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Design and Construction of Driven Pile Foundations – Comprehensive Design Examples

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and

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and '15 Interims.*



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16. ABSTRACT This document presents comprehensive design examples for driven pile foundations on highway structures. The worked design examples supplement the material presented in FHWA-NHI-16-009 and FHWA-NHI-16-010, the primary FHWA guidance documents on driven pile foundations. The worked LRFD design examples address strength, service and extreme limit state considerations for a two span bridge structure in highly variable subsurface conditions. Pile foundation design examples in cohesionless, cohesive, and layered soil profiles are presented as well as pile design on hard rock. The worked examples follow the step by step design and construction process outlined in Chapter 2 of FHWA-NHI-16-009.			
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CONVERSION FACTORS

Approximate Conversions to SI Units			Approximate Conversions from SI Units		
When You Know	Multiply By	To Find	When You Know	Multiply By	To Find
(a) Length					
inch (in.)	25.4	millimeter (mm)	millimeter (mm)	0.039	inch (in.)
foot (ft)	0.305	meter (m)	meter (m)	3.28	foot (ft)
yard (yd)	0.914	meter (m)	meter (m)	1.09	yard (yd)
mile (mi)	1.61	kilometer (km)	kilometer (km)	0.621	mile (mi)
(b) Area					
square inches (in ²)	645.2	square millimeters (mm ²)	square millimeters (mm ²)	0.0016	square inches (in ²)
square feet (ft ²)	0.093	square meters (m ²)	square meters (m ²)	10.764	square feet (ft ²)
Acres (ac)	0.405	hectares (ha)	hectares (ha)	2.47	Acres (ac)
square miles (mi ²)	2.59	square kilometers (km ²)	square kilometers (km ²)	0.386	square miles (mi ²)
square inches (in ²)	645.2	square millimeters (mm ²)	square millimeters (mm ²)	0.0016	square inches (in ²)
(c) Volume					
fluid ounces (oz)	29.57	milliliters (mL)	milliliters (mL)	0.034	fluid ounces (oz)
Gallons (gal)	3.785	liters (L)	liters (L)	0.264	Gallons (gal)
cubic feet (ft ³)	0.028	cubic meters (m ³)	cubic meters (m ³)	35.32	cubic feet (ft ³)
cubic yards (yd ³)	0.765	cubic meters (m ³)	cubic meters (m ³)	1.308	cubic yards (yd ³)
(d) Mass					
ounces (oz)	28.35	grams (g)	grams (g)	0.035	ounces
pounds (lb)	0.454	kilograms (kg)	kilograms (kg)	2.205	pounds
short tons (2000 lb) (T)	0.907	megagrams (tonne) (Mg)	megagrams (tonne) (Mg)	1.102	short tons (2000 lb)
(e) Force					
pound (lb)	4.448	Newton (N)	Newton (N)	0.2248	pound (lb)
(f) Pressure, Stress, Modulus of Elasticity					
pounds per square foot (psf)	47.88	Pascals (Pa)	Pascals (Pa)	0.021	pounds per square foot (psf)
pounds per square inch (psi)	6.895	kiloPascals (kPa)	kiloPascals (kPa)	0.145	pounds per square inch (psi)
(g) Density					
pounds per cubic foot (pcf)	16.019	kilograms per cubic meter (kgm ³)	kilograms per cubic meter (kgm ³)	0.0624	pounds per cubic feet (pcf)
(h) Temperature					
Fahrenheit temperature (°F)	5/9(°F- 32)	Celsius temperature (°C)	Celsius temperature (°C)	9/5(°C)+ 32	Fahrenheit temperature (°F)

Notes:

- 1) The primary metric (SI) units used in civil engineering are meter (m), kilogram (kg), second (s), Newton (N), and Pascal (Pa=N/m²).
- 2) In a "soft" conversion, an English measurement is mathematically converted to its exact metric equivalent.
- 3) In a "hard" conversion, a new rounded metric number is created that is convenient to work with and remember.

PREFACE

The purpose of this manual is to provide updated, state-of-the-practice information for the design and construction of driven pile foundations in accordance with the Load and Resistance Factor Design (LRFD) platform. Engineers and contractors have been designing and installing pile foundations for many years. During the past three decades, the industry has experienced several major improvements including newer and more accurate methods of predicting and measuring geotechnical resistance, vast improvements in design software, highly specialized and sophisticated equipment for pile driving, and improved methods of construction control. Previous editions of the FHWA Design and Construction of Driven Pile Foundations manual were published 1985, 1996, and 2006 and chronicled the many changes in design and construction practice over the past 30 years. This two volume edition, GEC-12, serves as the FHWA reference document for highway projects involving driven pile foundations.

Volume I, FHWA-NHI-16-009, covers the foundation selection process, site characterization, geotechnical design parameters and reporting, selection of pile type, geotechnical aspects of limit state design, and structural aspects of limit state design. Volume II, FHWA-NHI-16-010, addresses static load tests, dynamic testing and signal matching, rapid load testing, wave equation analysis, dynamic formulas, contract documents, pile driving equipment, pile accessories, driving criteria, and construction monitoring. Comprehensive design examples are presented in publication FHWA-NHI-16-064.

Throughout this manual, numerous references will be made to the names of software or technology that are proprietary to a specific manufacturer or vendor. Please note that the FHWA does not endorse or approve commercially available products, and is very sensitive to the perceptions of endorsement or preferred approval of commercially available products used in transportation applications. Our goal with this development is to provide recommended technical guidance for the safe design and construction of driven pile foundations that reflects the current state of practice and provides information on advances and innovations in the industry. To accomplish this, it is necessary to illustrate methods and procedures for design and construction of driven pile foundations. Where proprietary products are described in text or figures, it is only for this purpose.

The primary audience for this document is: agency and consulting engineers specialized in geotechnical and structural design of highway structures; engineering geologists and consulting engineers providing technical reviews, or who are engaged in the design, procurement, and construction of driven pile foundations. This document is also intended for management, specification and contracting specialists, as well as for construction engineers interested in design and contracting aspects of driven pile systems.

This document draws material from the three earlier FHWA publications in this field; FHWA-DP-66-1 by Vanikar (1985), FHWA HI 97-013 and FHWA HI 97-014 by Hannigan et al. (1998), and FHWA NHI-05-042 and FHWA NHI-05-043 by Hannigan et al. (2006). Photographs without specific acknowledgement in this two volume document are from these previous editions, their associated training courses, or from the consulting practice of GRL Engineers, Inc.

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LIST OF SYMBOLS

A	-	Pile cross sectional area.
A_d	-	Angular distortion.
A_p	-	Pile toe area.
A_s	-	Pile shaft surface area.
B	-	Width of pile group.
b	-	Pile width or diameter, Width/ Height of square.
b_f	-	Flange width of pile section.
bpm	-	Blow per minute.
C	-	Wave speed of pile material.
C'	-	Dimensionless bearing capacity index, determined from average corrected SPT N value, for layer with consideration of SPT hammer type.
C_a	-	Pile adhesion.
C_c	-	Compression index.
C_d	-	Pile perimeter at depth d.
C_F	-	Correction factor for K_δ when $\delta \neq \phi$.
C_n	-	Correction factor for SPT N value.
C_r	-	Recompression index.
C_s	-	Swell index.
c'	-	Effective cohesion.
D	-	Pile embedded length; Pile width for WEAP quake calculation.
D_B	-	Pile embedded length into bearing stratum.
D_{np}	-	Depth from reference to neutral plane.
d_w	-	Web depth of pile section.
E	-	Elastic modulus of pile material.
E_m	-	Rock Mass Modulus.
E_n	-	Rated hammer energy.
ER	-	SPT Hammer efficiency as determined by energy measurements in accordance with ASTM D4633.
E_s	-	Elastic modulus of soil.
e_o	-	Initial soil layer void ratio.
F_y	-	Yield stress of steel.
g	-	Acceleration due to gravity.
H	-	Soil layer thickness.

H_o	-	Initial soil layer thickness.
h	-	Ram stroke.
h_i	-	Thickness of soil strata.
h_w	-	Height of water (pressure head) for calculation of pore water pressure.
k_c	-	Modulus of subgrade reaction for cyclic lateral loading.
L	-	Total pile length.
N_t	-	Toe bearing capacity coefficient.
M_n	-	Nominal flexural resistance (structural).
M_p	-	Plastic moment about the weak axis.
M_r	-	Factored flexural resistance (structural).
M_{rx}	-	Factored flexural resistance about x-axis.
M_{ry}	-	Factored flexural resistance about y-axis.
M_u	-	Factored moment load (structural).
M_{ux}	-	Factored moment about x-axis.
M_{uy}	-	Factored moment about y-axis.
m_n	-	Dimensionless modulus number.
m_{nr}	-	Dimensionless recompression modulus number.
N_c	-	Dimensionless bearing capacity factor.
N_q	-	Dimensionless bearing capacity factor.
N_{60}	-	SPT N value corrected for 60% energy transfer.
$(N_1)_{60}$	-	SPT N value corrected for energy and overburden stress.
N_γ	-	Bearing capacity factor.
n	-	Number of piles in pile group.
P_m	-	P-multiplier for p-y curve.
P_n	-	Nominal structural resistance in axial compression.
P_r	-	Factored structural resistance in axial compression.
P_u	-	Factored axial load (structural).
P_{ui}	-	Maximum single pile axial load.
p_a	-	Atmospheric pressure.
p_f	-	Design foundation pressure.
Q	-	Factored Axial Load; Unfactored Axial Load.
Q_d	-	Dead or sustained load on a pile.
Q_{max}	-	Maximum axial compressive force in the pile.
q	-	Surcharge.
q_u	-	Unconfined compressive strength.
R_n	-	Nominal resistance.
R_{ndr}	-	Nominal driving resistance.
R_{ng}	-	Nominal resistance of pile group.
R_p	-	Nominal toe resistance.

R_r	-	Factored resistance.
R_{relax}	-	Resistance loss from relaxation.
R_{rg}	-	Factored resistance of the pile group.
R_s	-	Nominal shaft resistance.
R_{scour}	-	Resistance loss from scour.
R_{ug}	-	Nominal uplift resistance of the pile group.
S	-	Settlement.
S_d	-	Differential settlement of the foundation.
S_c	-	Settlement from primary consolidation.
S_h	-	Horizontal abutment movement.
s_u	-	Undrained shear strength.
t	-	Pipe pile wall thickness.
t_{cap}	-	Thickness of pile cap (8.9).
t_f	-	Flange thickness of pile section (8.2) (8.5).
t_{soil}	-	Thickness of compressible soil beneath neutral plane.
t_w	-	Web thickness of pile section.
V_n	-	Nominal shear resistance (structural).
V_r	-	Factored shear resistance (structural).
V_u	-	Factored shear load (structural).
W	-	Ram weight.
W_c	-	Estimated weight of pile cap.
W_g	-	Effective weight of the pile/soil block including pile cap weight.
W_s	-	Estimated weight of soil above pile cap.
w	-	Moisture content.
x	-	Distance along x-axis from the center of the column to each pile center.
Y_o	-	Pile head deflection.
y	-	Distance along y-axis from the center of the column to each pile center.
Z	-	Length of pile group .
Δ	-	Elastic deformation of pile.
$\Delta\varepsilon$	-	Change of strain.
$\Delta\sigma$	-	Change of stress.
ε	-	Strain.
ε_{50}	-	Strain at one half the maximum principal stress.
η_g	-	Pile group efficiency.
γ	-	Total unit weight of soil.
γ'	-	Buoyant unit weight of soil.
γ_d	-	Dead Load Factor.
γ_f	-	Unit weight of embankment fill.

- γ_i - Unit weight of soil strata for calculation of in-situ stress.
- γ_l - Load factor for force effect due to live loads.
- γ_p - Load factor for force effect due to permanent loads.
- γ_w - Unit weight of water.
- σ - Normal stress (pressure) on plane of failure, stress.
- σ' - Effective normal stress (pressure) on plane of failure ($\sigma - u$).
- σ'_d - Vertical effective stress at the center of depth increment d .
- σ_{dr} - Driving stress.
- σ'_o - Effective stress prior to stress increase.
- σ_p - Preconsolidation pressure or stress.
- σ_r - Reference stress for settlement with Janbu Tangent Modulus.
- σ'_v - Vertical effective stress.
- σ'_{vo} - Vertical effective stress at the sample depth.
- σ'_1 - Effective stress after stress increase.
- ϕ - Resistance factor, statistically based multiplier on nominal resistance.
- ϕ' - Effective Stress Friction Angle.
- ϕ_c - Resistance factor (pile structural resistance in compression).
- ϕ_{da} - Resistance factor (pile structural resistance during driving).
- ϕ_{dyn} - Resistance factor (based on the construction control method).
- ϕ_f - Resistance factor (pile structural resistance in flexure).
- ϕ_{stat} - Resistance factor (based on the static analysis method).
- ϕ_{ug} - Resistance factor for group uplift (based on the uplift analysis method).
- ϕ_{up} - Resistance factor (based on the uplift analysis method).
- ϕ_v - Resistance factor (pile structural resistance in shear).

LIST OF ACRONYMNS

AASHTO	-	American Association of State Highway and Transportation Officials
ASTM	-	American Society for Testing and Materials
BOR	-	Beginning of Restrike
CED	-	Closed End Diesel hammer
CEP	-	Closed End Pipe
CFA	-	Continuous Flight Auger
DA	-	Design Angular Distortion
DD	-	Downdrag
DF	-	Drag Force
DLT	-	Dynamic Load Test
EOD	-	End of Drive
ER	-	SPT hammer efficiency as determined by energy measurements
FHWA	-	Federal Highway Administration
I.D.	-	Inner diameter
NHI	-	National Highway Institute
O.D.	-	Outer Diameter
OEP	-	Open Ended Pipe
SA	-	Static Analysis
SPT	-	Standard Penetration Test
SLT	-	Static Load Test
WE	-	Wave Equation
WEAP	-	Wave Equation Analysis Program

APPENDIX D

COMPREHENSIVE DESIGN EXAMPLES

This appendix presents comprehensive design examples for a driven pile foundation project. The worked design examples supplement the material presented in publications FHWA-NHI-16-009 and FHWA-NHI-16-010, the primary FHWA guidance documents on driven pile foundations. The worked LRFD design examples address strength, service, and extreme limit state considerations for a bridge structure in highly variable subsurface conditions. Worked design examples in cohesionless, cohesive, and layered soil profiles are presented as well as pile design on hard rock. All limit states considerations are addressed as applicable in the worked design examples.

The bridge dimensions and superstructure loads were provided by a transportation agency, while the bridge structure is supported at two abutments and a pier. The soil profile for the worked design examples was developed to illustrate use of the manual's design methods and procedures in a variety of subsurface conditions. Each substructure location presents a different subsurface condition. A cohesionless soil profile is presented at the North Abutment. At the pier, worked examples in a layered subsurface profile are presented. At the South Abutment, worked examples for a cohesive soil profile underlain by a hard bedrock are presented. Strength, service, and extreme limit states are addressed at each substructure location as appropriate. An economic evaluation of candidate pile types is also included at each substructure location.

The worked design examples follow the step by step design and construction process outlined in Chapter 2 of FHWA-NHI-16-009. The design process flow chart introduced in Chapter 2 is followed in the worked design examples and is presented herein as Figure D-1.

