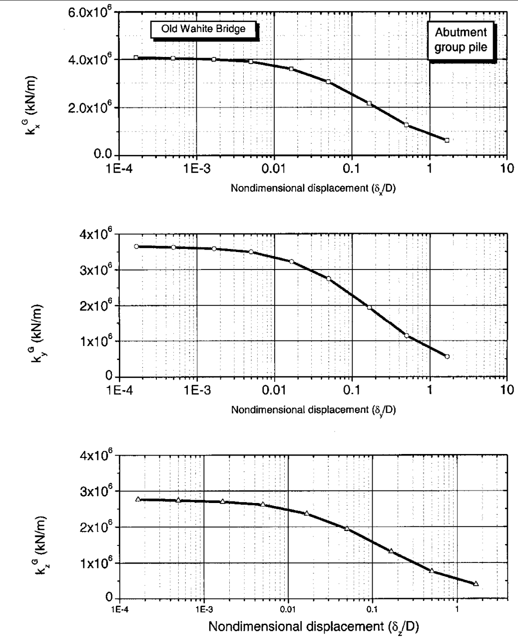
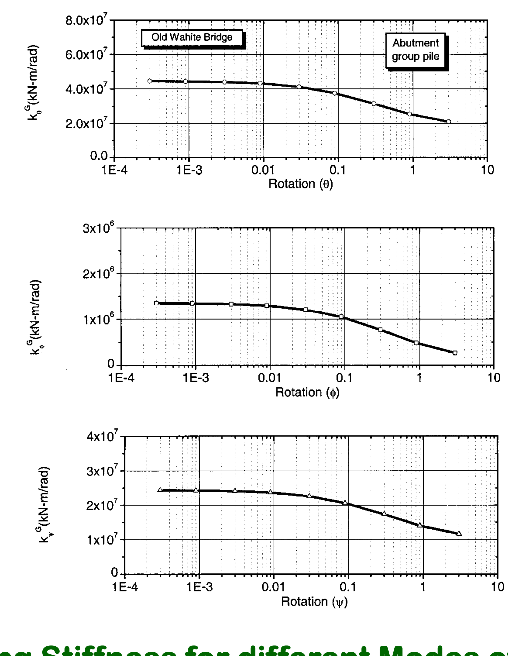
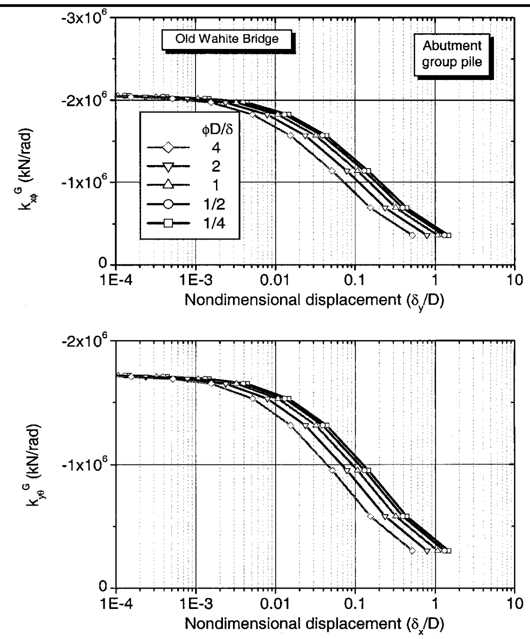


Spring Stiffness for different Modes of Vibration 79



Spring Stiffness for different Modes of Vibration 80



Spring Stiffness for different Modes of Vibration

Iterative Solution

1. Assume G_1 of soil for any instant of time (if t=0, assume $G_1 = G_{max}$)
2. Obtain all k's and c's
3. Solve equation of motion for displacement at that instant of time
4. Estimate shear strain in the soil. Appropriate displacement (X, Y or Z) and shear strain (g) relationships are used
5. Estimate G_2 for strain calculated in (4) above
6. If G_1 and G_2 are within acceptable range, the solution is OK and go to step 7, otherwise assume a new value of G_1' in (1) above as $(G_1+G_2)/2$ and repeat step 2 and 6
7. Repeat step 1-6 at other time with G_1 in (6) above to complete the time domain solution

