MEMORANDUM

TO: Mike Rigsby, P.E., PBQD
Jim Schettler, P.E., Jacobs Civil, Inc.
Eric Peiffer, P.E., Jacobs Civil, Inc.

FROM: Hisham Sarieddine, P.E.
Gerard Buechel, P.E.
Ming-Jiun (Jim) Wu, P.E., Ph.D.

DATE: January 17, 2003

RE: PRELIMINARY DEEP FOUNDATION ENGINEERING ANALYSES, EXISTING PILES, ALASKAN WAY VIADUCT PROJECT

This memorandum presents the results of our preliminary foundation engineering analyses for evaluating the capacities of the existing piles supporting Bents 54 through 130 of the Alaskan Way Viaduct. Our evaluation is based on information included in design drawings approved for construction on October 13, 1950, for the existing viaduct provided by PBQD/Jacobs Civil. We understand that as-built drawings for the viaduct are not available.

Based on the design drawings for the existing viaduct, Bents 69 through 100 and Bent116 are supported on 12x53 (12-inch size, 53 pounds/foot weight), steel H-piles. The remaining bents, Bents 54 through 68 as well as Bents 101 through 130 (except Bent 116), are supported on precast, concrete piles. The precast, concrete piles are 16-inch octagonal in size with the bottom 16 feet tapering from 16 inches down to 8 inches at the pile tip. The design drawings also present the design length of the piles for each bent and the design load of each pile type. Based on available information, the approximate tip elevations of the existing piles are shown on the attached Figure 1. The design loads presented on the available drawings include the following:

Load Type	Steel H-pile	Precast Concrete Pile
"Dead + Live"	48 tons	40 tons
"Dead + Live + Temp + Earthquake + Traction"	60 tons	50 tons

The design drawings indicate that the piles were to be driven using a hammer with a minimum driving energy of 15,000 foot-pounds. They also indicate that the piles should be driven to a sufficient depth to develop a minimum design load of 48 tons for the steel, H-piles and 40 tons for the precast, concrete piles.

Our preliminary engineering analyses were performed based on subsurface conditions encountered in recent and previous field explorations completed along the project alignment. Subsurface conditions underlying the project alignment generally consist of Holocene deposits overlying Pre-Vashon Glacial soils. The Holocene deposits consist of fill (Hf) followed by Estuarine (He), Alluvium (Ha), and Beach (Hb) deposits. The Pre-Vashon Glacial soils underlying the Holocene deposits consist of Glaciomarine Drift (Qpgm), Glacial Outwash (Qpgo), and Glaciolacustrine Deposits (Qpgl). Groundwater depth is estimated to range from 5 to 10 feet below the current ground surface.

During an earthquake, the Holocene deposits are generally susceptible to liquefaction. The earthquake shaking would result in development of positive excess porewater pressure in the saturated cohesionless soils. As discussed in the Washington Department of Transportation's (WSDOT's) technical report titled "Seismic Vulnerability of the Alaskan Way Viaduct – Geotechnical Engineering Aspects", dated July 1995 (WSDOT's 1995 Report), the porewater pressure in the dense soils underlying the liquefied soils would be much lower than that in the upper, liquefied soils at the end of an earthquake. As a result, the hydraulic gradient in the liquefied soils would initially cause water to flow down into the dense soils and thus increase the excess porewater pressure in these soils. Typically, the porewater pressure buildup in the dense granular soils is very small under seismic loading conditions, because dense soils tend to dilate under dynamic loading.

An increase in porewater pressure in the dense soils would result in a reduction of the effective overburden pressure and consequently a decrease of the shear strength of these soils. Figures 6.37 through 6.42 of WSDOT's 1995 Report present an estimated variation of excess porewater pressure, in terms of Pore Pressure Ratio (Ru), with depth at different times after the beginning of earthquake shaking, and an estimated variation of Ru with time at a depth of 2.5 feet below the top of dense soils. The estimated Ru values can be used to estimate the reduction in shear strength of the dense soils at different times during and after the earthquake.

Based on ground motion plots presented on Figure 4.3 (a copy of this figure is attached) of WSDOT's 1995 Report, maximum ground motions are estimated to occur 5 to 15 seconds after the start of the earthquake and negligible ground motions are anticipated after about 50 seconds. Figures 6.37 and 6.38 (a copy of these figures is attached) present a variation of Ru with time at a depth of 2.5 feet below the top of dense soils for 20 and 30 feet of soft soils overlying the dense soils, respectively. These figures show an estimated Ru value of about 0.25 after 10 seconds, and an estimated maximum Ru value of about 0.65 after 75 to 100 seconds. The reported variations of ground motions and estimated Ru values indicate that the maximum anticipated ground motions and maximum Ru values do not occur at the same time. Based on the data discussed above, maximum Ru values occur after ground motions have become negligible.

The purpose of our evaluation is to estimate the axial capacity of the existing piles and to develop soil input parameters to perform lateral resistance analyses on these piles using the computer program LPILE^{PLUS} (Ensoft, 2002). The axial capacity analyses and the provided LPILE parameters consider both static and seismic loading conditions. As discussed previously, during an earthquake, the Holocene deposits are susceptible to liquefaction, and the underlying dense soils may experience a strength reduction during and after the earthquake. Therefore, our preliminary analyses and subsequent preliminary recommendations assume that the loose to medium dense, Holocene deposits would liquefy and the underlying dense soils would experience some strength reduction during seismic loading as indicated in WSDOT's 1995 Report.

Based on available subsurface information, the potentially liquefiable Holocene deposits are generally underlain by either dense to very dense Holocene Hb deposits, or by Pre-Vashon Glacial soils. In our opinion, strength reduction due to earthquake-induced excess porewater pressure, would likely occur in the dense to very dense Hb deposits and possibly in approximately the upper 3 to 5 feet of the Pre-Vashon, granular Glacial soils. Beneath approximately the upper 3 to 5 feet, strength reduction in the Pre-Vashon, granular Glacial soils, in our opinion, would be minimal.