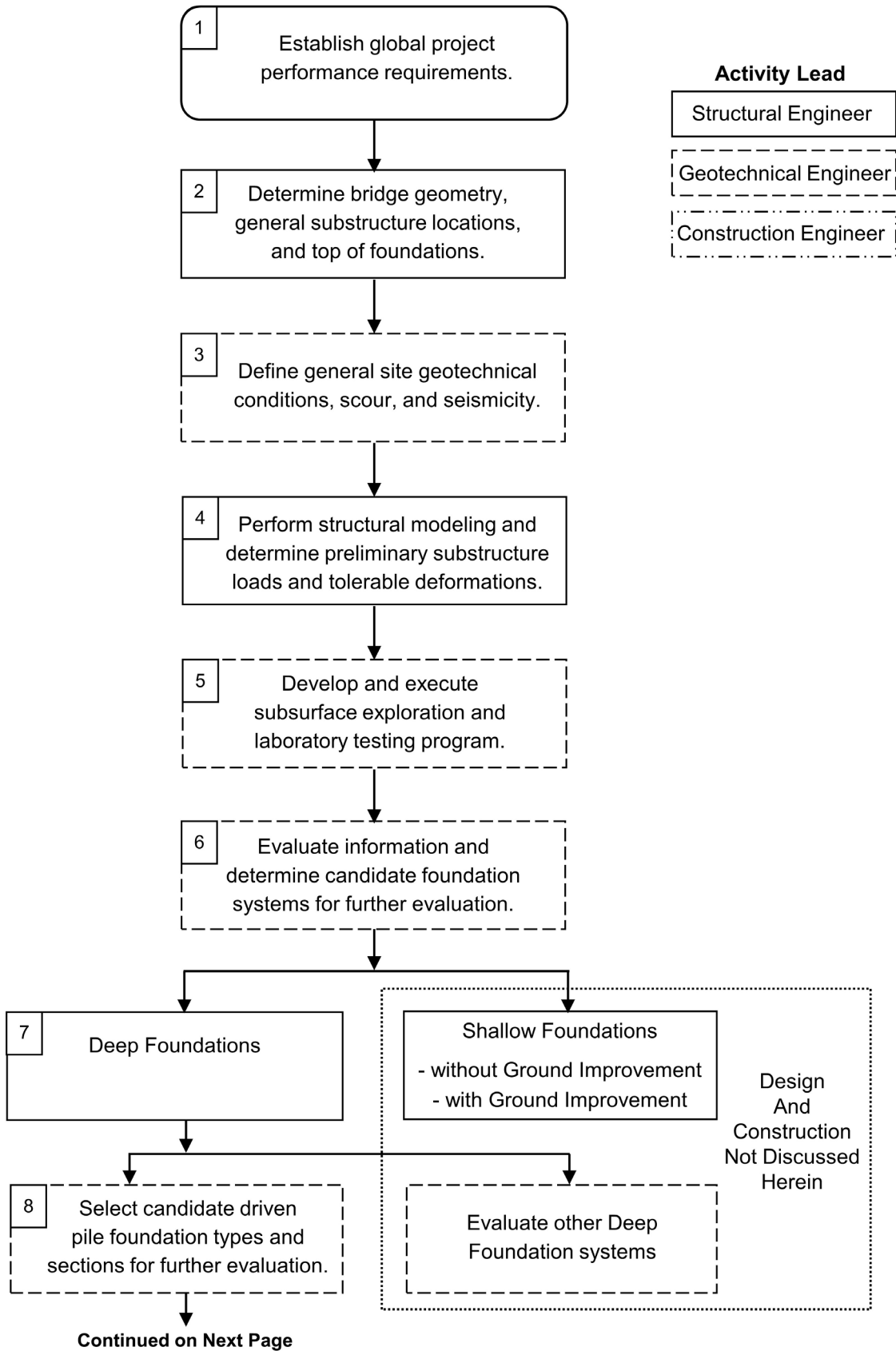


Figure D-1 Design process flow chart.

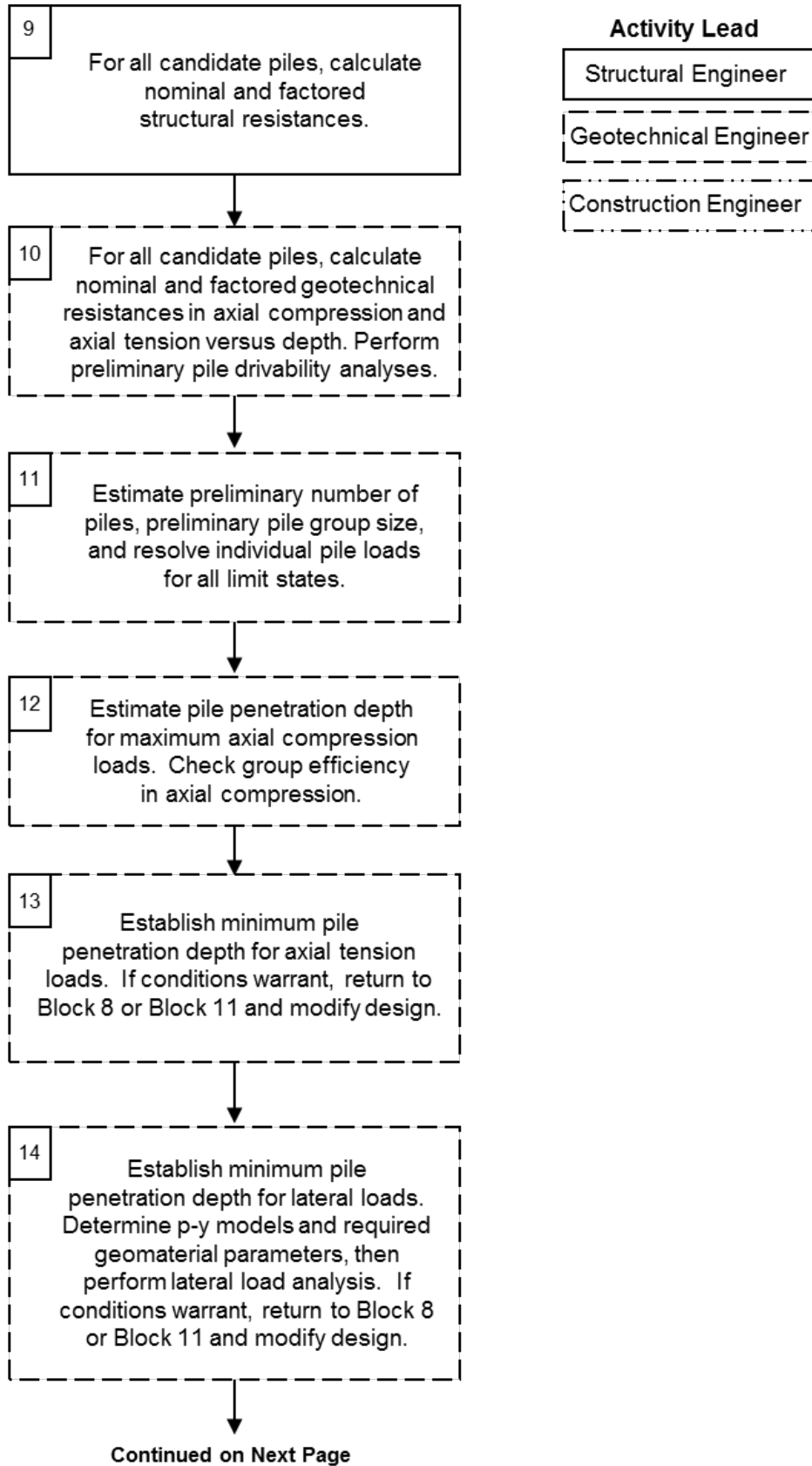


Figure D-1 Design process flow chart (continued).

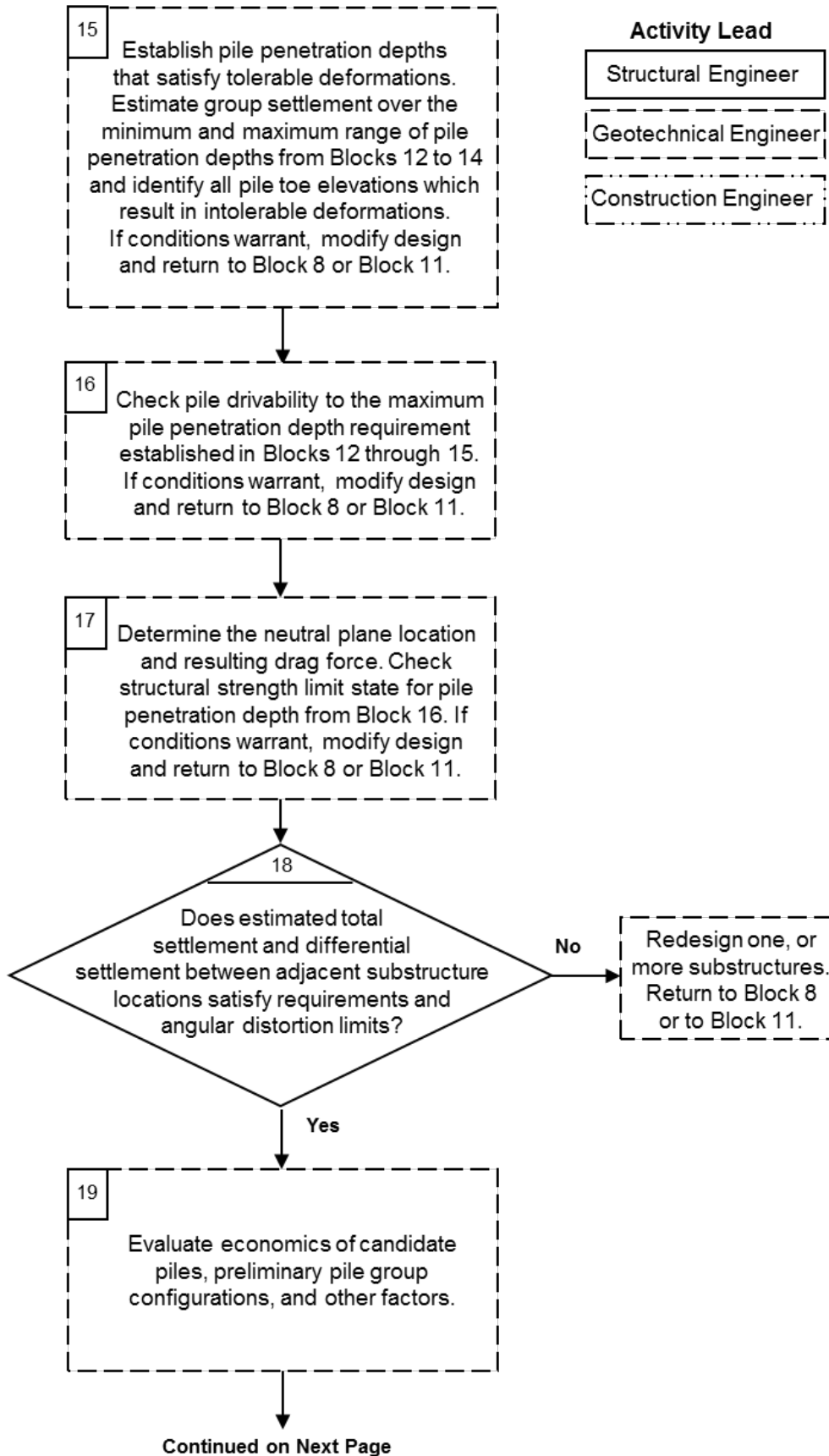
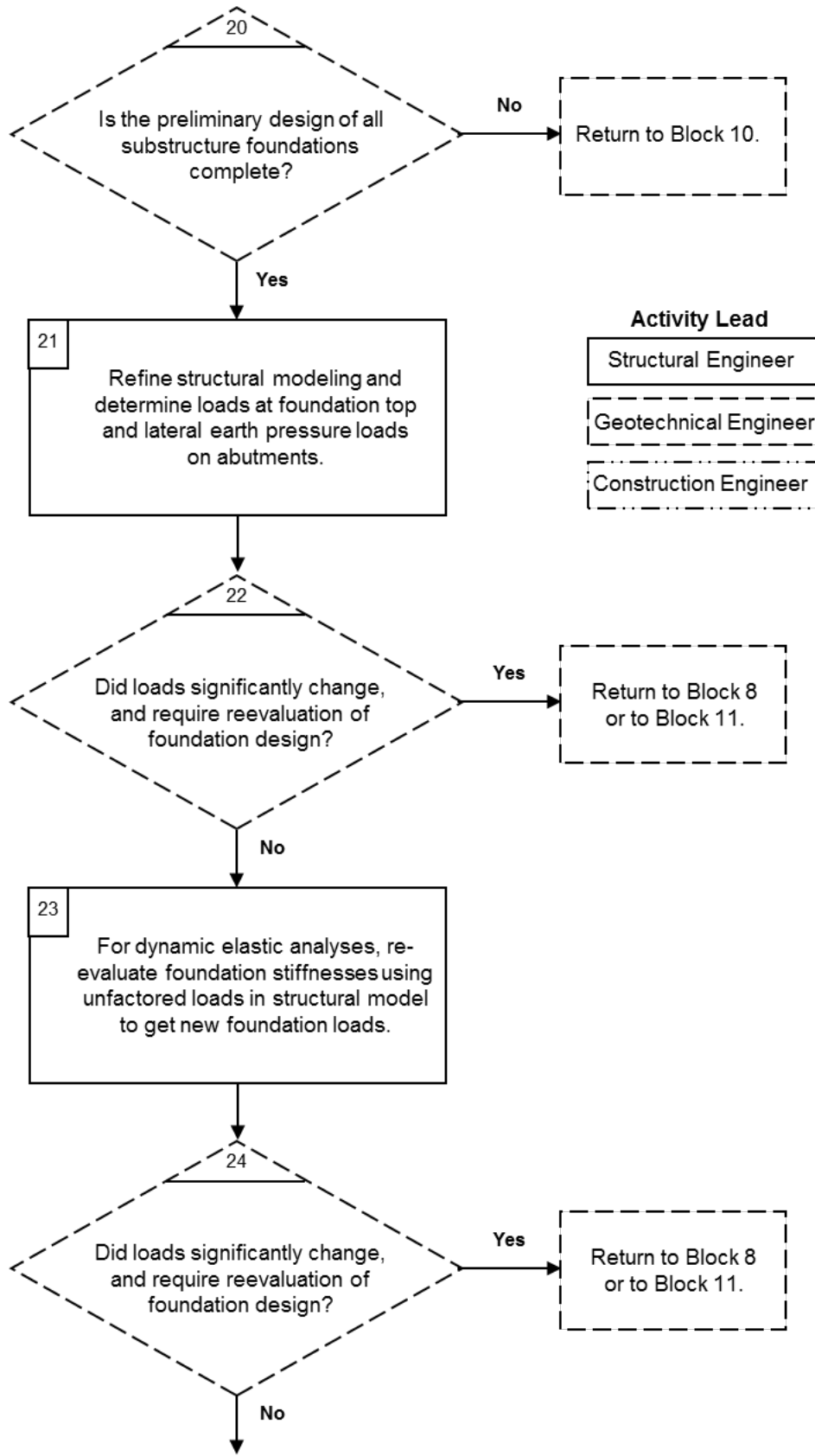


Figure D-1 Design process flow chart (continued).



- Activity Lead**
- Structural Engineer
 - Geotechnical Engineer
 - Construction Engineer

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Figure D-1 Design process flow chart (continued).

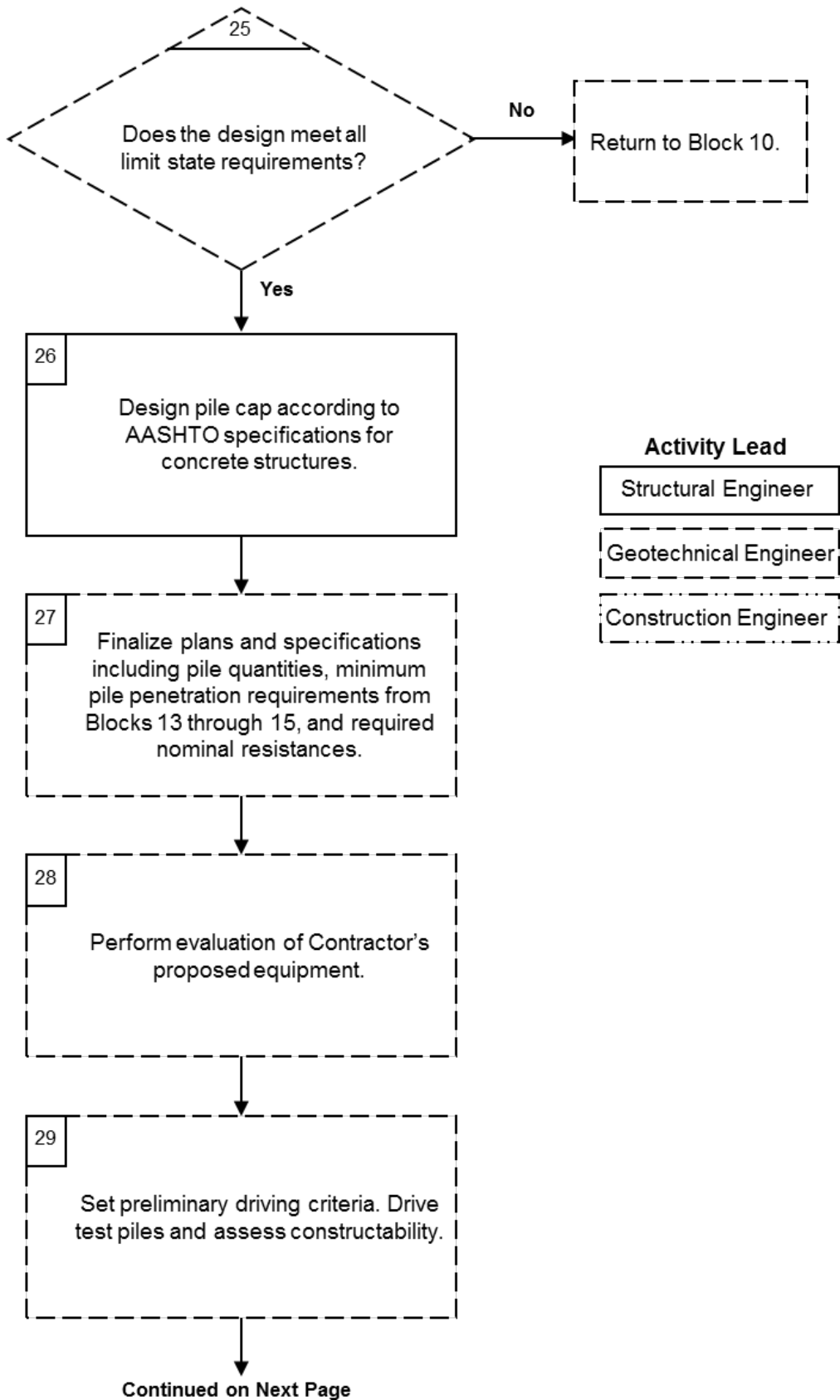


Figure D-1 Design process flow chart (continued).