Non-Linear Iterative Solution Technique

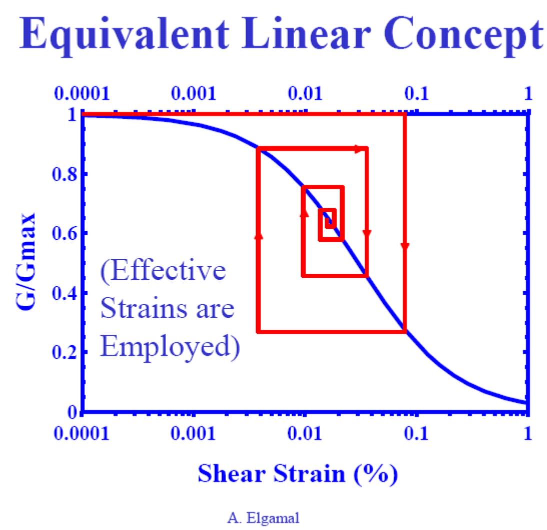
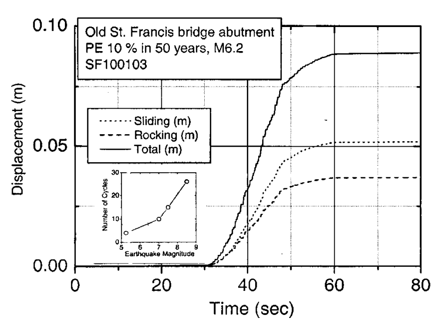
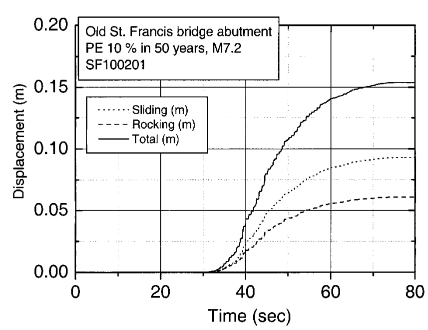


Figure 8.36 Time Histories of Sliding, Rocking and Total Permanent Displacement of the Old St. Francis River Bridge Abutment PE 10% in 50 Years, Magnitudes 6.2 and 7.2.

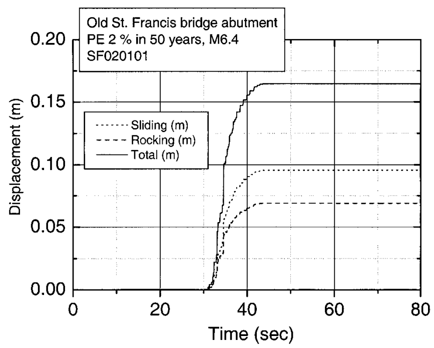


a.Magnitude 6.2

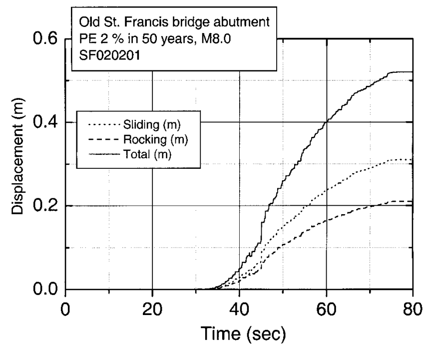


b. Magnitude 7.2

Figure 8.37 Time Histories of Sliding, Rocking and Total Permanent Displacement of the Old St. Francis River Bridge Abutment PE 2% in 50 Years, Magnitudes 6.4 and 8.



a.Magnitude 6.4



b. Magnitude 8.0

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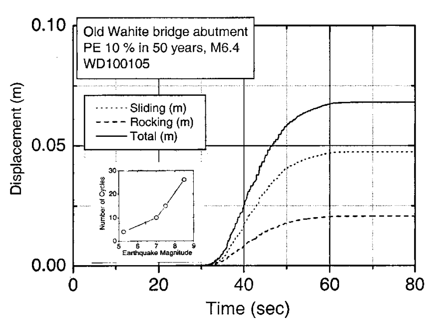
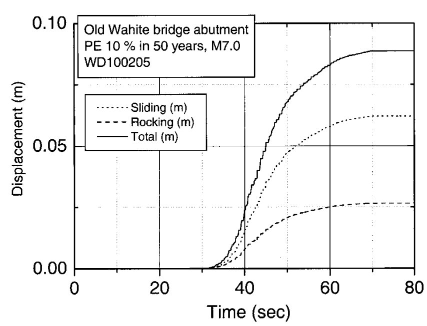
Figure 8.63 Time Histories of Sliding, Rocking and Total Permanent Displacement of the Old Wahite Ditch Bridge Abutment PE 10% in 50 Years, Magnitudes 6.4 and 7.0.