We estimated the strength reduction in the dense to very dense Hb deposits due to earthquake-induced excess porewater pressure using two Ru values. An estimated Ru value of 0.25 (10 seconds after start of earthquake) was used to determine the strength reduction during peak ground motion, and an estimated Ru value of 0.65 was used to determine the maximum strength

reduction. In both cases, liquefiable conditions are assumed in the upper Holocene deposits. As discussed previously, based on WSDOT's 1995 Report, the maximum Ru would occur after ground motions have become negligible. Therefore, we recommend that static loading conditions be considered under the maximum Ru condition.

Based on soil conditions underlying the existing viaduct alignment and the estimated embedment of the existing piles into the dense soils based on available data, we grouped Bents 54 through 130 into five different areas. Presented below is a description for each area along with our preliminary engineering evaluation of the existing piles in that group.

Bents 54 Through 57

The existing piles in this group consist of precast concrete piles. Based on provided design information and available subsurface information, these piles appear to be embedded more than five feet into glacially consolidated Qpgo deposits. In our opinion, these piles would provide an allowable static axial capacity of 40 tons under static loading conditions. It is also our opinion that strength reduction in the Qpgo deposits greater than 3 to 5 feet, due to earthquake-induced excess porewater pressure, would be minimal. As a result, these piles would have a suitable factor-of-safety (FS) under seismic loading conditions, despite the fact that the upper Holocene soils would likely liquefy.

Also, due to liquefaction of the upper Holocene deposits, the lateral capacity of the piles would be reduced under seismic loading conditions. We recommend that the lateral capacity of these piles be evaluated.

Bents 58 Through 65

The existing piles in this group consist of precast concrete piles. Available subsurface information shows that the site in this area is underlain by potentially liquefiable Holocene deposits overlying very dense Hb deposits and very stiff He deposits. The He deposits extend to an approximate depth of 50 feet below existing ground surface, and they are underlain by Pre-Vashon Glacial soils. Driving records for a test pile installed in this segment indicate that the pile was likely driven into the underlying Pre-Vashon Glacial soils. However, lengths presented on available design drawings indicate that the existing piles are probably bearing in the He deposits.

Attached Figure 2 provides our estimated ultimate capacity for 16-inch, tapered, prestressed concrete piles versus depth. Estimated ultimate compressive capacities are provided under static loading conditions, as well as, under seismic loading conditions using three Ru values. The Ru values were used to estimate strength reduction of the very dense Hb deposits (Ru = 0 assume no reduction in strength). The allowable pile compression capacity can be obtained by applying an appropriate FS to the ultimate compressive capacity plots. Our capacity estimates indicate that existing piles supporting Bents 58 through 65 would not have an adequate FS during or immediately after an earthquake event. We recommend that the capacities of these piles be further evaluated.

Table 1 provides our recommended parameters for input into the LPILE^{PLUS} program under static, cyclic, and seismic loading conditions. Under seismic loading conditions, recommended parameters are provided for the very dense Hb deposits (Soil Layer 4) using Ru values of 0.25 and 0.65.

Recommended LPILE^{PLUS} parameters presented in Table 1 do not reflect the effect of deep foundation group action. The recommended efficiency (reduction) factors due to group action are presented in Table 3. The efficiency factors are based on recommendations presented in a 1998 ENSOFT Seminar (ENSOFT, 1998).

Bents 66 Through 68

The existing piles in this group consist of precast concrete piles. Based on provided design information and available subsurface information, these piles could be embedded less than 5 feet into Qpgm deposits. In our opinion, these piles would provide an allowable static axial capacity of 40 tons under static loading conditions. However, earthquake-induced excess porewater pressures could reduce the shear strength in the upper 3 to 5 feet of the Qpgm deposits. In our opinion, such reduction may reduce the FS against applied axial loads to less than one during and immediately after an earthquake event. We recommend that the capacities of these piles be further evaluated.

Also, due to liquefaction of the upper Holocene deposits, the lateral capacity of the piles would be reduced under seismic loading conditions. We recommend that the lateral capacity of these piles be evaluated.

Bents 69 Though 100 and Bent 116

The existing piles supporting these bents consist of steel H-piles. Based on provided design information and available subsurface information, these piles appear to be embedded more than five feet into Pre-Vashon Glacial deposits. In our opinion, these piles would provide an allowable static axial capacity of 48 tons under static loading conditions. It is also our opinion that strength reduction in the Pre-Vashon Glacial deposits greater than 3 to 5 feet, due to earthquake-induced excess porewater pressure, would be minimal. As a result, these piles would have a suitable FS under seismic loading conditions, despite the fact that the upper Holocene soils would likely liquefy.

Also, due to liquefaction of the upper Holocene deposits, the lateral capacity of the piles would be reduced under seismic loading conditions. We recommend that the lateral capacity of these piles be evaluated.

Bents 101 Through 102

The existing piles in this group consist of prestressed concrete piles. Based on provided design information and available subsurface information, these piles appear to be embedded more than five feet into Qpnf deposits. In our opinion, these piles would provide an allowable static axial capacity of 40 tons under static loading conditions. It is also our opinion that strength reduction in the Qpnf deposits greater than 3 to 5 feet, due to earthquake-induced excess porewater pressure, would be minimal. As a result, these piles would have a suitable FS under seismic loading conditions, despite the fact that the upper Holocene soils would likely liquefy.

Also, due to liquefaction of the upper Holocene deposits, the lateral capacity of the piles would be reduced under seismic loading conditions. We recommend that the lateral capacity of these piles be evaluated.

Bents 103 Through 110

The existing piles in this group consist of prestressed concrete piles. Available subsurface information show that the site in this area is underlain by potentially liquefiable Holocene deposits overlying very dense Hb deposits. The He deposits extend to an approximate depth of 40 feet below existing ground surface, and they are underlain by Pre-Vashon Glacial soils.

Lengths presented on available design drawings indicate that the existing piles are probably bearing in the very dense Hb deposits.

Attached Figure 3 provides our estimated ultimate capacity for 16-inch, tapered, prestressed concrete piles versus depth. Estimated ultimate compressive capacities are provided under static loading conditions and under seismic loading conditions using three Ru values. The Ru values were used to estimate strength reduction of the very dense Hb deposits (Ru = 0 assume no reduction in strength). The allowable compression capacity can be obtained by applying an appropriate FS to the ultimate compressive capacity plots. Our capacity estimates indicate that existing piles supporting Bents 103 through 110 would not have an adequate FS during or immediately after an earthquake event. We recommend that the capacities of these piles be further evaluated.