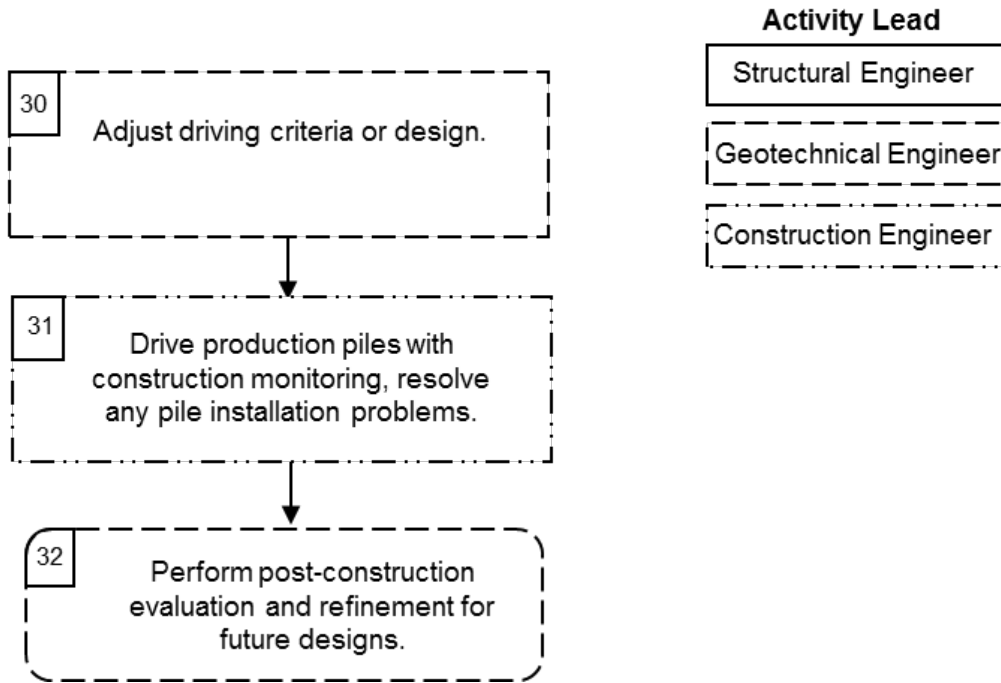


Figure D-1 Design process flow chart (continued).

D.1 Block 1: Establish Global Project Performance Requirements

The general structure requirements have been determined and are summarized below.

1. The project will consist of new twin river bridges, one for northbound traffic and one for southbound traffic. Both bridges will be designed for three travel lanes.
2. The project will be constructed at one time.
3. The general structure layout and approximate substructure locations are known but approach grades have yet to be established.
4. The foundation engineer has visited the site. During normal flow, the river is approximately 80 feet wide and 5 feet deep at the proposed bridge location. The north side of the river has a bank approximately 20 feet high consisting of silty sand. The ground surface on the south side of the river has a slightly more gradual slope with surficial soils consisting of silty clay. The new bridges will be approximately 200 feet long with a higher approach embankment required on the south side.5. Based on other bridge structures similar to those contemplated for this project, limit state axial compression loads are anticipated to be on the order of 2000 to 2500 kips at the abutments and 2500 to 3500 kips at the pier.

Based on safety, drainage, and rideability considerations, a maximum total settlement at any substructure location of 1.5 inches and maximum differential settlement between substructure locations of 1 inch are desired. In addition, the lateral deformation is limited to 1 inch.

6. The new bridge structure is not subject to vessel or vehicle impact loading or seismic activity. However, substantial scour is a design consideration.
7. Lateral squeeze may or may not be a design consideration depending upon the subsurface conditions encountered.
8. No modifications appear warranted in the preliminary design such as adjustments in substructure locations or span lengths.

9. No site or surrounding environmental considerations (low headroom, utility conflicts, aggressive soil environments, limitations on noise, vibrations, etc.) need to be considered in the design.
10. There are no special factors influencing bridge span length.

D.2 Block 2: Determine Structure (Bridge) Geometry, Substructure Locations and Elevations

The general bridge geometry, probable substructure locations, and the top of foundation elevations have been established. This information is presented in Table D-1.

Table D-1 General Bridge Geometry Summary

Substructure	Substructure Station	Top of Foundation Elevation (feet)
North Abutment	1223 + 26	+ 312.0
Pier 2	1224 + 26	+ 280.0
South Abutment	1225 + 26	+ 307.0

D.3 Block 3: Define General Site Geotechnical Conditions, Scour, and Seismicity

Available foundation plans and structure performance have been collected from other projects in the vicinity. The project is located in an area known for variable subsurface conditions. Post construction issues of unsatisfactory structure performance have not been reported in the vicinity. The hydraulic engineer has been consulted to determine probable scour depths that may impact the foundation selection. Seismicity is not a design consideration.

D.4 Block 4: Perform Preliminary Structure Modeling. Determine Preliminary Substructure Loads and Tolerable Deformations

Preliminary structural analysis and modeling of the proposed bridge structure has been performed at this time. The preliminary strength, service and extreme events limit state loads and performance requirements at the foundation top have also been established. The tolerable vertical and lateral deformations previously stated in Block 1 have been confirmed by the bridge office as performance requirements.

D.5 Block 5: Develop and Execute Subsurface Exploration and Laboratory Testing Program for Feasible Foundation Systems

Prior to this stage, only the general bridge geometry, preliminary superstructure limit state loads and general site geotechnical conditions were known. The substructure locations now have been determined. A subsurface exploration and laboratory testing program has been implemented with a single boring performed at each substructure location. The soil boring logs for the southbound bridge are presented in Figures D-2 through D-4.

At each respective sample depth, the boring logs indicate the SPT N value in cohesionless soil layers, or the undisturbed Shelby tube sampling interval in cohesive layers. The results from subsequent unconfined compression tests and other index tests performed on the undisturbed samples are also provided on the boring logs at the respective sample depth. The results of the subsurface exploration and laboratory testing program are used to prepare a subsurface profile; define soil and rock parameters including strength, compressibility, parameter variation, liquefaction susceptibility, and seismic earth pressure parameters; define subsurface water conditions, as well as identify critical cross sections for design.

The results of the subsurface exploration program and laboratory test results have been used to generate a generalized soil profile which is presented in Figure D-5. This generalized soil profile is an invaluable resource as the foundation selection process proceeds. A design profile for each substructure location will be developed from the information gathered in this block and used in later blocks.

SUBSURFACE EXPLORATION LOG				BORING NO. S-1			SHEET 1 OF 3				
District	2			Hammer Fall-Casing	--			Line	Baseline		
County	Sussex			Hammer Fall Sampler	--			Station	1223 + 26		
Project	Sunrise Expressway			Wt. of Hammer Casing	--			Offset	20 ft Lt		
Structure	Freedom Bridge			Wt. of Hammer Sampler	140 lb			Surface EL.	315.0 ft		
Date Start	5/28/2011			SPT Hammer Type	Automatic			Water Table EL.	300.0 ft		
Date Finish	5/28/2011			Core Barrel Type	Double Tube			Logged By	MLB		
Backfill/Sealed	5/28/2011			Drill Method	Mobile B57 & HSA			Time	9:30 PM		

I.D.	D E P T H ft	B L O W S /6"	N	Soil Description and Remarks	q _u tsf	Y _d pcf	W _n %	I.D.	D E P T H ft	B L O W S /6"	N	Soil Description and Remarks	q _u tsf	Y _d pcf	W _n %
SS-1	1	1	4	Loose Silty Fine Sand (SM)				SS-5	2	2	8	Loose Silty Fine Sand (SM)			
	1	3							3	5					
	3														
SS-2	5		4				SS-6	25		13	Medium Dense Coarse Sand, Little Silt (SP)				
	1	1						3	6						
	2	2						7							
SS-3	10		6				SS-7	30		15					
	2	2						4	7						
	2	4						8							
SS-4	15		6				SS-8	35		11					
	2	2						3	5						
	3	3						6							
	20							40							

Figure D-2 Boring Log S-1, Page 1 of 3.

SUBSURFACE EXPLORATION LOG				BORING NO. <u>S-1</u>		SHEET <u>2</u> OF <u>3</u>	
District	<u>2</u>	Hammer Fall-Casing	<u>--</u>	Line	<u>Baseline</u>		
County	<u>Sussex</u>	Hammer Fall Sampler	<u>--</u>	Station	<u>1223 + 26</u>		
Project	<u>Sunrise Expressway</u>	Wt. of Hammer Casing	<u>--</u>	Offset	<u>20 ft Lt</u>		
Structure	<u>Freedom Bridge</u>	Wt. of Hammer Sampler	<u>140 lb</u>	Surface EL.	<u>315.0 ft</u>		
Date Start	<u>5/28/2011</u>	SPT Hammer Type	<u>Automatic</u>	Water Table EL.	<u>300.0 ft</u>		
Date Finish	<u>5/28/2011</u>	Core Barrel Type	<u>Double Tube</u>	Logged By	<u>MLB</u>		
Backfill/Sealed	<u>5/28/2011</u>	Drill Method	<u>Mobile B57 & HSA</u>	Time	<u>9:30 PM</u>		

I.D.	D E P T H ft	B L O W S /6"	N	Soil Description and Remarks	q _u tsf	Y _d pcf	w _n %	I.D.	D E P T H ft	B L O W S /6"	N	Soil Description and Remarks	q _u tsf	Y _d pcf	w _n %
SS-9		6 7 8	15	Medium Dense Coarse Sand, Little Silt (SP)				SS-13		18 21 20	41	Dense Gravel with Sand (GW)			
SS-10		45 7 9 9	18					SS-14		65 20 22 21	43				
SS-11		50 18 21 19	40	Dense Gravel with Sand (GW)				SS-15		70 21 20 21	41	Note: Occasional Cobbles 69-70 ft			
SS-12		55 17 19 20	39	Note: Occasional Cobbles 54-55 ft				SS-16		75 20 22 22	44				
		60								80					

Figure D-2 Boring Log S-1, Page 2 of 3.

SUBSURFACE EXPLORATION LOG				BORING NO. <u>S-1</u>			SHEET <u>3</u> OF <u>3</u>								
District	<u>2</u>	Hammer Fall-Casing	<u>--</u>	Line	<u>Baseline</u>										
County	<u>Sussex</u>	Hammer Fall Sampler	<u>--</u>	Station	<u>1223 + 26</u>										
Project	<u>Sunrise Expressway</u>	Wt. of Hammer Casing	<u>--</u>	Offset	<u>20 ft Lt</u>										
Structure	<u>Freedom Bridge</u>	Wt. of Hammer Sampler	<u>140 lb</u>	Surface EL.	<u>315.0 ft</u>										
Date Start	<u>5/28/2011</u>	SPT Hammer Type	<u>Automatic</u>	Water Table EL.	<u>300.0 ft</u>										
Date Finish	<u>5/28/2011</u>	Core Barrel Type	<u>Double Tube</u>	Logged By	<u>MLB</u>										
Backfill/Sealed	<u>5/28/2011</u>	Drill Method	<u>Mobile B57 & HSA</u>	Time	<u>9:30 PM</u>										
I.D.	D E P T H	B L O W S	N	Soil Description and Remarks	q _u	γ _d	w _n	I.D.	D E P T H	B L O W S	N	Soil Description and Remarks	q _u	γ _d	w _n
	ft	/6"			tsf	pcf	%		ft	/6"			tsf	pcf	%
SS-17		22 22 23	45	Dense Gravel with Sand (GW)											
												EOB @ 102.0 ft			
SS-18		85 23 24 24	48							105					
SS-19		90 22 24 22	46							110					
SS-19		95 25 24 23	47							115					
RC-1				Limestone Bedrock Rock Core 1 97.0 - 102.0 ft REC = 83% RQD = 81%						120					

Figure D-2 Boring Log S-1, Page 3 of 3.

SUBSURFACE EXPLORATION LOG				BORING NO. <u>S-2</u>		SHEET <u>1</u> OF <u>2</u>	
District	<u>2</u>	Hammer Fall-Casing	<u>--</u>	Line	<u>Baseline</u>		
County	<u>Sussex</u>	Hammer Fall Sampler	<u>--</u>	Station	<u>1224 + 26</u>		
Project	<u>Sunrise Expressway</u>	Wt. of Hammer Casing	<u>--</u>	Offset	<u>20 ft Lt</u>		
Structure	<u>Freedom Bridge</u>	Wt. of Hammer Sampler	<u>140 lb</u>	Surface EL.	<u>285.0 ft</u>		
Date Start	<u>6/2/2011</u>	SPT Hammer Type	<u>Automatic</u>	Water Table EL.	<u>290.0 ft</u>		
Date Finish	<u>6/3/2011</u>	Core Barrel Type	<u>Double Tube</u>	Logged By	<u>MLB</u>		
Backfill/Sealed	<u>6/3/2011</u>	Drill Method	<u>Mobile B57 & HSA</u>	Time	<u>5:30 PM</u>		

I.D.	D E P T H ft	B L O W S /6"	N	Soil Description and Remarks	q _u tsf	Y _d pcf	W _n %	I.D.	D E P T H ft	B L O W S /6"	N	Soil Description and Remarks	q _u tsf	Y _d pcf	W _n %
SS-1	1 1 2	3		Loose Silty Fine Sand (SM)				UD-1				Very Stiff Silty Clay (CL)	3.10	98	31
SS-2	5 2 2 2	4						UD-2	25				3.21	104	23
SS-3	21 25 26	51		Extremely Dense Gravel (GP)											
SS-4	10 26 27 25	52						UD-3	30				3.25	105	22
SS-5	15 26 28 27	55						UD-4	35				3.30	103	24
	20								40						

Figure D-3 Boring Log S-2, Page 1 of 2.

SUBSURFACE EXPLORATION LOG				BORING NO. S-2			SHEET 2 OF 2				
District	2			Hammer Fall-Casing	--			Line	Baseline		
County	Sussex			Hammer Fall Sampler	--			Station	1224 + 26		
Project	Sunrise Expressway			Wt. of Hammer Casing	--			Offset	20 ft Lt		
Structure	Freedom Bridge			Wt. of Hammer Sampler	140 lb			Surface EL.	285.0 ft		
Date Start	6/2/2011			SPT Hammer Type	Automatic			Water Table EL.	290.0 ft		
Date Finish	6/3/2011			Core Barrel Type	Double Tube			Logged By	MLB		
Backfill/Sealed	6/3/2011			Drill Method	Mobile B57 & HSA			Time	5:30 PM		

I.D.	D E P T H ft	B L O W S /6"	N	Soil Description and Remarks	q _u tsf	Y _d pcf	W _n %	I.D.	D E P T H ft	B L O W S /6"	N	Soil Description and Remarks	q _u tsf	Y _d pcf	W _n %
UD-5				Very Stiff Silty Clay (CL)	3.34	105	22	SS-8	19	19	41	Dense Gravel with Sand (GP)			
UD-6	45				3.40	106	21	SS-9	65	21	50				
SS-6	50	18	38	Dense Gravel with Sand (GP) Note: Occasional Cobbles 52-53 ft				RC-1	70			Limestone Bedrock Rock Core 1 70.0 - 75.0 ft REC = 90% RQD = 79%			
		19													
SS-7	55	21	46						75			EOB @ 75.0 ft			
		22													
	60	24													
									80						

Figure D-3 Boring Log S-2, Page 2 of 2.