a.Magnitude 6.4



b. Magnitude 7.0

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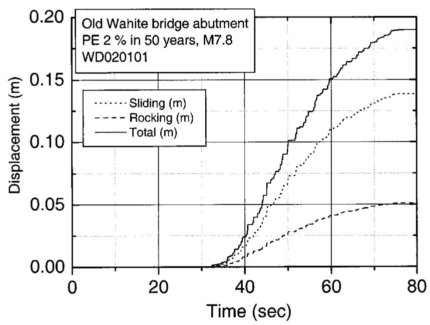
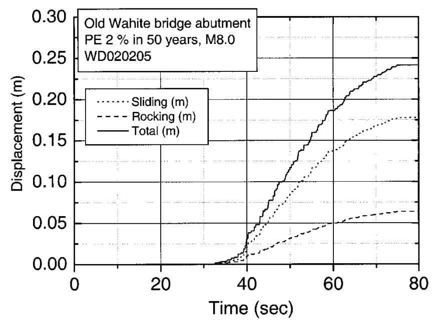
Figure 8.64 Time Histories of Sliding, Rocking and Total Permanent Displacement of the Old Wahite Ditch Bridge Abutment PE 2% in 50 Years, Magnitudes 7.8 and 8.0.



a.Magnitude 7.8



b. Magnitude 8.0

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Figure…Displacement at the Top of the Old St.Francis Bridge Abutment

Displacement at top of abutment	PE 10% in 50 years		PE 2% in 50 years	
	M6.2	M7.2	M6.4	M8.0
Sliding (m)	0.052	0.093	0.096	0.31
Rocking (m)	0.037	0.061	0.069	0.21
Total (m)	0.089	0.154	0.165	0.52
Significant Cycles	8	11	9	20
Displacement in 1-cycle	0.011	0.014	0.018	0.026

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Figure …Displacement at the Top of the Old Wahite Ditch Bridge Abutment

Displacement at top of abutment	PE 10% in 50 years		PE 2% in 50 years	
	M6.4	M7.0	M7.8	M8.0
Sliding (m)	0.037	0.028	0.139	0.178
Rocking (m)	0.018	0.053	0.0513	0.064
Total (m)	0.056	0.080	0.190	0.242
Significant Cycles	9	10	18	20
Displacement in 1-cycle	0.007	0.008	0.011	0.012

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SEISMIC HAZARD ANALYSIS

1. Selection of Credible Synthetic Ground Motion
2. Shake Analysis
3. Liquefaction Analysis
4. Abutment Analysis
5. Slope Stability Analysis

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Slope Stability of Abutment Fills

Seven cross-sections from the St. Francis River Bridge site were selected for slope stability analysis (Figure 5.5), as were seven from the Wahite Ditch Bridge site (Figure 5.6). At both sites, the cross-sections represented the steepest site slopes. 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. references.

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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.

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SOIL PROPERTY ESTIMATION Cont.

•For cohesionless soils, $(N1)_{60}$ 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 Bridge site and the Wahite Ditch Bridge site

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Table 8.4: Soil Properties used for the Slope Stability Analysis, St. Francis River Bridge Site

Soil Characteristics*				
Class	γ_{moist} (pcf)	$\gamma_{saturated}$ (pcf)	c (psf)	ϕ (deg.)
CL	121.34	133.50	858	30
ML	106.00	122.50	450	34
SM	115.00	127.00	50	35
SP	134.90	141.90	0.0	40

* Soil characteristics obtained from slope stability procedures, Section (5.5.1)