Table 2 provides our recommended parameters for input into the LPILE^{PLUS} program under static, cyclic, and seismic loading conditions. Under seismic loading conditions, recommended parameters are provided for the very dense Hb deposits (Soil Layer 3) using Ru values of 0.25 and 0.65.

Recommended LPILE^{PLUS} parameters presented in Table 2 do not reflect the effect of deep foundation group action. The recommended efficiency (reduction) factors due to group action are presented in Table 3. The efficiency factors are based on recommendations presented in a 1998 ENSOFT Seminar (ENSOFT, 1998).

Bents 111 Through 130

The existing piles in this group consist of prestressed concrete piles, except Bent 116, which consists of steel H-piles. Based on provided design information and available subsurface information, these piles could be embedded less than five feet into Pre-Vashon Glacial deposits. In our opinion, these piles would provide an allowable static axial capacity of 40 tons under static loading conditions. However, earthquake-induced excess porewater pressures could reduce the shear strength in the upper 3 to 5 feet of the Pre-Vashon Glacial deposits. In our opinion, such reduction may reduce the FS against applied axial loads to less than one during and immediately after an earthquake event. We recommend that the axial capacities of these piles be further evaluated.

Also, due to liquefaction of the upper Holocene deposits, the lateral capacity of the piles would be reduced under seismic loading conditions. We recommend that the lateral capacity of these piles be also evaluated.

Foundation Capacity Increase

We understand that the current viaduct retrofit scheme includes utilizing the axial capacity of the existing pile foundations. Based on our preliminary analyses and the results of WSDOT's 1995 Report, it appears that the axial capacity of several of the prestressed concrete pile foundations that do not have an embedment into the glacially consolidated soils greater than about 5 feet would not have sufficient axial capacity under seismic loading conditions.

One method to increase the capacity of the piles would be to utilize jet grouting around the pile tips to mitigate the potential pore pressure increase in the soils and the resulting pile capacity reduction. For conceptual cost estimates, we recommend that jet grouting be considered at the existing pile foundations. The jet grout zone should extend outside the limits of the piles a distance of about 4 to 5 feet and about 5 feet below the top of the glacial soils. The top of the jet grout zone would depend upon the depth of embedment of the piles into the glacially consolidated materials. For conceptual cost estimating purposes, it is recommended that the jet grout extend along the pile a distance of about 5 feet above the top of the glacial soils.

Another method to increase the capacity of the existing pile foundations would be to modify the existing pile cap and install additional piles, such as mini-piles within the foundation. We recommend that both additional piles and jet grouting be evaluated further if the retrofit option is pursued.

REFERENCES

ENSOFT, Inc., 1998, Design of deep foundations: piles and drilled shafts under lateral and axial loadings, a seminar/workshop featuring computer programs from ENSOFT, Inc., Austin, Texas, April 23-24, 1998: Austin, Tex.

Reese, L.C., and Wang, S.T., 2002, Technical manual of documentation of computer program LPILE^{PLUS} 4.0 for Windows, a program for the analysis of piles and drilled shafts under lateral loads: Austin, Texas, ENSOFT, Inc.

HJS:GJB:JW/hjs

Enclosures:
- Table 1 – Recommended Parameters for Development of P-Y Curves Using LPILE^{PLUS}, Bents 58 through 65
- Table 2 – Recommended Parameters for Development of P-Y Curves Using LPILE^{PLUS}, Bents 103 through 110
- Table 3 – LPILE^{PLUS} Efficiency Factors for Pile Groups
- Figure 1 – Generalized Subsurface Profile and Estimated Existing Pile Tip Elevations
- Figure 2 – Estimated Ult. Compression Capacity, Existing Prestressed Concrete Piles, Bents 58 through 65
- Figure 3 – Estimated Ult. Compression Capacity, Existing Prestressed Concrete Piles, Bents 103 through 110
- Copies of Figure 4.3, 6.37, and 6.38 of WSDOT's 1995 Report

TABLE 1

Recommended Parameters for Development of P-Y Curves Using LPILE^{PLUS}
Bents 58 through 65

Boring No.	Approximate Ground Elevation at Boring Location (feet)	Soil Layer	Upper Boundary Depth (feet)	Lower Boundary Depth (feet)	Soil Type	Effective Unit Weight, γ' (pcf)			Cohesion, c (psf)		Friction Angle, ϕ (°)		Initial Modulus of Subgrade Reaction, k (pci)			ε_{50}
						Static/ Cyclic	Liquefied Ru = 0.25 (3)	Liquefied Ru = 0.65 (3)	Static	Cyclic/ Liquefied	Static/ Cyclic	Liquefied	Static/ Cyclic	*Liquefied Ru = 0.25 (3)*	*Liquefied Ru = 0.65 (3)*	
011-0087	16	1	0	10	Sand	110	83	39	0	0	30	20	35	23	23	-
		2	10	16	Sand	47	35	16	0	0	30	5	25	2	2	-
		3	16	26	Sand	58	44	20	0	0	34	5	90	9	9	-
		4	26	36	Sand	62	47	22	0	0	38	38	125	100	50	-
		5	36	53	Stiff Clay	58	58	58	2,500	2,000	0	0	650	350	350	0.005
		6	53	-	Sand	62	62	62	0	0	40	40	125	125	125	-

Notes:

(1) Parameters given above are based on subsurface conditions encountered in indicated boring.

(2) Based on subsurface conditions along alignment, scatterred zones of soil may liquefy under earthquake loading. Seismic loading may also result in strength reduction for some soil layers mainly soils overlying liquefied zones. Parameters under seismic loading are provided.

(3) Pore Pressure Ratio (Ru) values for the underlying very dense beach deposits (Soil Layer 4) were estimated from figures presented in WSDOT's 1995 Report. An Ru = 0.25 was estimated about 10 seconds after start of earthquake, and a maximum Ru = 0.65 was estimated after about 85 seconds.

(4) Parameters given above do not reflect effect of deep foundation group action.