SUBSURFACE EXPLORATION LOG				BORING NO. S-3		SHEET 1 OF 3	
District	2		Hammer Fall-Casing	--		Line	Baseline
County	Sussex		Hammer Fall Sampler	--		Station	1225 + 26
Project	Sunrise Expressway		Wt. of Hammer Casing	--		Offset	20 ft Lt
Structure	Freedom Bridge		Wt. of Hammer Sampler	140 lb		Surface EL.	310.0 ft
Date Start	6/4/2011		SPT Hammer Type	Automatic		Water Table EL.	305.0 ft
Date Finish	6/5/2011		Core Barrel Type	Double Tube		Logged By	MLB
Backfill/Sealed	6/5/2011		Drill Method	Mobile B57 & HSA		Time	5:30 PM

I.D.	D E P T H ft	B L O W S /6"	N	Soil Description and Remarks	q _u tsf	Y _d pcf	W _n %	I.D.	D E P T H ft	B L O W S /6"	N	Soil Description and Remarks	q _u tsf	Y _d pcf	W _n %	
UD-1				Medium Silty Clay (CL)	0.65	86	26	UD-5				Medium Silty Clay (CL)	0.72	85	31	
														Stiff Silty Clay (CL)		
UD-2	5					0.66	85	29	UD-6	25					1.79	95
UD-3	10					0.68	85	30	UD-7	30				1.85	97	27
UD-4	15				0.70	84	32	UD-8	35				1.93	100	24	
	20								40							

Figure D-4 Boring Log S-3, Page 1 of 3.

SUBSURFACE EXPLORATION LOG				BORING NO. S-3			SHEET 2 OF 3				
District	2			Hammer Fall-Casing	--			Line	Baseline		
County	Sussex			Hammer Fall Sampler	--			Station	1225 + 26		
Project	Sunrise Expressway			Wt. of Hammer Casing	--			Offset	20 ft Lt		
Structure	Freedom Bridge			Wt. of Hammer Sampler	140 lb			Surface EL.	310.0 ft		
Date Start	6/4/2011			SPT Hammer Type	Automatic			Water Table EL.	305.0 ft		
Date Finish	6/5/2011			Core Barrel Type	Double Tube			Logged By	MLB		
Backfill/Sealed	6/5/2011			Drill Method	Mobile B57 & HSA			Time	5:30 PM		

I.D.	D E P T H ft	B L O W S /6"	N	Soil Description and Remarks	q _u tsf	Y _d pcf	W _n %	I.D.	D E P T H ft	B L O W S /6"	N	Soil Description and Remarks	q _u tsf	Y _d pcf	W _n %
UD-9				Stiff Silty Clay (CL)	2.00	99	26	UD-13				Very Stiff Silty Clay (CL)	3.36	105	23
	45								65						
UD-10				Very Stiff Silty Clay (CL)	3.11	104	24	UD-14					3.39	106	21
	50								70				3.50	107	21
UD-11					3.19	103	25	UD-15							
	55								75				3.55	108	20
UD-12					3.30	106	22	UD-16							
	60								80						

Figure D-4 Boring Log S-3, Page 2 of 3.

SUBSURFACE EXPLORATION LOG				BORING NO. S-3			SHEET 3 OF 3													
District	2			Hammer Fall-Casing	--		Line	Baseline												
County	Sussex			Hammer Fall Sampler	--		Station	1225 + 26												
Project	Sunrise Expressway			Wt. of Hammer Casing	--		Offset	20 ft Lt												
Structure	Freedom Bridge			Wt. of Hammer Sampler	140 lb		Surface EL.	310.0 ft												
Date Start	6/4/2011			SPT Hammer Type	Automatic		Water Table EL.	305.0 ft												
Date Finish	6/5/2011			Core Barrel Type	Double Tube		Logged By	MLB												
Backfill/Sealed	6/5/2011			Drill Method	Mobile B57 & HSA		Time	5:30 PM												
I.D.	D E P T H ft	B L O W S /6"	N	Soil Description and Remarks	q _u tsf	Y _d pcf	W _n %	I.D.	D E P T H ft	B L O W S /6"	N	Soil Description and Remarks	q _u tsf	Y _d pcf	W _n %					
UD-17				Very Stiff Silty Clay (CL)	3.58	109	19					EOB @101.0 ft								
UD-18	85												105							
UD-19	90				3.60	108	20													
UD-20	95				3.65	110	19	110												
SS-1		RC-1		Limestone Bedrock Rock Core 1 96.0 - 101.0 ft REC = 93% RQD =88%	3.70	110	18	115												
	100											120								

Figure D-4 Boring Log S-3, Page 3 of 3.

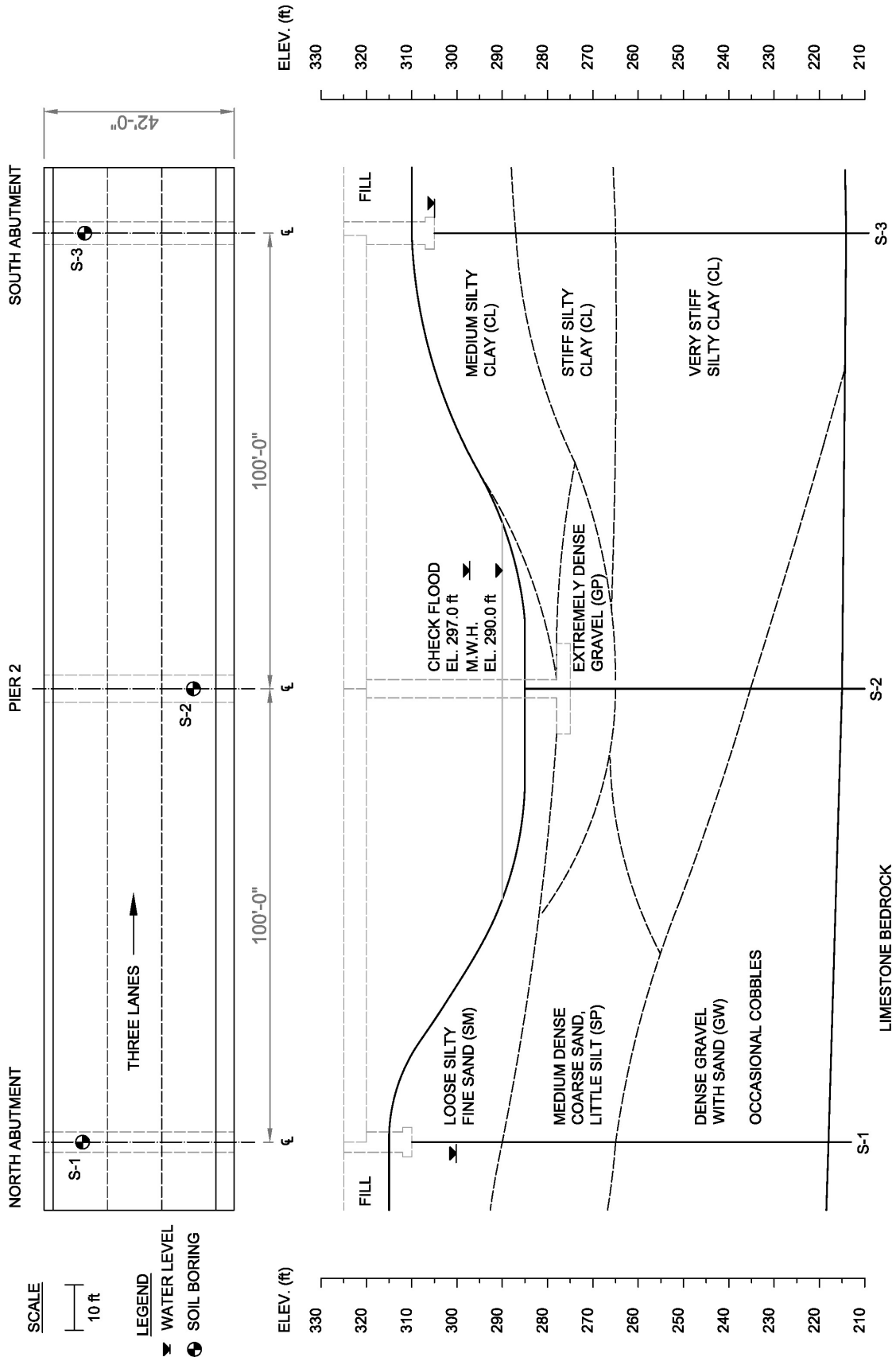


Figure D-5 Site Profile View.
19

D.6 Block 6: Evaluate Information and Determine Candidate Foundation Systems

The question to be answered is what candidate foundation systems are appropriate for consideration based on the site conditions. This question will be answered based primarily on the strength and compressibility of the geomaterials, the proposed loading conditions, the project deformation limits, the project schedule, and the foundation cost. The hydraulic analysis indicates the potential for substantial scour at the center pier. The selection of the appropriate foundation system including cost considerations is discussed in greater detail in Chapter 3.

D.7 Block 7: Determine if a Deep Foundation is Required

Based on the strength and compressibility of the near surface geomaterials, scour concerns, as well as the required construction timeline, it is decided that a deep foundation system is required for structure support. Driven piles and other deep foundation systems must now be evaluated. Through this process, some foundation systems become less viable as constructability and cost considerations are refined. In the final analysis, the design, construction, and testing costs associated with addressing the variable subsurface conditions at this site are prohibitive for all of the drilled deep foundation solutions. Accordingly, driven piles are selected for the structure support; and drilled shaft, micropile, CFA pile, and other drilled deep foundation systems are eliminated.

D.8 Block 8: Select Candidate Driven Pile Types and Sections

Driven pile foundation systems consisting of steel H-piles, closed-end steel pipe piles, and prestressed concrete piles are initially considered technically and economically feasible. However, due to pile drivability concerns presented by the extremely dense gravel layer at Pier 2, closed-end steel pipe piles and prestressed concrete piles are eliminated from further consideration. In this geographic region, steel H-piles are commonly used. Local contractors are familiar with H-pile installation and splicing, and H-piles are readily available from a local manufacturing facility. For these reasons, in addition to the ease with which H-piles can accommodate length variations associated with site variability, H-piles are viewed as the primary candidate pile type. Several H-pile sections are viable, and those sections will be advanced as candidate driven pile foundation systems.

Foundation loads are now closer to being finalized as are the tolerable deformation limits for the structure. At this stage in the design process, detailed analyses are required at each substructure location. Therefore, Block 9 through Block 19 will be repeated for each of the three substructure locations: North Abutment, Pier 2, and South Abutment. The design process will be presented for these blocks at a given substructure location before repeating the same steps at the next substructure location.

D.9 Block 9: North Abutment – Calculate Nominal and Factored Structural Resistances for all Candidate Piles

The nominal structural resistance is now evaluated for five candidate H-pile sections. The H-pile sections selected for evaluation include a HP 10x42, a HP 12x53, a HP 12x74, a HP 14x89, and a HP 14x117. A detailed step by step example for calculation of the nominal structural resistance was previously presented in Section 8.5.3 for an HP 14x117 H-pile section. Therefore, this process is not repeated for the five candidate pile sections. Table D-2 presents the calculated nominal structural resistances in axial compression, flexure, and shear for the five candidate sections. An unbraced length of 1 foot was assumed in these calculations.

Table D-2 Nominal Structural Resistances in Axial Compression, Flexure and Shear

H-pile Section	HP 10x42	HP 12x53	HP 12x74	HP 14x89	HP 14x117
P_n , Nominal Resistance in Axial Compression (kips)	618	767	1088	1303	1718
M_{ny} , Nominal Resistance in Weak Axis Flexure (kip-ft)	82	114	118	257	380
M_{nx} , Nominal Resistance in Strong Axis Flexure (kip-ft)	176	295	433	592	807
V_n , Nominal Resistance in Shear (kips)	118	149	214	246	331

It is anticipated that the piles at the North Abutment will be driven into the dense gravel with sand deposit, or possibly to the underlying bedrock. The dense gravel with sand deposit contains occasional cobbles. In these conditions, pile shoes are recommended for use with the H-piles to reduce the risk of damage. Therefore, the applicable structural resistance factors, ϕ_c , are 0.5 for axial compression resistance and 0.7 for combined axial compression and flexural resistance. A resistance factor

of $\phi_v = 1.0$ is used for shear, and a resistance factor of $\phi_f = 1.0$ is applicable for flexure only. Table D-3 summarizes the calculated factored structural resistances in axial compression, combined axial compression and flexure, flexure, and shear.

Table D-3 Factored Structural Resistance in Axial Compression, Flexure and Shear

H-pile Section	HP 10x42	HP 12x53	HP 12x74	HP 14x89	HP 14x117
P_r , Factored Resistance in Axial Compression, $\phi_c = 0.5$ (kips)	309	383	544	652	859
P_r , Factored Resistance in Axial Compression and Flexure, $\phi_c = 0.7$ (kips)	433	537	762	912	1203
M_{ry} , Factored Resistance in Weak Axis Flexure, $\phi_f = 1.0$ (kip-ft)	82	114	118	257	380
M_{rx} , Factored Resistance in Strong Axis Flexure, $\phi_f = 1.0$ (kip-ft)	176	295	433	592	807
V_r , Factored Resistance in Shear $\phi_v = 1.0$ (kips)	118	149	214	246	331

D.10 Block 10: North Abutment – Calculate Nominal and Factored Geotechnical Resistances in Axial Compression and Tension Versus Depth for all Candidate Piles; Perform Preliminary Pile Drivability Analyses

The engineering properties of the subsurface materials at the North Abutment were determined in Block 5. The results of the boring program and laboratory tests are now used to develop a design profile for each substructure location. Engineering judgement was used in developing the design profile to delineate the subsurface conditions into layers with similar properties. An effective stress diagram, depicted in Figure D-6, was also constructed in association with the design profile. This diagram includes the total stress, porewater pressure, and effective stress versus depth. Figure D-6 also presents the basic soil profile for quick reference to the relevant soil layers. The effective stress diagram was computed using Equations 5-7 through 5-9 from Chapter 5. These equations are repeated below. For Figure D-6, the total stress, porewater pressure, and effective stress were calculated at 1 foot increments using a spreadsheet.

$$\sigma_{vo} = \sum_i^n (\gamma_i h_i) \quad [\text{Eq. 5-7}]$$

$$u = \gamma_w h_w \quad [\text{Eq. 5-8}]$$

$$\sigma'_{vo} = \sum_i^n (\gamma_i h_i) - \gamma_w h_w \quad [\text{Eq. 5-9}]$$

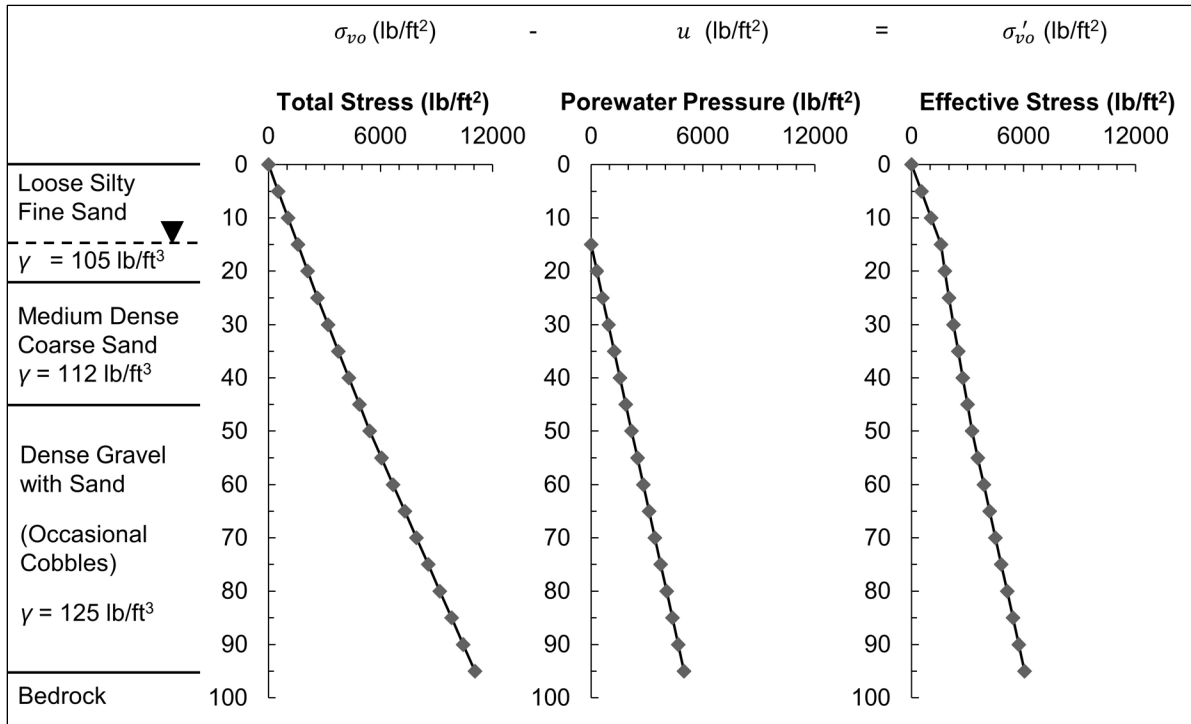


Figure D-6 Effective stress diagram for North Abutment using Boring S-1.