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

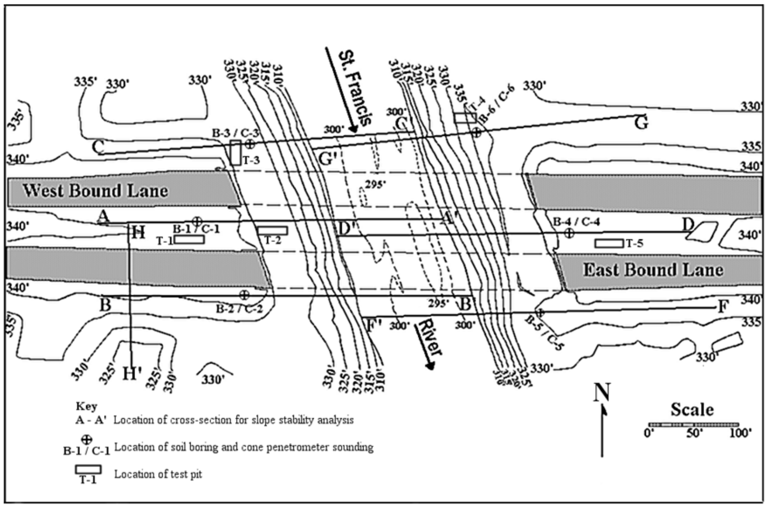

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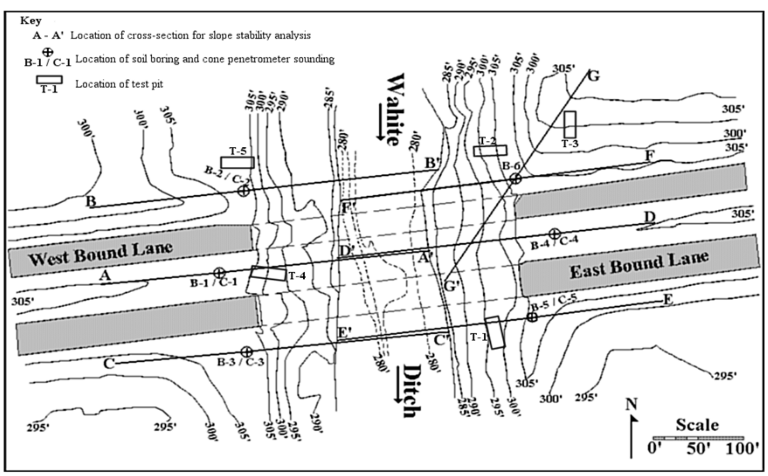
Figure 5.6: Wahite Ditch Bridge Site Topography, Cross-Sections and Boring Locations



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Design Horizontal and Vertical Earthquake Accelerations in Slope Stability Analysis

Three sets of ground accelerations were selected for the St. Francis River Bridge site and the Wahite Ditch Bridge site based on the SHAKE91 analysis. 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 (See Table 5.3).

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Table 5.3: Design Horizontal and Vertical Earthquake Accelerations for Slope Stability Analysis

a. Francis River Bridge 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 Bridge 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

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Figure 8.11: Example Slope Stability Results for St. Francis River Bridge Site

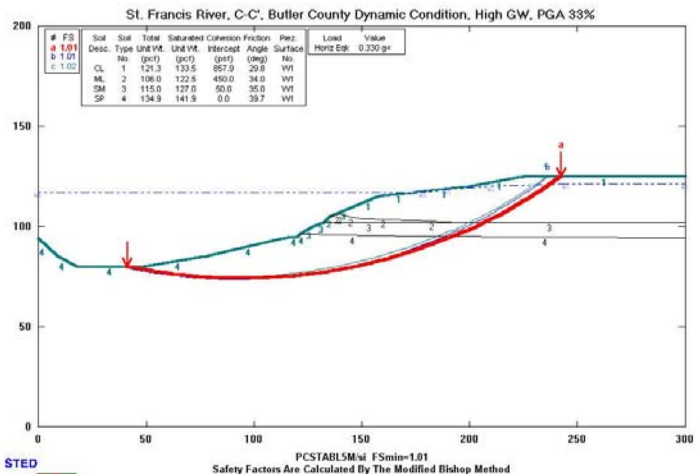

Table 8.5: Slope Stability Results for the St. Francis River Bridge Site

Factor of Safety for Most Sensitive Potential Failure Plane							
Cross-Section	A - A'	**B – B'**	**C – C'**	**D –D'**	**E – E'**	**F – F'**	**G – G'**
Static							
Low GW	2.63	2.76	2.88	2.71	2.52	1.93	3.96
High GW	3.06	3.14	3.48	3.23	2.87	2.02	2.67
Pseudo-Static Set 1* 10% PE in 50 years							
Low GW (0.135)	1.73	1.74	1.82	1.79	1.59	1.41	2.60
High GW (0.135)	1.61	1.68	1.78	1.72	1.64	1.23	1.74
2% PE in 50 years							
Low GW (0.331)	1.31	1.10	1.17	1.18	1.08	0.98	1.71
High GW (0.331)	0.93	0.97	1.01	1.00	0.94	0.74	1.08