TABLE 2

Recommended Parameters for Development of P-Y Curves Using LPILE^{PLUS}
Bents 103 through 110

Boring No.	Approximate Ground Elevation at Boring Location (feet)	Soil Layer	Upper Boundary Depth (feet)	Lower Boundary Depth (feet)	Soil Type	Effective Unit Weight, γ' (pcf)			Cohesion, c (psf)		Friction Angle, ϕ (°)		Initial Modulus of Subgrade Reaction, k (pci)			ε_{50}
						Static/ Cyclic	Liquefied Ru = 0.25 (3)	Liquefied Ru = 0.65 (3)	Static	Cyclic/ Liquefied	Static/ Cyclic	Liquefied	Static/ Cyclic	Liquefied Ru = 0.25 (3)	Liquefied Ru = 0.65 (3)	
TW-21	16	1	0	10	Sand	115	86	40	0	0	32	22	60	40	40	-
		2	10	25	Sand	47	35	16	0	0	30	5	50	5	5	-
		3	25	40	Sand	62	47	22	0	0	38	38	125	100	50	-
		4	40	-	Sand	62	62	62	0	0	40	40	125	125	125	-

Notes:

(1) Parameters given above are based on subsurface conditions encountered in indicated boring.

(2) Based on subsurface conditions along alignment, scatterred zones of soil may liquefy under earthquake loading. Seismic loading may also result in strength reduction for some soil layers mainly soils overlying liquefied zones. Parameters under seismic loading are provided.

(3) Pore Pressure Ratio (Ru) values for the underlying very dense beach deposits (Soil Layer 3) were estimated from figures presented in WSDOT's 1995 Report. An Ru = 0.25 was estimated about 10 seconds after start of earthquake, and a maximum Ru = 0.65 was estimated after about 85 seconds.

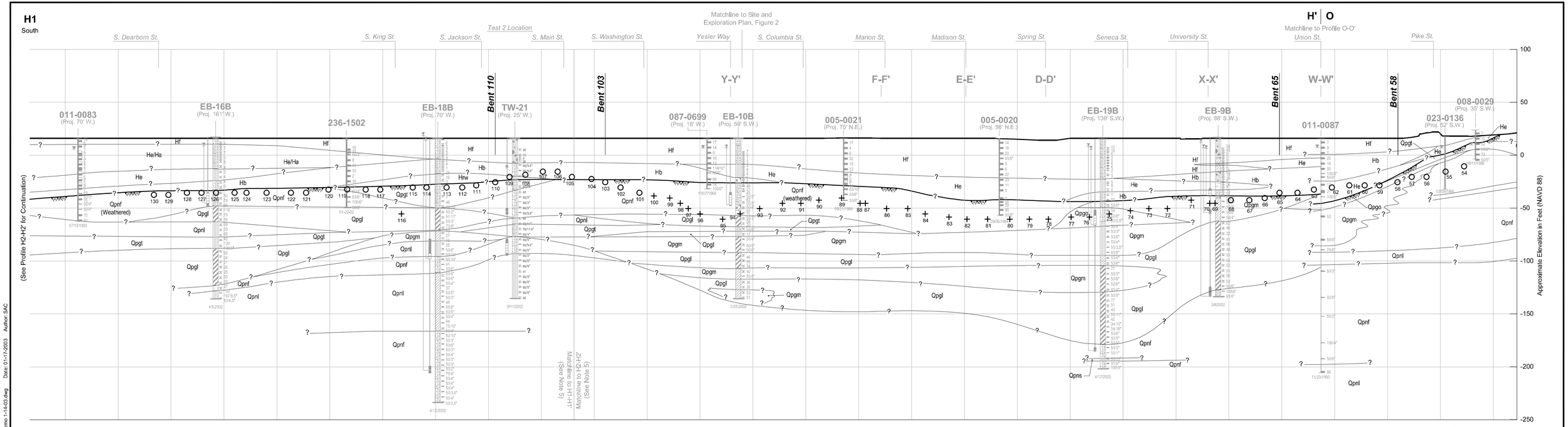
(4) Parameters given above do not reflect effect of deep foundation group action.

TABLE 3

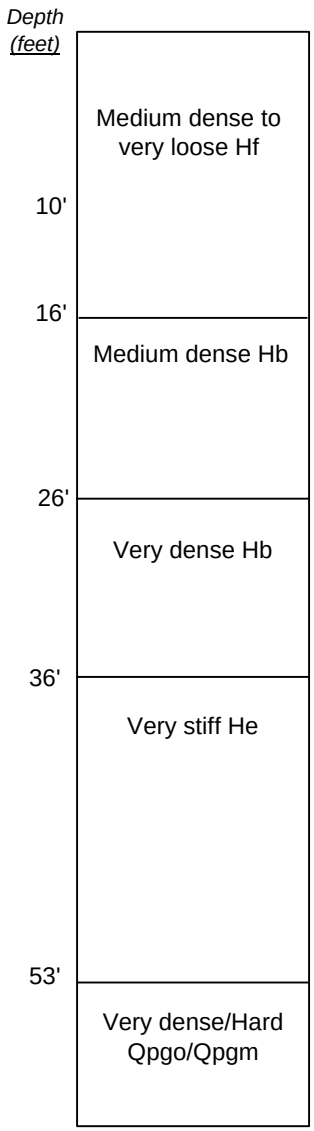
LPILE^{PLUS} Efficiency Factors for Pile Groups