To continue development of an idealized design profile, field SPT N values were corrected for hammer energy transfer and vertical effective stress. The field SPT N values were first corrected for energy transfer using Equation 5-1. These results are presented in Table D-4. Typical correlations for SPT hammer type and energy transfer are provided in Section 5.1.1 (e.g., typical energy transfer of 80% for automatic hammer). The SPT hammer on the drill rig used for this project’s subsurface exploration program was calibrated in accordance with ASTM D4633. Results of this calibration indicated an average energy transfer of 75%. Therefore, the *ER* value in Equation 5-1 is equal to 75.

$$N_{60} = N \left(\frac{ER}{60} \right) \quad [\text{Eq. 5-1}]$$

The SPT N value was corrected for vertical effective stress as presented in Equation 5-2, using the Peck et al. (1974) correction factor, C_n . The depth value for this calculation was taken from the middle depth of the SPT sampling event after the 6

inch seating interval (e.g., the SPT N value recorded from 0.5-1.5 feet was corrected using the vertical effective stress at a depth of 1 foot).

$$(N_1)_{60} = C_n N_{60} \quad [\text{Eq. 5-2}]$$

$$C_n = 0.77 \log \left(\frac{20}{\sigma'_{vo}} \right) \quad [\text{Eq. 5-3}]$$

Table D-4 Correction of Field SPT N Value for Energy and Vertical Effective Stress at the North Abutment using S-1

Soil Layer	Depth (ft)	σ'_{vo} (ksf)	Field N value	N_{60}	C_n	$(N_1)_{60}$
1	1	0.105	4	5	1.99	10
1	6	0.630	4	5	1.39	7
1	11	1.155	6	8	1.19	9
1	16	1.618	6	8	1.07	8
1	21	1.831	8	10	1.03	10
2	26	2.058	13	16	0.99	16
2	31	2.306	15	19	0.95	18
2	36	2.554	11	14	0.92	13
2	41	2.802	15	19	0.89	17
2	46	3.050	18	23	0.86	19
3	51	3.363	40	50	0.83	41
3	56	3.676	39	49	0.80	39
3	61	3.989	41	51	0.77	40
3	66	4.302	43	54	0.75	40
3	71	4.615	41	51	0.72	37
3	76	4.928	44	55	0.70	39
3	81	5.241	45	56	0.68	38
3	86	5.554	48	60	0.66	40
3	91	5.867	46	58	0.64	37
3	96	6.180	47	59	0.62	37

After correcting the SPT N values for energy transfer and vertical effective stress, an average corrected N value was determined for each respective layer. The N value from the first SPT sample in Layer 1 was not included in this procedure since the bottom of the footing at the North Abutment is below this sample depth.

Table D-5 presents the average $(N_1)_{60}$ value for each layer at the North Abutment, the coefficient of variation, COV, within the layer, and the effective stress friction angle, ϕ' , chosen for the layer. The coefficient of variation for each layer was less than 25% indicating low variability within each of the identified layers. Therefore, separating the soil profile into additional layers was not performed, as it would have been with the existence of higher variability soil.

The $(N_1)_{60}$ values were used to estimate the effective stress friction angle of each soil layer in accordance with Table 5-5. Layer 3 consists of hard angular gravel with sand. Therefore, as discussed in Section 5.5.1 of Chapter 5, the design friction angle in this layer was limited to 36 degrees for estimating the shaft resistance while a friction angle of 40 degrees was used to estimate the toe resistance.

Table D-5 Effective Stress Friction Angle Correlations at the North Abutment

Soil Layer	Average $(N_1)_{60}$	COV	ϕ' shaft (degrees)	ϕ' toe (degrees)
1	9	14.4%	33	33
2	17	13.5%	36	36
3	39	3.7%	36	40

The design profile for the North Abutment is presented in Figure D-7, with the bottom of footing elevation 5 feet below ground surface elevation. Some soil properties in the design soil profile such as the elastic moduli, E_s , and the initial cyclic modulus of subgrade reaction, k_c , have been selected based on published correlations in the absence of laboratory and field testing. For this particular soil profile, the elastic moduli were determined from the SPT correlation in Table 5-11 and the initial cyclic moduli of subgrade reaction were selected based on representative values shown in Table 7-22. The figure format used to summarize the design profile is used at all substructure locations and therefore includes design value placeholders for cohesive soil parameters, the undrained shear strength, s_u , and 50% strain factor, ϵ_{50} . However, cohesive soils are not present at the North Abutment.

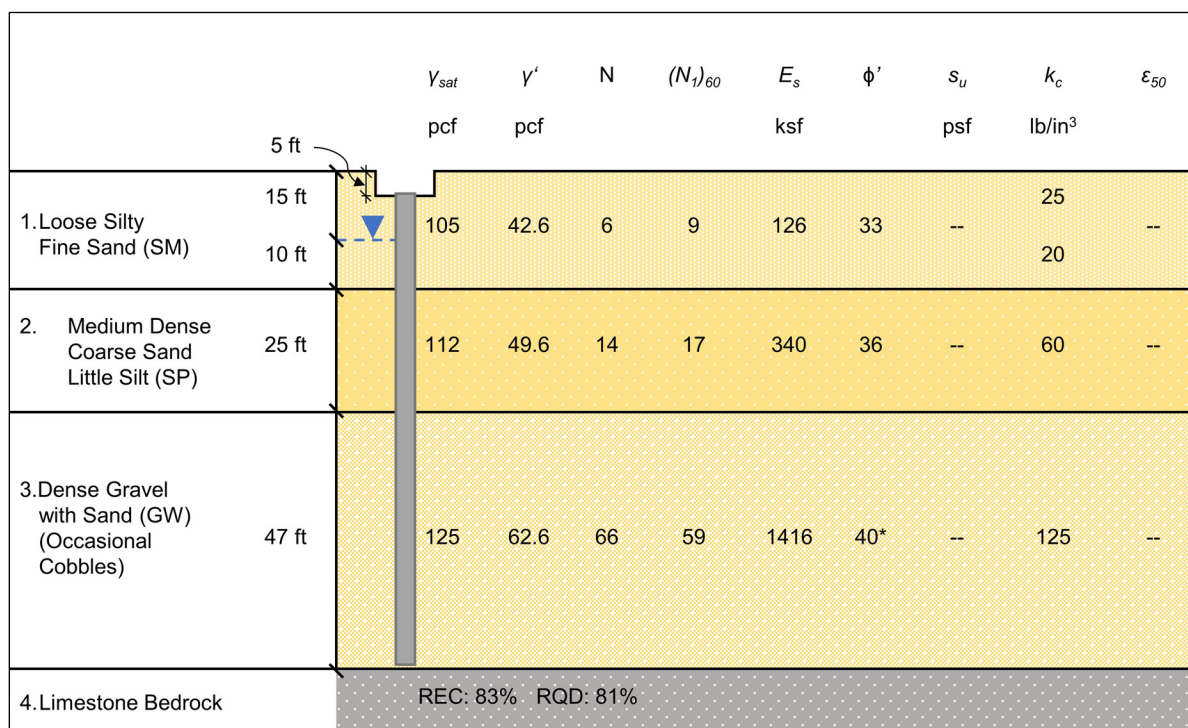


Figure D-7 Design profile at the North Abutment.

D.10.1 Geotechnical Resistance in Axial Compression

The nominal geotechnical resistance in axial compression is now determined for the selected candidate H-pile sections. The nominal geotechnical resistance can be calculated by hand or with computer software using an appropriate analysis method for cohesionless soils and the pile type. Chapter 7 describes appropriate methods for this purpose as well as computer programs available at the time of this manual's publication.

For this abutment, the DrivenPiles computer program was used to calculate the nominal resistance, shaft resistance, and toe resistance each as a function of depth for each of the five candidate H-piles. This software program was selected since it uses the FHWA recommended Nordlund method. This method is appropriate for calculating the nominal geotechnical resistance of H-piles in cohesionless soils such as those encountered in Boring S-1. The DrivenPiles program code also allows the analyst to select a different friction angle for shaft and toe resistance calculations in any layer. This option was utilized for the gravel with sand and occasional cobbles comprising Layer 3.

A summary of the shaft, toe, and nominal geotechnical resistance determined by the Nordlund static analysis method is presented for one of the candidate pile sections

in Table D-6. These results are also presented graphically in Figure D-8. The depth indicated in both Table D-6 and Figure D-8 is referenced from the bottom of footing, which is 5 feet below the original ground surface elevation. The nominal shaft, nominal toe, and nominal geotechnical resistances were calculated for all of the candidate pile sections in a similar manner. The nominal geotechnical resistances in axial compression versus pile penetration depth are presented in Figure D-9 for all candidate pile types.

Once the nominal geotechnical resistance in axial compression versus pile penetration depth has been calculated, the factored geotechnical resistance in axial compression versus pile penetration depth can be determined. The factored geotechnical resistance depends on the resistance determination method selected for the design.

If pile installation will be controlled by driving to a depth determined by static analysis method, then the factored geotechnical resistance in axial compression as a function of pile penetration depth can be determined by multiplying the calculated nominal resistance at a given depth by the resistance factor associated with the static analysis method, ϕ_{stat} . AASHTO (2014) resistance factors for static analysis methods are presented in Table 7-1 of this manual.

If the nominal resistance will be confirmed by a field determination method, the factored geotechnical resistance in axial compression as a function of pile penetration depth can be determined by multiplying the calculated nominal geotechnical resistance at a given depth by the resistance factor associated with the field determination method, ϕ_{dyn} . AASHTO (2014) resistance factors for field resistance determination methods are presented in Table 7-2.

Table D-6 Nominal Shaft, Nominal Toe and Nominal Geotechnical Resistance for
HP 12x74 at the North Abutment

Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Geotechnical Resistance (kips)	Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Geotechnical Resistance (kips)
0.01	0.0	16.4	16.4	37	151.0	155.4	306.4
1	1.1	19.7	20.7	38	157.9	155.4	313.3
2	2.3	23.0	25.3	39	165.0	155.4	320.4
3	3.7	26.2	30.0	40	172.1	155.4	327.5
4	5.3	29.5	34.9	41	179.4	155.4	334.8
5	7.2	32.8	40.0	42	186.8	155.4	342.2
6	9.2	36.1	45.2	43	194.3	155.4	349.7
7	11.4	39.4	50.7	44	202.0	155.4	357.4
8	13.7	42.6	56.4	44.99	209.7	155.4	365.1
9	16.3	45.9	62.2	45.01	209.8	398.8	608.6
9.99	19.1	49.2	68.2	46	217.6	406.4	624.0
10.01	19.1	49.2	68.3	47	225.7	414.1	639.8
11	22.0	50.5	72.5	48	233.9	421.8	655.7
12	25.0	51.3	76.2	49	242.2	428.1	670.3
13	28.0	51.3	79.3	50	250.7	428.1	678.8
14	31.2	51.3	82.4	51	259.4	428.1	687.5
15	34.4	51.3	85.6	52	268.2	428.1	696.3
16	37.7	51.3	88.9	53	277.2	428.1	705.3
17	41.0	51.3	92.3	54	286.3	428.1	714.4
18	44.5	51.3	95.7	55	295.5	428.1	723.6
19	48.0	51.3	99.2	56	304.9	428.1	733.0
19.99	51.5	51.3	102.8	57	314.5	428.1	742.6
20.01	51.6	110.2	161.8	58	324.2	428.1	752.3
21	56.5	112.8	169.3	59	334.1	428.1	762.2
22	61.5	115.5	176.9	60	344.1	428.1	772.2
23	66.6	118.1	184.7	61	354.2	428.1	782.3
24	71.9	120.8	192.7	62	364.6	428.1	792.7
25	77.2	123.5	200.7	63	375.0	428.1	803.1
26	82.7	126.2	208.9	64	385.7	428.1	813.7
27	88.3	128.8	217.2	65	396.4	428.1	824.5
28	94.0	131.5	225.6	66	407.3	428.1	835.4
29	99.9	134.2	234.1	67	418.4	428.1	846.5
30	105.9	136.9	242.7	68	429.6	428.1	857.7
31	111.9	139.5	251.5	69	441.0	428.1	869.1
32	118.2	142.2	260.4	70	452.5	428.1	880.6
33	124.5	144.9	269.4	71	464.2	428.1	892.3
34	130.9	147.6	278.5	72	476.0	428.1	904.1
35	137.5	150.2	287.7	73	488.0	428.1	916.1
36	144.2	152.9	297.1	74	500.1	428.1	928.2

Table D-6 Nominal Shaft, Nominal Toe and Nominal Geotechnical Resistance for HP 12x74 at the North Abutment (continued)

Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Geotechnical Resistance (kips)	Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Geotechnical Resistance (kips)
75	512.4	428.1	940.5	84	629.8	428.1	1057.8
76	524.9	428.1	953.0	85	643.5	428.1	1071.6
77	537.4	428.1	965.5	86	657.48	428.1	1085.58
78	550.2	428.1	978.3	87	671.57	428.1	1099.67
79	563.05	428.1	991.15	88	685.82	428.1	1113.92
80	576.09	428.1	1,004.19	89	700.21	428.10	1,128.31
81	589.28	428.1	1,017.38	90	714.76	428.10	1,142.86
82	602.6	428.1	1030.7	91	729.46	428.10	1,157.56
83	616.1	428.1	1044.2	91.99	744.16	428.10	1,172.26

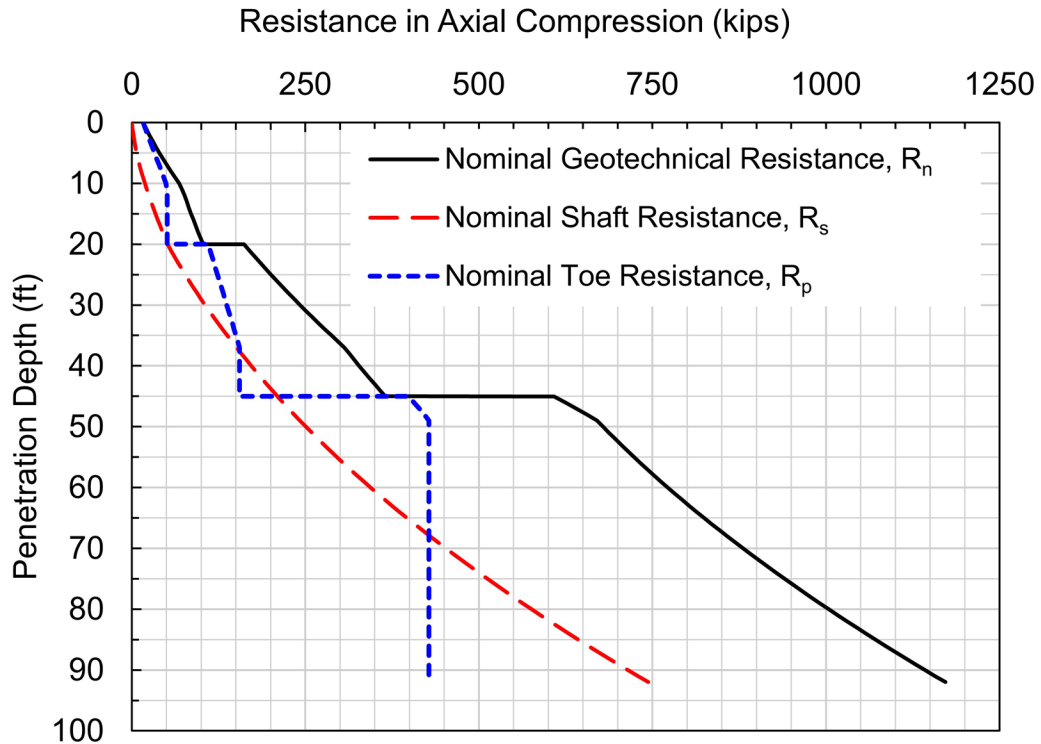


Figure D-8 Geotechnical resistance in axial compression versus pile penetration depth for HP 12x74 candidate pile section at North Abutment.