*** Peak ground acceleration values calculated with the computer program *SHAKE91* Section 5.4.**

Table 8.5: Slope Stability Results for the St. Francis River Bridge Site, Cont.

Pseudo-Static Set 2 10% PE (HGA, VGA)							
Low GW (0.135,+0.048)	1.68	1.64	1.76	1.74	1.55	1.39	2.59
Low GW (0.135,-0.048)	1.77	1.75	1.87	1.83	1.62	1.43	2.62
High GW (0.135,+0.048)	1.55	1.61	1.71	1.66	1.54	1.19	1.64
High GW (0.135,-0.048)	1.67	1.73	1.84	1.77	1.63	1.26	1.75
2% PE (HGA, VGA)							
Low GW (0.331,+0.170)	0.95	0.91	0.97	0.99	0.92	0.84	1.58
Low GW (0.331,-0.170)	1.28	1.26	1.33	1.32	1.20	1.08	1.82
High GW (0.331,+0.170)	0.70	0.74	0.78	0.78	0.74	0.57	0.88
High GW (0.331,-0.170)	1.10	1.14	1.20	1.17	1.09	0.86	1.25
Pseudo-Static Set 3 10% PE (HGA, VGA)							
Low GW (0.012,+0.090)	2.50	2.50	2.71	1.80	2.21	1.89	3.91
Low GW (0.012,-0.090)	2.57	2.61	2.81	1.95	2.24	1.89	3.74
High GW (0.012,+0.090)	2.89	2.98	3.29	3.08	2.74	1.95	2.50
High GW (0.012,-0.090)	2.87	2.94	3.25	3.02	2.70	1.91	2.62
2% PE (HGA, VGA)							
Low GW (0.014,+0.221)	2.39	2.37	2.58	2.49	2.14	1.88	4.06
Low GW (0.014,-0.221)	2.59	2.66	2.86	2.66	2.23	1.89	3.65
High GW (0.014,+0.221)	2.90	2.46	3.28	3.11	2.78	1.95	2.34
High GW (0.014,-0.221)	2.85	2.91	3.21	2.96	2.68	1.88	2.67

*** Peak ground acceleration values calculated with the computer program *SHAKE91* Section 5.4.**

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CONCLUSIONS:

1. Credible synthetic ground motion has been selected for St.Francis and Wahite sites.
2. Wave propagation analysis shows that there is amplification of rock motion to the base of abutment for 10%PE and attenuation of rock motion for 2% PE.
3. The soils tend to liquefy under high seismic ground motion.
4. The bridge abutments experience displacement of the order of 2.6cm (St.Francis) to 1.2cm (WAHITE). The piles are likely to be stable at their joint with the abutment for this order of displacements.
5. Slopes do become unstable under severe ground motions as both the St. Francis and the Wahite Ditch Sites.

RECOMMENDATION:

1. Densification of soils if liquefaction occurs.

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QUESTIONS ??

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Earthquake hazard mitigation

- Metallic dampers will be introduced to retrofit bridges for reduction of seismic responses of bridges by means of isolation and damping.

Fiber reinforced polymers wrapping for improved column capacity

- Columns externally wrapped with FRP sheets and/or reinforced with near-surface embedded FRP rods were shown effective in increasing design strength. They can be used to enhance the seismic performance of bridges.

Mitigation measures for potential flooding problems

- Mitigation measures for potential flooding problems, such as levee embankment stabilization and surface water diversion, will be evaluated and prioritized.

Mitigation of Liquefaction

- Mitigation measure for liquefaction such as gravel and wick drains and other similar measures will be evaluated for use at these sites.

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END OF SLIDE SHOW

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