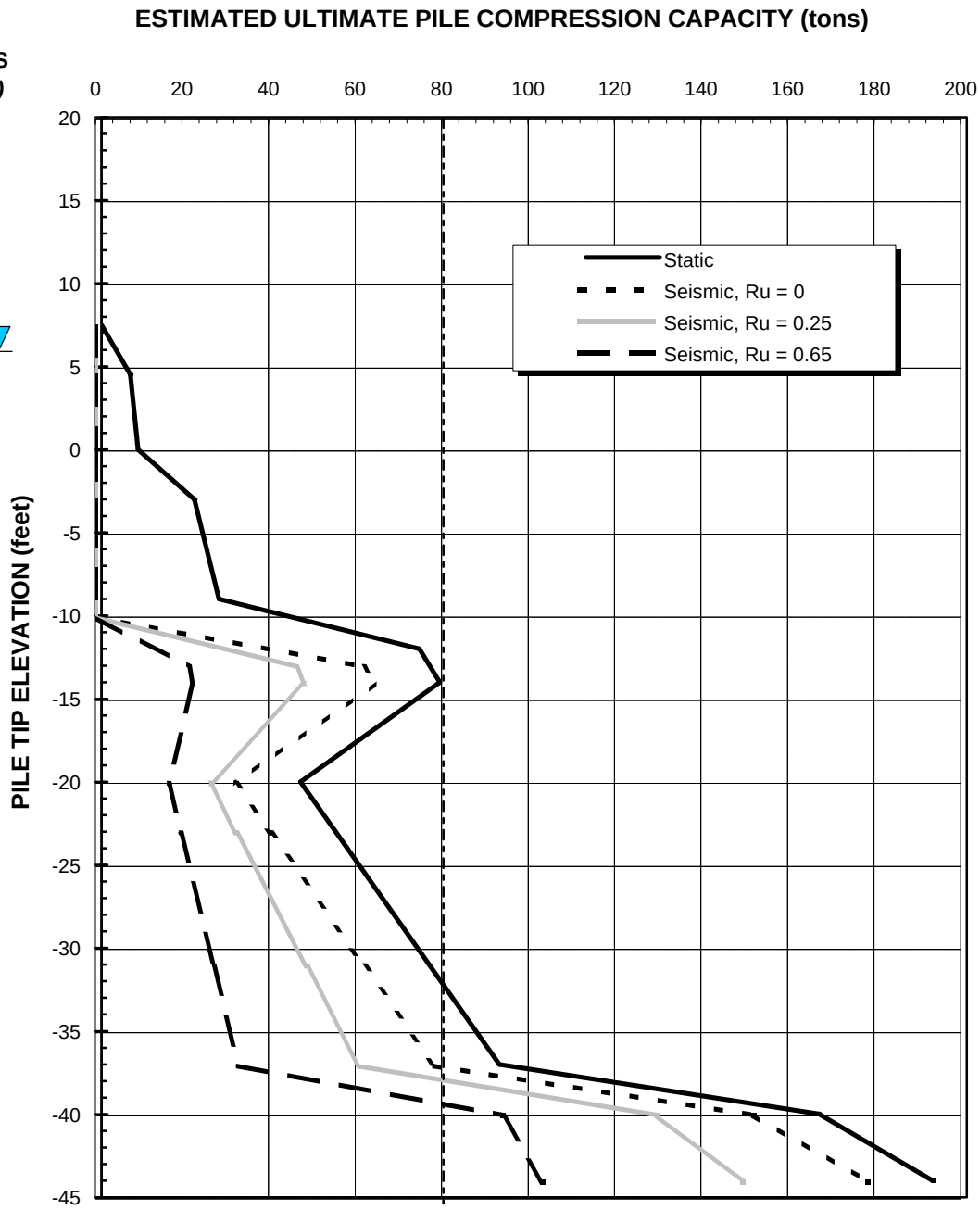
Spacing	Efficiency Factor, P_m
2D	0.28
2.5D	0.50
3D	0.72
3.5D	0.85
4D	0.91
4.5D	0.94
5D	0.98
5.5D	1.00

NOTE:
The efficiency factors are based on recommendations presented in a 1998 ENSOFT Seminar (ENSOFT 1998)


H1
South
H' | O
Matchline to Profile O-O'
Matchline to Site and Exploration Plan, Figure 2
S. Dearborn St.
S. King St.
S. Jackson St.
Test 2 Location
S. Main St.
S. Washington St.
Yesler Way
S. Columbia St.
Marion St.
Madison St.
Spring St.
Seneca St.
University St.
Union St.
Pike St.
Bent 110
Bent 103
Bent 65
Bent 58
Y-Y'
F-F'
E-E'
D-D'
X-X'
W-W'
011-0083 (Proj. 70' W.)
EB-16B (Proj. 161' W.)
236-1502
EB-18B (Proj. 70' W.)
TW-21 (Proj. 25' W.)
087-0699 (Proj. 18' W.)
EB-10B (Proj. 59' S.W.)
005-0021 (Proj. 75' N.E.)
005-0020 (Proj. 98' N.E.)
EB-19B (Proj. 139' S.W.)
EB-9B (Proj. 88' S.W.)
011-0087
023-0136 (Proj. 52' S.W.)
008-0029 (Proj. 35' S.W.)
(See Profile H2-H2' for Continuation)
Matchline to H1-H1' (See Note 5)
Matchline to H2-H2' (See Note 5)
Approximate Elevation in Feet (NAVD 88)
100
50
0
-50
-100
-150
-200
-250
Hf
He/Ha
He
Hb
Hrw
Qpnf (Weathered)
Qpgl
Qpnl
Qpgt
Qpgm
Qpnf
Qpns
Qpgo
0 200 400
Horizontal Scale in Feet
0 50 100
Vertical Scale in Feet
Vertical Exaggeration = 4X
NOTE
Shown elevations assume bottom of pile cap is 6.5 feet below ground surface elevation, and elevation at ground surface equals 16 feet.
LEGEND
101 Concrete Pile Designation and Approximate Location and Tip Elevation
100 Steel Pile Designation and Approximate Location and Tip Elevation
DRAFT
Alaskan Way Viaduct and Seawall Project
Seattle, Washington
GENERALIZED SUBSURFACE PROFILE AND ESTIMATED EXISTING PILE TIP ELEVATIONS
January 2003
21-1-09490-306
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants
FIG. 1
File: I:\Drafting\21\09490\306\21-1-09490-306 Memo 1-14-03.dwg Date: 01-17-2003 Author: SAC

GENERALIZED SUBSURFACE CONDITIONS
(Based on Boring 011-0087)





NOTES

1. Compressive capacity is a summation of skin friction and end bearing.
2. Allowable capacities can be obtained by applying an appropriate factor-of-safety to the estimated ultimate values.
3. Pore Pressure Ratio (Ru) values for the underlying very dense beach deposits were estimated from figures presented in WSDOT's Final Technical Report, dated July 1995. An Ru = 0.25 was estimated about 10 seconds after start of earthquake, and a maximum Ru = 0.65 was estimated after about 85 seconds. Soil strength parameters were reduced by a factor of (1-Ru) for estimating capacities.
4. Calculations assume ground water at elevation of 6 feet (10 feet depth below existing ground surface).