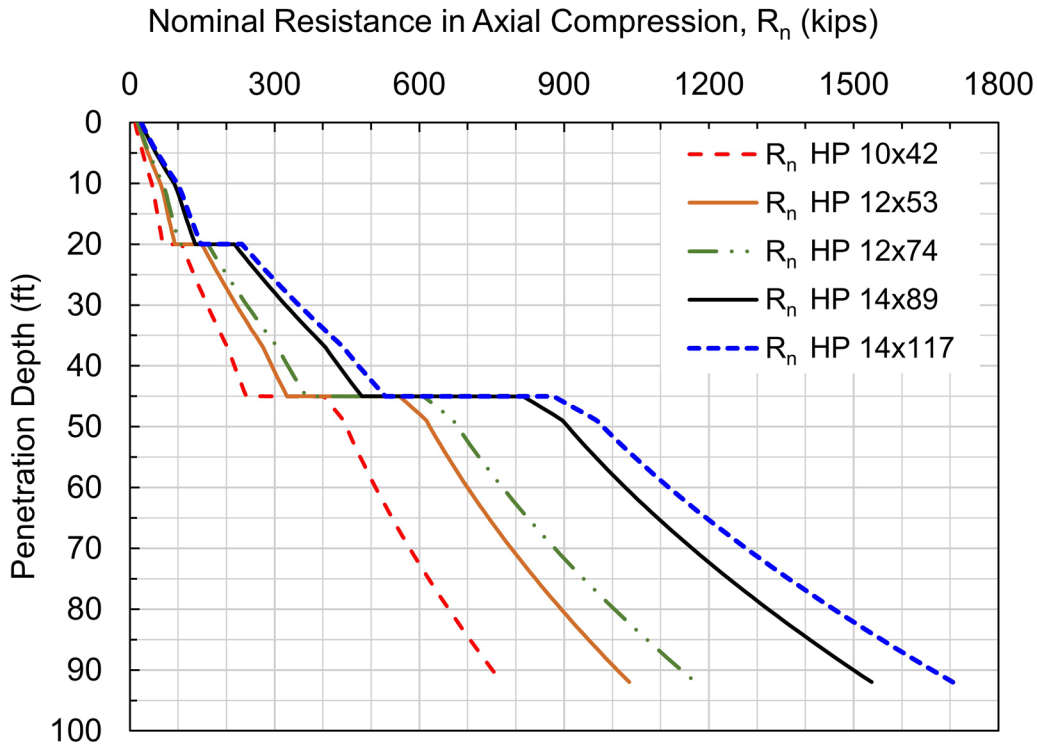


Figure D-9 Nominal geotechnical resistance in axial compression versus pile penetration depth for all candidate pile sections at the North Abutment.

Figure D-10 presents a design chart of the nominal resistance, R_n , and the factored geotechnical resistance, R_r , in axial compression versus pile penetration depth for the HP 12x74 H-pile section at the North Abutment. The design chart includes the nominal geotechnical resistance as well as a number of factored geotechnical resistances. The factored geotechnical resistances are presented for a selection of resistance determination methods. The factored geotechnical resistance versus penetration depth is plotted for resistance determination by a static load test with dynamic testing of 2% of the piles ($\phi_{dyn}=0.80$), by dynamic testing of at least two piles per site condition, but no less than 2% of the production piles, with signal matching ($\phi_{dyn}=0.65$), and by wave equation analysis ($\phi_{dyn}=0.50$). The factored geotechnical resistance is also plotted for resistance determination by the Nordlund static analysis method ($\phi_{stat}=0.45$). For a single pile section (HP 12x74), this figure illustrates the effects the various resistance determination methods have on the pile length required for a given factored resistance, the factored resistance available from a given pile section, and the potential impact of these factors on the number of piles needed to resist axial compression loads.

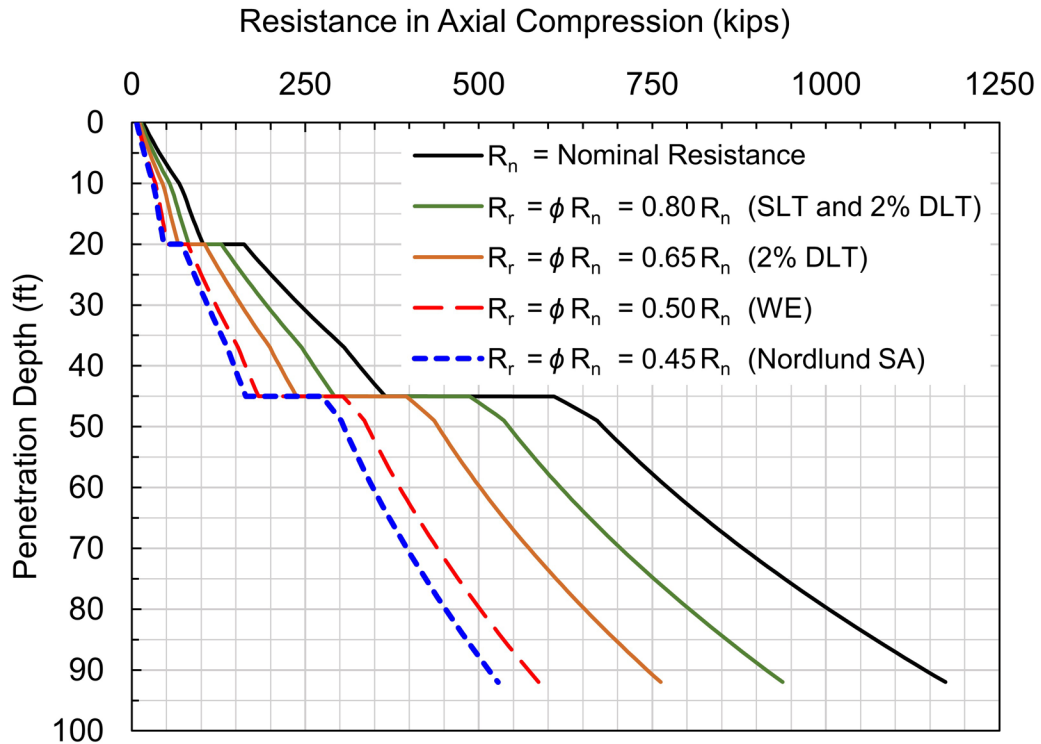


Figure D-10 Design chart of nominal and factored geotechnical resistance in axial compression versus pile penetration depth for HP 12x74 at the North Abutment.

The factored geotechnical resistance in axial compression for all candidate pile sections is presented in Figures D-11 through D-14 based on the same resistance determination methods. For all the candidate pile sections, these figures illustrate the effects the various resistance determination methods have on the pile length required for a given factored resistance, the factored resistance available from a given pile section, and the potential impact of these factors on the number of piles needed to resist axial compression loads.

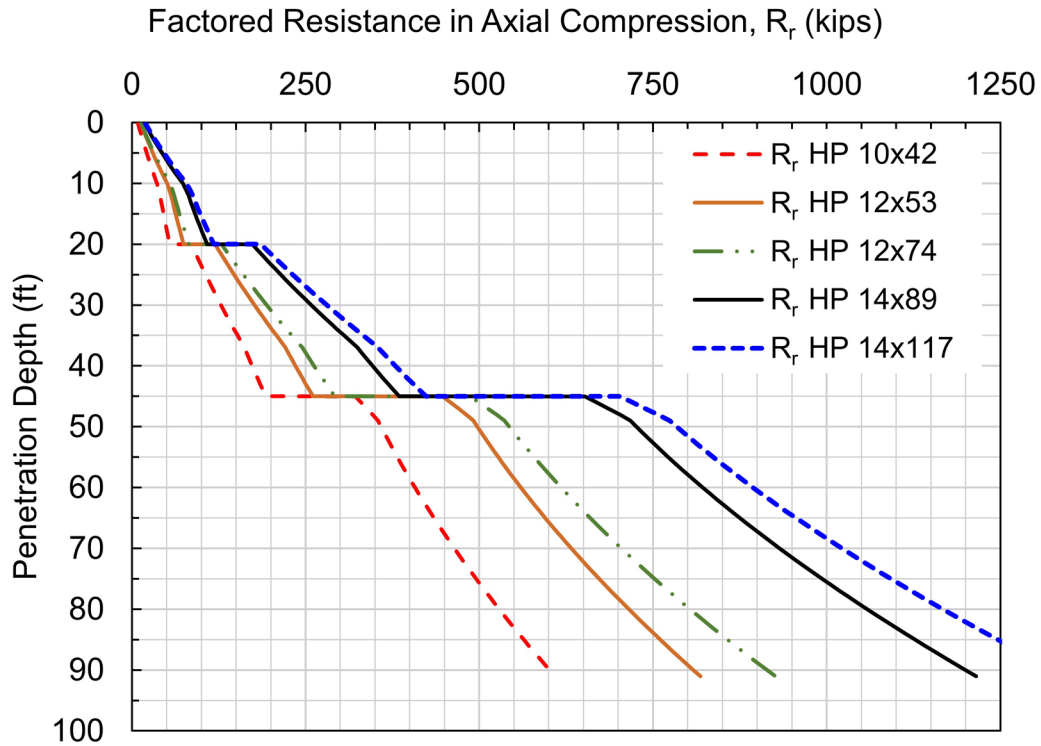


Figure D-11 Factored geotechnical resistance, R_r , in axial compression based on field determination by static load test and dynamic testing 2% of the piles, $\phi_{dyn}=0.80$.

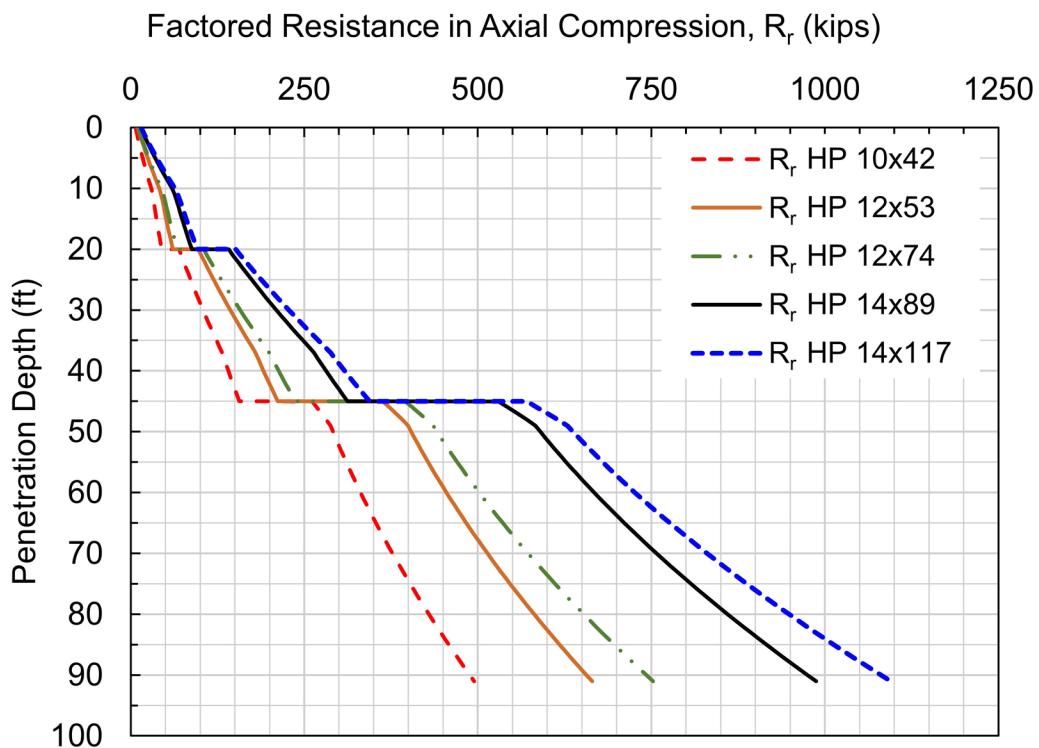


Figure D-12 Factored geotechnical resistance, R_r , in axial compression based on field determination by dynamic testing 2% of the piles, $\phi_{dyn}=0.65$.

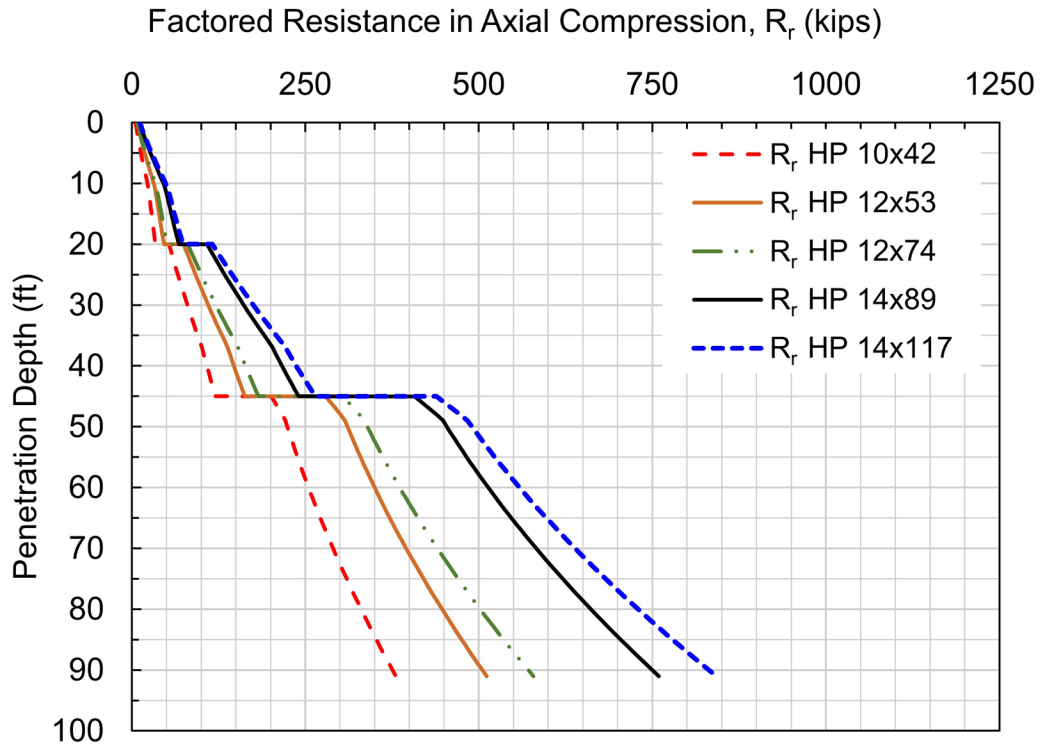


Figure D-13 Factored geotechnical resistance, R_r , in axial compression based on field determination by wave equation analysis, $\phi_{dyn}=0.50$.