SR99: Alaskan Way Viaduct Project
Seattle, Washington

ESTIMATED ULT. COMPRESSION CAPACITY EXISTING 16-INCH, TAPERED, PRESTRESSED CONCRETE PILES BENTS 58 THROUGH 65

January 2003 21-1-09490-306

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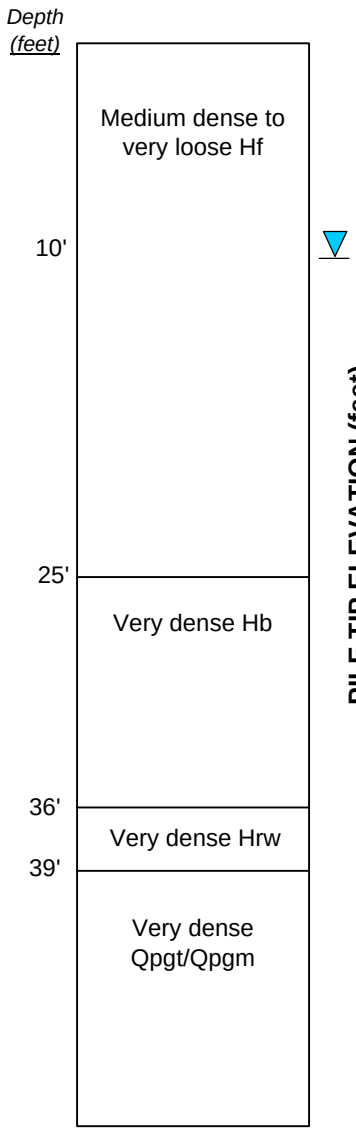
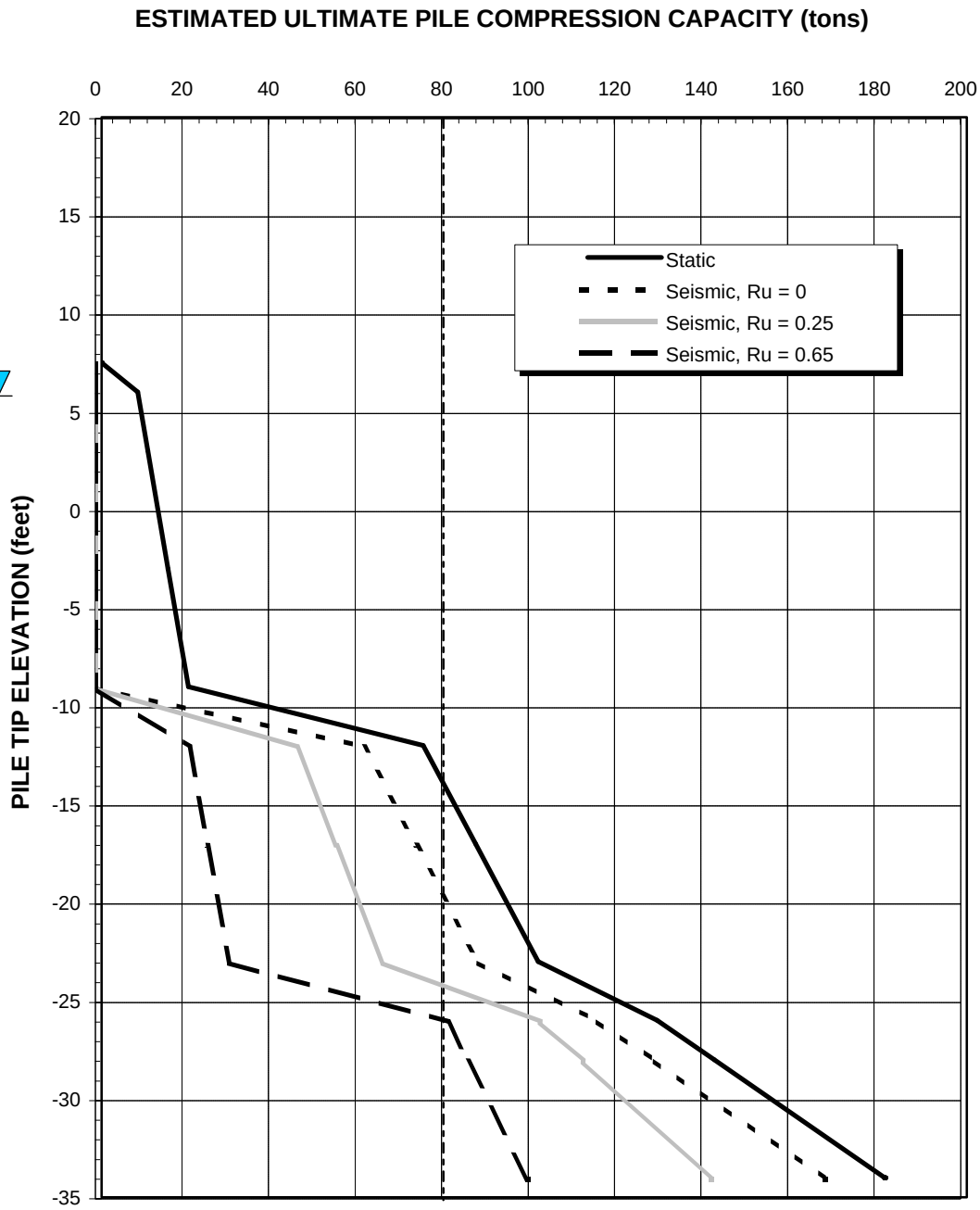
FIG. 2

GENERALIZED SUBSURFACE CONDITIONS
(Based on Boring TW-21)





NOTES

1. Compressive capacity is a summation of skin friction and end bearing.
2. Allowable capacities can be obtained by applying an appropriate factor-of-safety to the estimated ultimate values.
3. Pore Pressure Ratio (Ru) values for the underlying very dense beach deposits were estimated from figures presented in WSDOT's Final Technical Report, dated July 1995. An Ru = 0.25 was estimated about 10 seconds after start of earthquake, and a maximum Ru = 0.65 was estimated after about 85 seconds. Soil strength parameters were reduced by a factor of (1-Ru) for estimating capacities.
4. Calculations assume ground water at elevation of 6 feet (10 feet depth below existing ground surface).

SR99: Alaskan Way Viaduct Project
Seattle, Washington

ESTIMATED ULT. COMPRESSION CAPACITY EXISTING 16-INCH, TAPERED, PRESTRESSED CONCRETE PILES BENTS 103 THROUGH 110

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FIG. 3

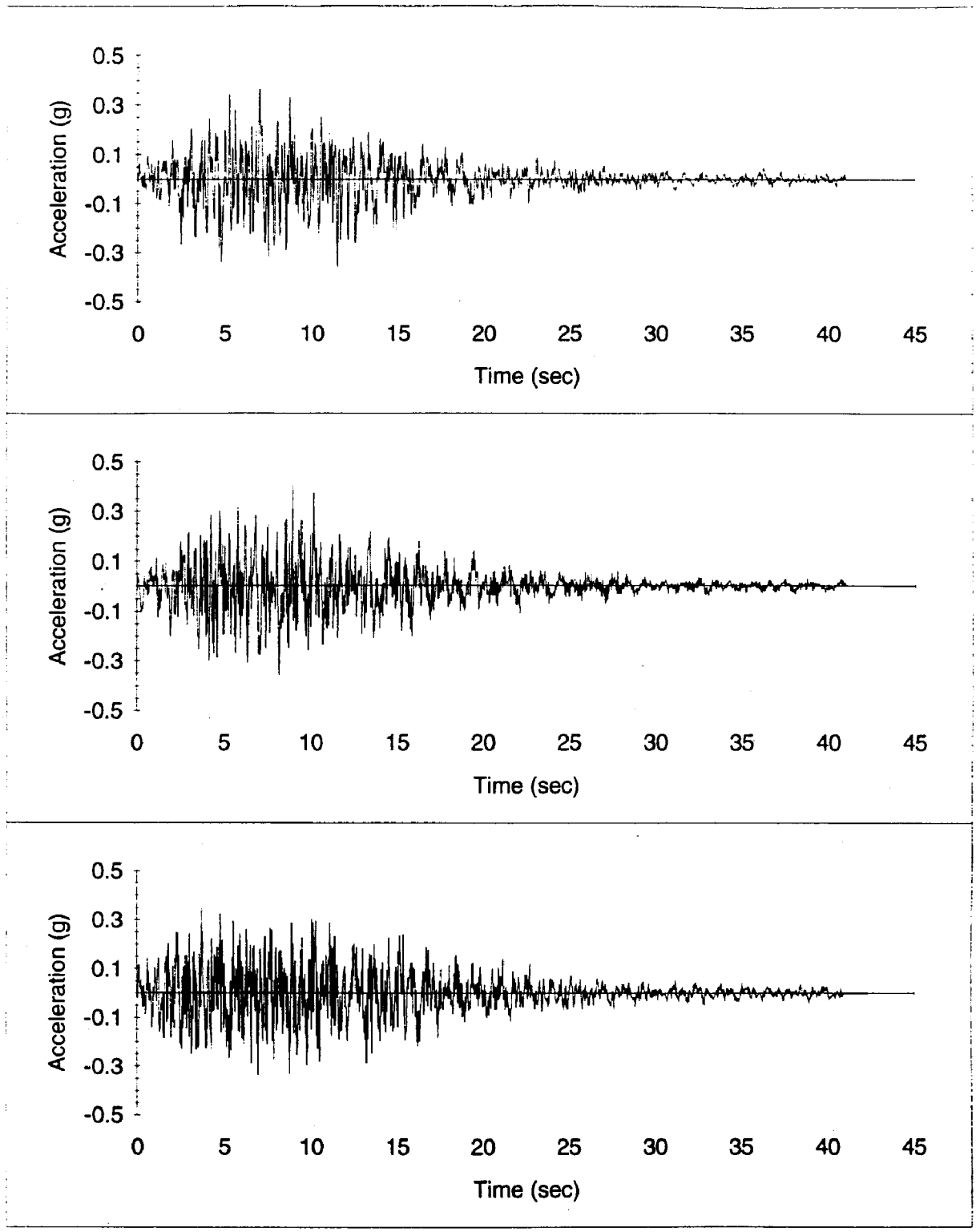

Figure 4.3. Bedrock Outcrop Motions from PSHA and Deconvolution Analyses

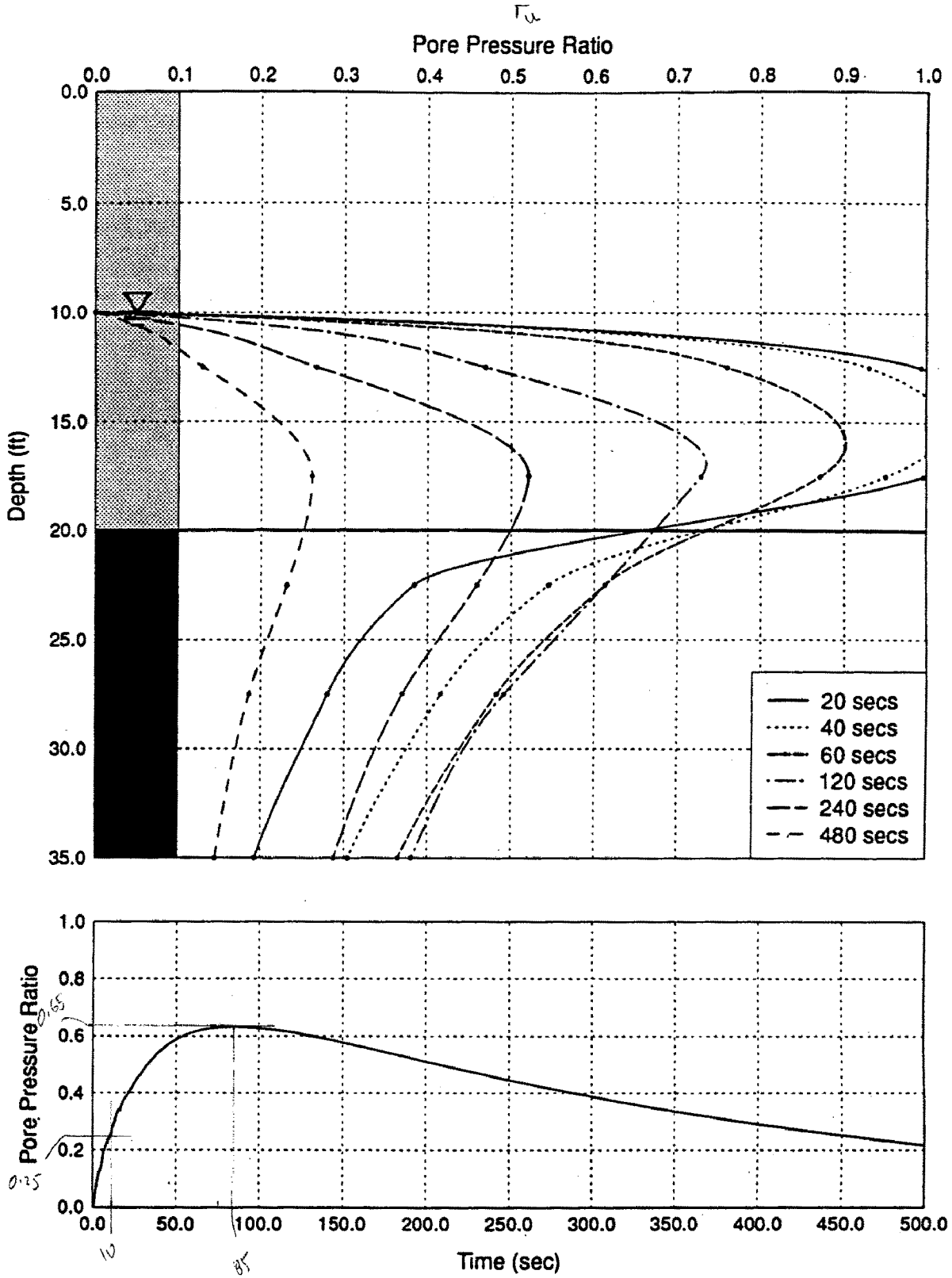

Figure 6.37 (a) Variation of Pore Pressure Ratio with Depth at Different Times after the Beginning of Earthquake