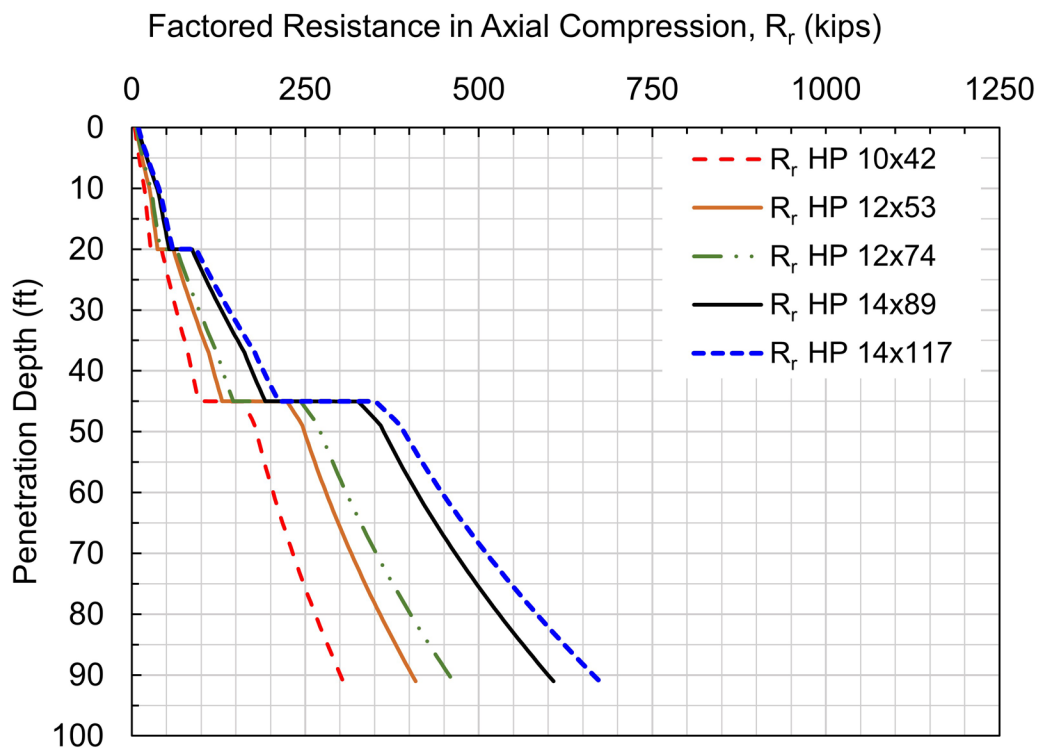


Figure D-14 Factored geotechnical resistance, R_r , in axial compression based on determination using Nordlund Method static analysis, $\phi_{stat}=0.45$.

D.10.2 Geotechnical Resistance in Axial Tension

In a similar manner, the nominal and factored geotechnical resistances in axial tension (uplift) were calculated using the Nordlund method and with the DrivenPiles computer program. Figure D-15 presents the nominal shaft resistance versus penetration depth for all the candidate pile sections. As outlined in Section 7.2.3.2.1 of Chapter 7, the factored resistance in axial tension is the shaft resistance multiplied by the resistance factor, ϕ_{up} , for the resistance determination method.

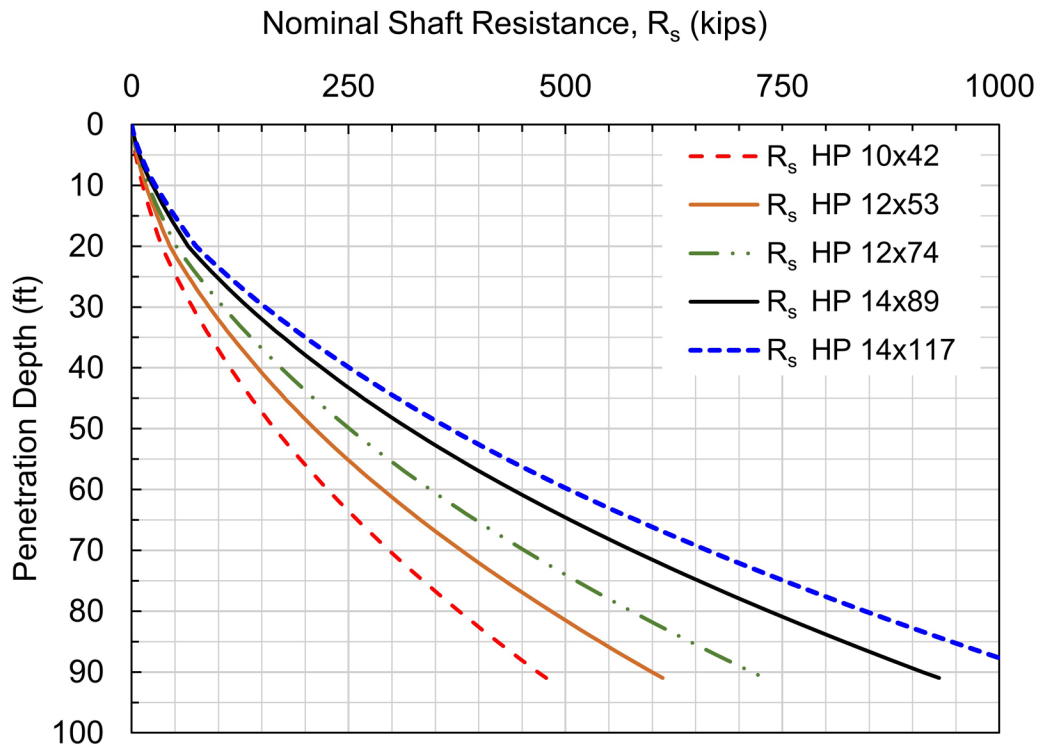


Figure D-15 Nominal shaft resistance versus penetration depth for all candidate pile sections at the North Abutment.

Figure D-16 presents a design chart of the nominal shaft resistance, R_s , and the factored geotechnical resistance, R_r , in axial tension versus pile penetration depth for the HP 12x74 H-pile section at the North Abutment. A resistance factor of 0.60 is used when the tension resistance is determined by a static load test, 0.50 when determined by a dynamic test with signal matching, and 0.35 when determined by the Nordlund static analysis method. For a single pile section (HP 12x74), this figure illustrates the effects the various resistance determination methods have on the pile length required for a given factored resistance, the factored resistance available from a given pile section, and the potential impact of these factors on the number of piles needed to resist axial tension loads.

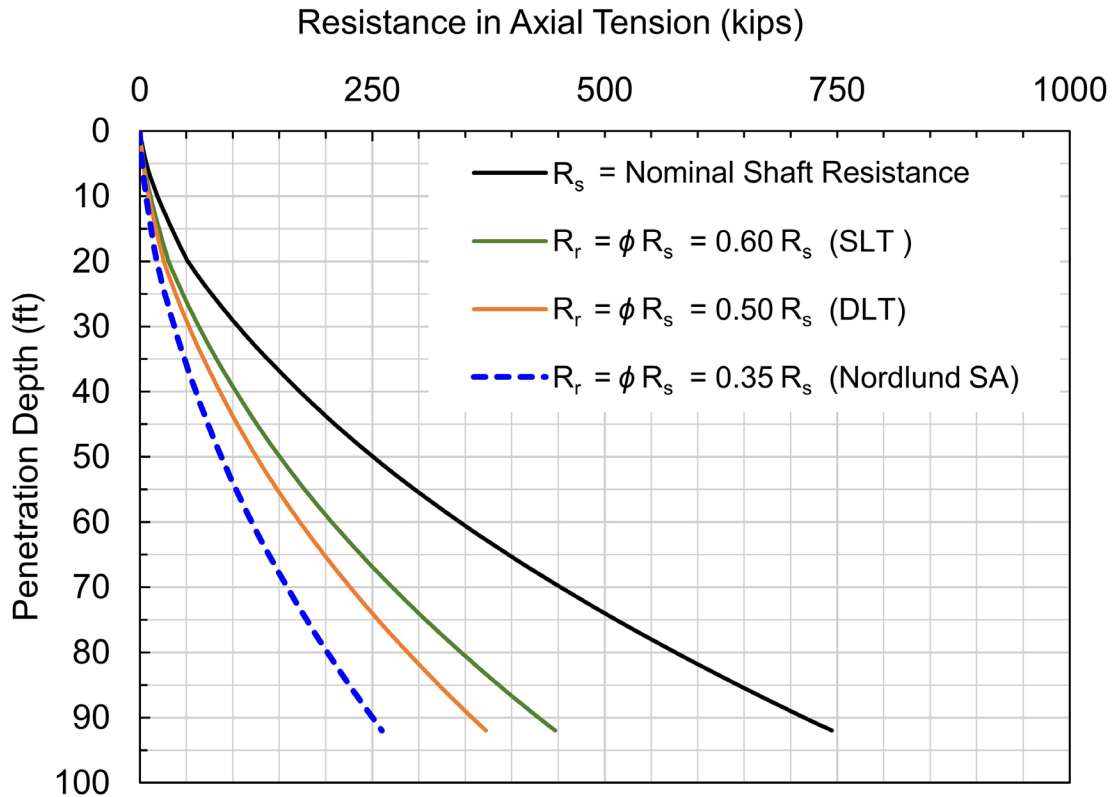


Figure D-16 Design chart of nominal and factored geotechnical resistance in axial tension for HP 12x74 at the North Abutment.

Figures D-17 to D-19 present the factored geotechnical resistance in axial tension versus penetration depth based on the field determination method for each of the candidate pile sections. For all the candidate pile sections, these figures illustrate the effects the various resistance determination methods have on the pile length required for a given factored resistance, the factored resistance available from a given pile section, and the potential impact of these factors on the number of piles needed to resist axial tension loads.

A review of the soil profile at the North Abutment indicates no unsuitable soil layers are present that should be ignored for load support. Abutment scour and liquefaction due to a seismic event are also not considerations.

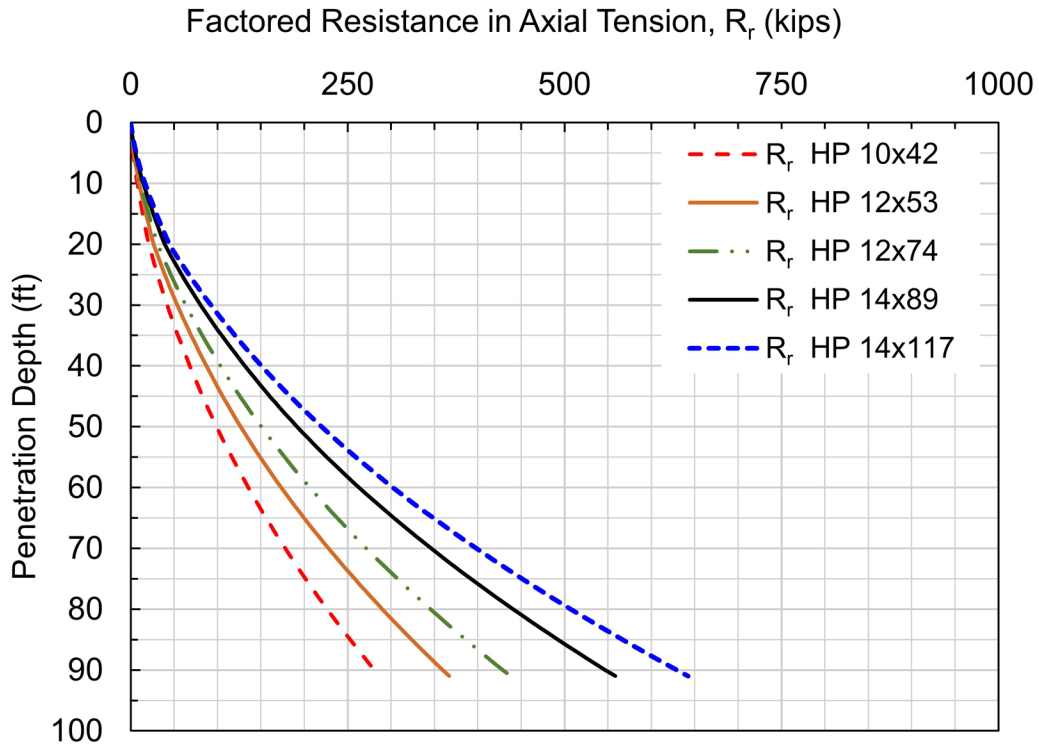


Figure D-17 Factored geotechnical resistance, R_r , in axial tension based on field determination by static load test, $\phi_{dyn}=0.60$.

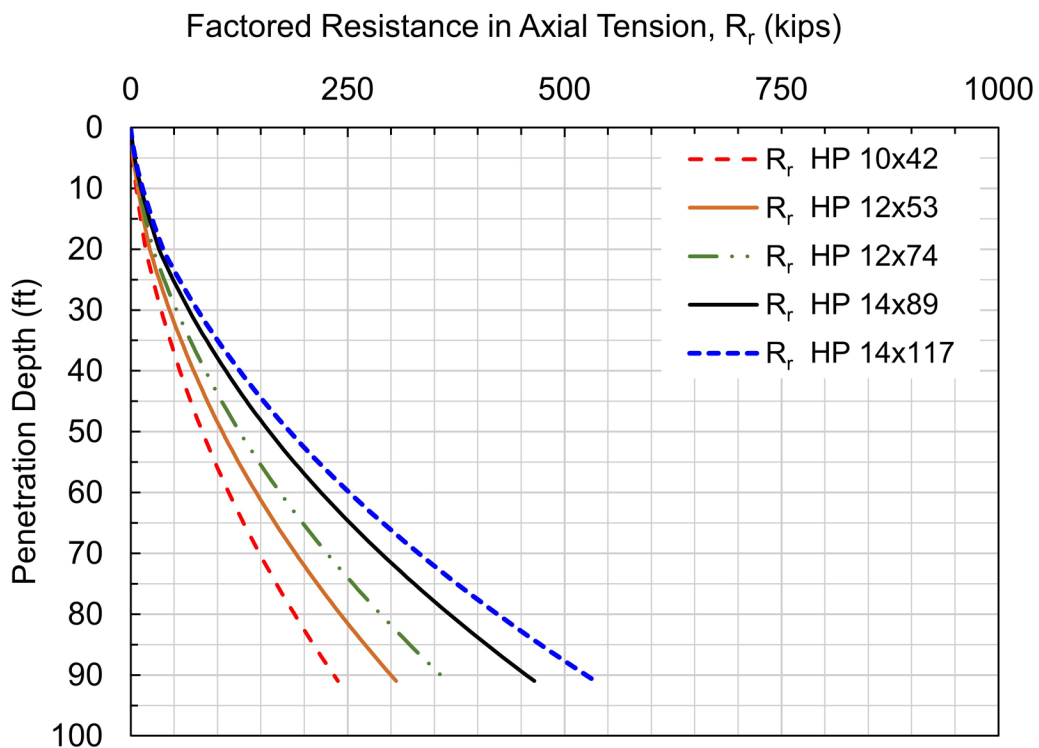


Figure D-18 Factored geotechnical resistance, R_r , in axial tension based on field determination by dynamic testing with signal matching, $\phi_{dyn}=0.50$.

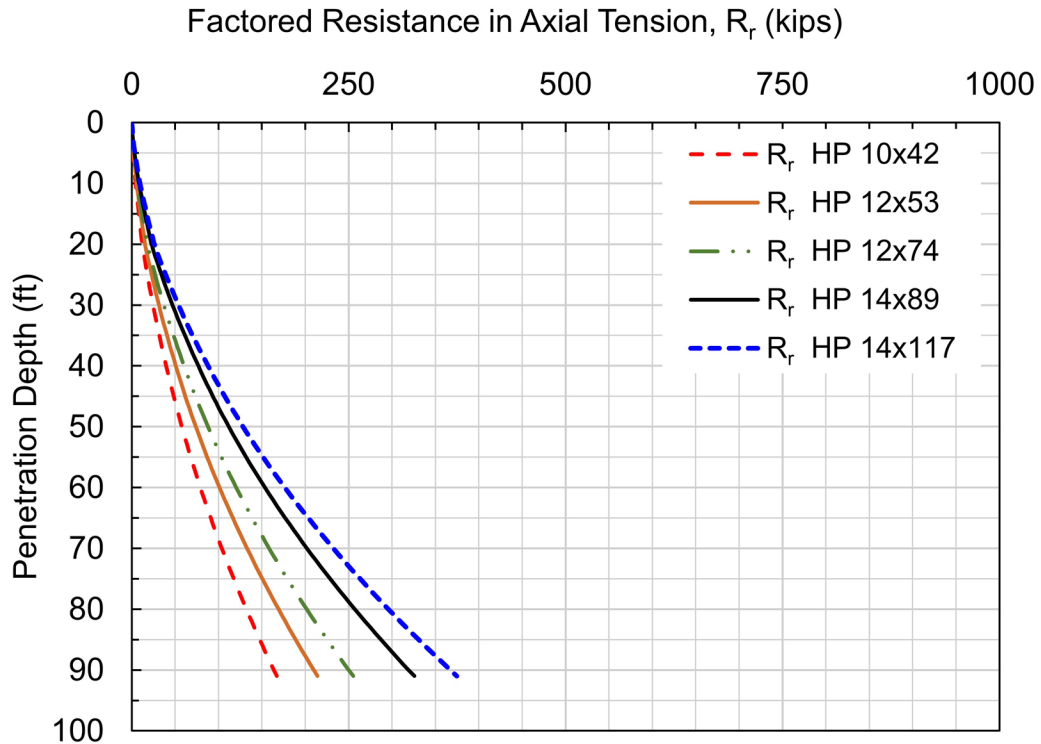


Figure D-19 Factored geotechnical resistance, R_r , in axial tension based on determination using Nordlund Method static analysis, $\phi_{stat}=0.35$.