Shaking for 20 Ft Soft Soil, and (B) Variation of Pore Pressure Ratio with Time in the Vicinity of the Tips of the Existing Pile Foundations (2.5 Ft Below Top of Till)

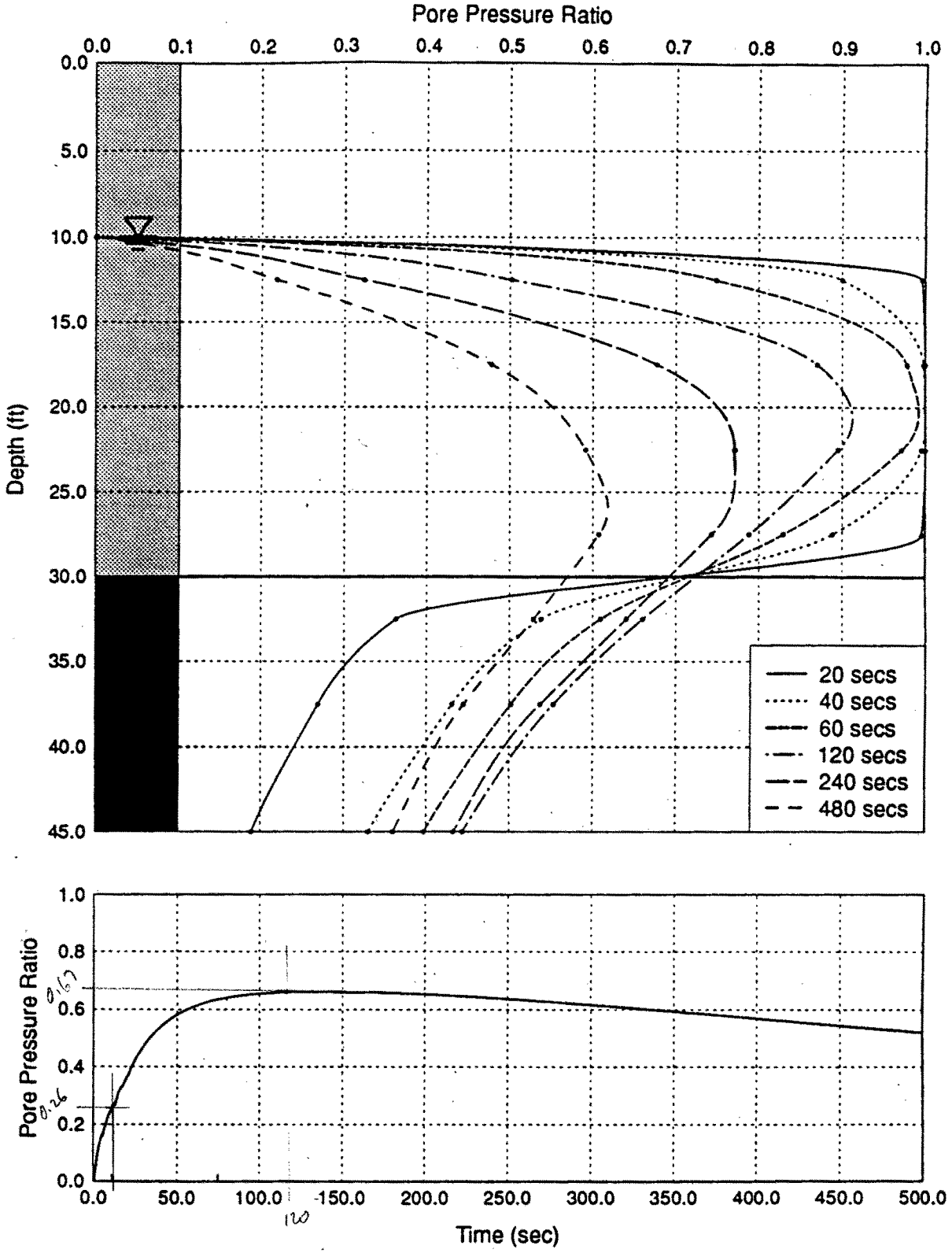

Figure 6.38 (a) Variation of Pore Pressure Ratio with Depth at Different Times after the Beginning of Earthquake Shaking for 30 Ft Soft Soil and (B) Variation of Pore Pressure Ratio with Time in the Vicinity of the Tips of the Existing Pile Foundations (2.5 Ft Below Top of Till)