D.10.3 Preliminary Pile Drivability Assessment

A preliminary assessment of pile drivability is now performed. A check of pile drivability at this time is essential to assess the constructability of candidate pile types and/or sections and to eliminate sections with insufficient drivability. Section 12.4 provides a detailed discussion of wave equation drivability analyses and their applications.

A candidate pile section must be capable of being driven to the penetration depth necessary to achieve the nominal geotechnical resistance in axial compression and tension, and to a penetration depth necessary to satisfy lateral load demands as well as axial and lateral deformation requirements. A suitably sized pile hammer must be capable of driving the pile to this penetration depth and nominal resistance at a reasonable blow count without exceeding material stress limits. As detailed in Chapter 12, the blow count should be between 30 and 120 blows per foot at the nominal resistance. If the pile cannot be driven within these requirements, a larger pile hammer, a pile section with greater impedance, or pile installation aids such as predrilling or jetting may be required to satisfy or improve drivability.

Driving stresses during pile installation should remain below the driving stress limits associated with the pile type and material strength. For the candidate steel H-piles, compression driving stress limits are given by Equation 8-33. As per ASTM A-572 requirements, new steel H-piles are rolled with a minimum yield stress of 50 ksi.

The driving stress limit, σ_{dr} , for candidate pile sections is then calculated as follows:

$$\sigma_{dr} = \phi_{da} (0.9 F_y) \quad [\text{Eq. 8-33}]$$

Where: ϕ_{da} = resistance factor, 1.0 for steel piles.
 F_y = yield stress, 50 ksi.

Therefore, the driving stress limit, σ_{dr} , is 45 ksi.

$$\sigma_{dr} = (1.0)(0.9 (50 \text{ ksi})) = 45 \text{ ksi}$$

Drivability analyses were performed for all five candidate pile sections. Since the specific pile hammer is often unknown at this point in the design, a reasonably sized, commonly available single acting diesel hammer was chosen for each of the candidate pile sections. As noted in Section 15.19, a hammer having a ram weight of 1 to 2% of the larger of the required nominal resistance or required nominal driving resistance often provides a reasonable initial estimate of a trial hammer size for wave equation analysis.

Table D-7 summarizes the factored structural resistance in axial compression, P_r , and the corresponding minimum and maximum nominal driving resistance associated with full section utilization and field determination methods ranging from a static load test with dynamic testing ($\phi_{dyn}=0.80$) to the FHWA modified Gates dynamic formula ($\phi_{dyn}=0.40$). Given that utilization of the full structural resistance is uncommon for piles driven in soil, a reasonable initial estimate of the trial hammer size for a wave equation drivability analysis is 1 to 1.5% of the minimum R_{ndr} . Driving stresses could exceed specified limits by choosing a hammer with a ram weight significantly larger than 2% of the minimum R_{ndr} . For each pile hammer, the wave equation default values were used for the helmet weight, hammer cushion materials, and the hammer cushion material properties.

Table D-7 Summary of Pile Hammers Used in Drivability Analyses

Pile Section	Pile Cross Sectional Area (in ²)	Factored Structural Resistance, P _r (kips)	Minimum R _{ndr} $\phi_{dyn}=0.80$ (kips)	Maximum R _{ndr} $\phi_{dyn}=0.40$ (kips)	Ram Weight 1% of Min R _{ndr} (%)	Trial Diesel Hammer Model	Ram Weight (kips)	Rated Energy (ft-kips)
HP 10x42	12.4	309	386	773	3.86	D25-52	5.51	62.0
HP 12x53	15.5	383	478	958	4.78	D30-52	6.62	74.4
HP 12x74	21.8	544	680	1360	6.80	D36-52	7.94	89.3
HP 14x89	26.1	652	815	1630	8.15	D46-52	10.14	114.1
HP 14x117	34.4	860	1075	2150	10.07	D50-52	11.03	124.0

For the soil resistance model, the output from DrivenPiles was converted to unit shaft resistance and unit toe resistance values and then input into the wave equation program. Similar soil resistances are thereby calculated versus depth by both the static analysis and wave equation analysis programs.

The dynamic soil properties for each soil layer were chosen in accordance with wave equation program recommendations. Selection of soil quake and damping parameters is discussed in Section 12.6.7. For the North Abutment profile, some soil setup is expected in the upper silty fine sand layer and the underlying layer of medium dense sand with little silt. Therefore soil setup factors of 1.2 were selected for these two layers. No setup was expected in the dense gravel with sand of Layer 3. Soil setup is discussed in Section 7.2.4.2 and a summary of typical soil setup factors is provided in Table 7-16. A summary of the dynamic soil properties chosen for the drivability analyses are summarized in Table D-8.

In soils that exhibit setup, the long term nominal resistance may be higher than the nominal driving resistance. Therefore, a gain/loss factor of 0.833 was used to estimate the nominal driving resistance versus depth in the drivability analyses. This gain/loss factor was determined from the inverse of the highest soil setup factor within the soil model (e.g., 1 divided by 1.2 equals 0.833). A gain/loss factor of 1 would be used if it was desired to model the nominal resistance instead of the nominal driving resistance and not consider the soil strength loss during driving and any subsequent soil setup. Refer to Chapter 12 for more detailed discussion on the selection of dynamic soil properties and soil setup factors.

Table D-8 Dynamic Soil Properties for North Abutment Soil Profile

Soil Layer	Pile Section	Shaft Quake (in)	Toe Quake (in)	Shaft Damping (s/ft)	Toe Damping (s/ft)	Soil Set-Up Factor
1	HP 10 x 42	0.10	0.17	0.10	0.15	1.2
1	HP 12 x 53	0.10	0.20	0.10	0.15	1.2
1	HP 12 x 74	0.10	0.20	0.10	0.15	1.2
1	HP 14 x 89	0.10	0.23	0.10	0.15	1.2
1	HP 14 x 117	0.10	0.23	0.10	0.15	1.2
2	HP 10 x 42	0.10	0.17	0.05	0.15	1.2
2	HP 12 x 53	0.10	0.20	0.05	0.15	1.2
2	HP 12 x 74	0.10	0.20	0.05	0.15	1.2
2	HP 14 x 89	0.10	0.23	0.05	0.15	1.2
2	HP 14 x 117	0.10	0.23	0.05	0.15	1.2
3	HP 10 x 42	0.10	0.08	0.05	0.15	1.0
3	HP 12 x 53	0.10	0.10	0.05	0.15	1.0
3	HP 12 x 74	0.10	0.10	0.05	0.15	1.0
3	HP 14 x 89	0.10	0.12	0.05	0.15	1.0
3	HP 14 x 117	0.10	0.12	0.05	0.15	1.0

The DrivenPiles program calculates the nominal driving resistance which models the soil strength lost during driving as well as the geotechnical nominal resistance once setup occurs. Figure D-20 presents the nominal shaft, toe, and driving resistance versus pile penetration depth for the HP 12x74 H-pile section at the North Abutment. These nominal driving resistances are presented numerically in Table D-9. To quantify the expected soil setup at a given pile penetration depth, the values from Table D-9 can be compared against the nominal resistance previously presented in Table D-6. Figure D-21 illustrates the difference between the expected nominal driving resistance and the geotechnical nominal resistance after setup for the HP 12x74 candidate pile section. For example, at a depth of 45 feet, the expected setup from shaft resistance is 35 kips.

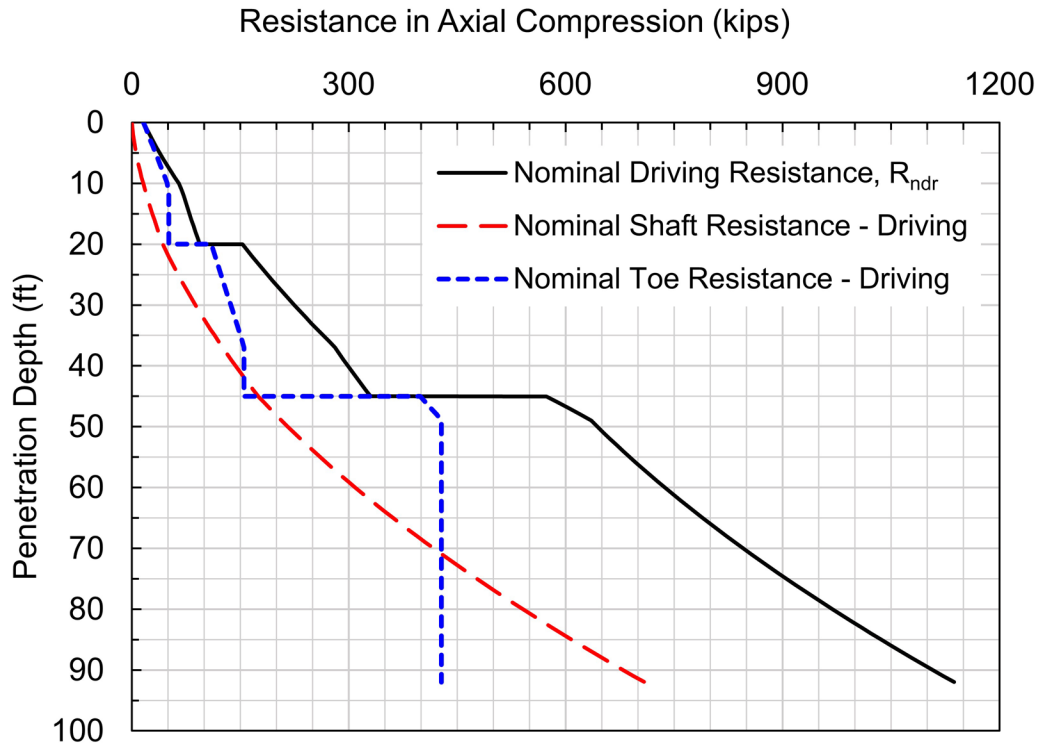


Figure D-20 Nominal driving resistance for HP 12x74 at the North Abutment.

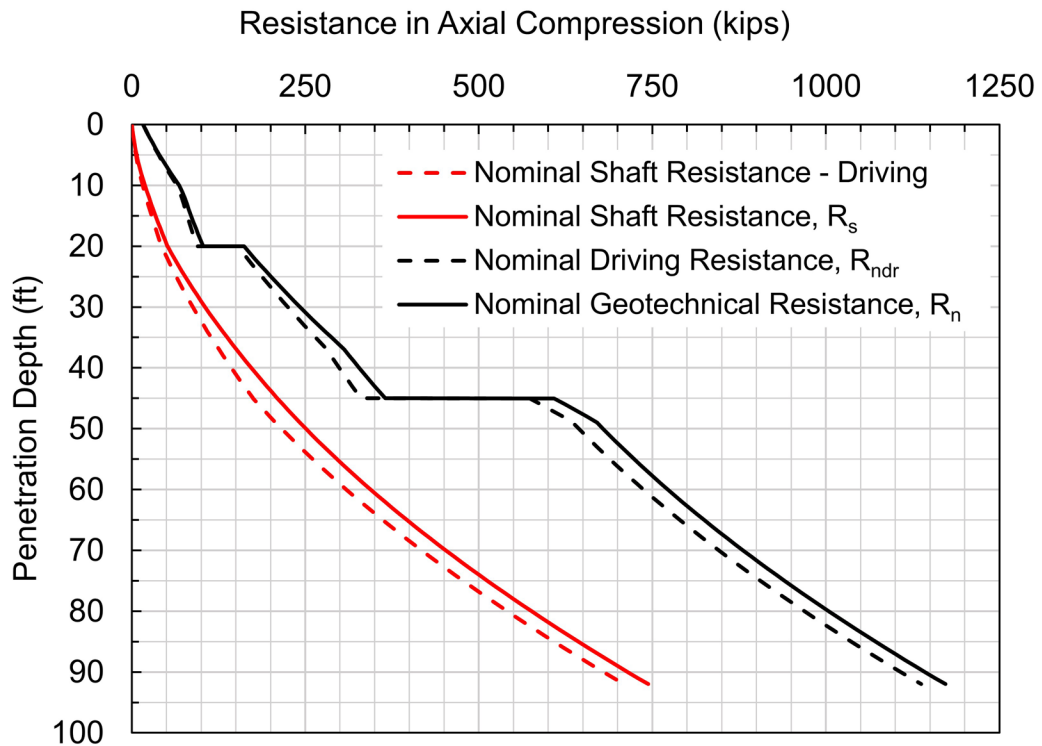


Figure D-21 Comparison of nominal driving resistance and nominal geotechnical resistance in axial compression for HP 12x74 at the North Abutment.

Table D-9 Summary of Nominal Driving Resistance Versus Pile Penetration
Depth for HP 12x74 at the North Abutment

Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Driving Resistance (kips)	Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Driving Resistance (kips)
0.01	0.01	16.43	16.44	37	125.82	155.41	281.23
1	0.87	19.68	20.55	38	131.59	155.41	287
2	1.91	22.96	24.87	39	137.46	155.41	292.87
3	3.1	26.24	29.34	40	143.43	155.41	298.84
4	4.45	29.52	33.97	41	149.5	155.41	304.91
5	5.96	32.8	38.76	42	155.67	155.41	311.08
6	7.63	36.08	43.71	43	161.94	155.41	317.35
7	9.46	39.36	48.82	44	168.31	155.41	323.72
8	11.45	42.64	54.09	44.99	174.71	155.41	330.12
9	13.59	45.92	59.52	45.01	174.86	398.77	573.63
9.99	15.88	49.17	65.04	46	182.68	406.4	589.07
10.01	15.92	49.21	65.14	47	190.73	414.1	604.83
11	18.32	50.53	68.85	48	198.93	421.8	620.73
12	20.8	51.26	72.06	49	207.28	428.1	635.38
13	23.35	51.26	74.6	50	215.78	428.1	643.88
14	25.96	51.26	77.21	51	224.44	428.1	652.53
15	28.63	51.26	79.89	52	233.24	428.1	661.34
16	31.37	51.26	82.63	53	242.2	428.1	670.3
17	34.18	51.26	85.43	54	251.31	428.1	679.4
18	37.05	51.26	88.3	55	260.56	428.1	688.66
19	39.98	51.26	91.24	56	269.97	428.1	698.07
19.99	42.95	51.26	94.2	57	279.53	428.1	707.63
20.01	43.02	110.15	153.17	58	289.25	428.1	717.34
21	47.05	112.8	159.85	59	299.11	428.1	727.21
22	51.23	115.47	166.7	60	309.12	428.1	737.22
23	55.5	118.14	173.64	61	319.29	428.1	747.39
24	59.87	120.81	180.69	62	329.6	428.1	757.7
25	64.35	123.48	187.83	63	340.07	428.1	768.17
26	68.92	126.15	195.07	64	350.69	428.1	778.79
27	73.6	128.83	202.43	65	361.46	428.1	789.56
28	78.37	131.51	209.88	66	372.38	428.1	800.48
29	83.24	134.19	217.43	67	383.45	428.1	811.55
30	88.21	136.86	225.08	68	394.67	428.1	822.77
31	93.29	139.54	232.83	69	406.05	428.1	834.15
32	98.46	142.21	240.67	70	417.57	428.1	845.67
33	103.73	144.88	248.61	71	429.25	428.1	857.35
34	109.1	147.55	256.66	72	441.08	428.1	869.17
35	114.57	150.22	264.79	73	453.06	428.1	881.15
36	120.15	152.89	273.03	74	465.18	428.1	893.28

Table D-9 Summary of Nominal Driving Resistance Versus Pile Penetration Depth for HP 12x74 at the North Abutment (continued)

Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Driving Resistance (kips)	Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Driving Resistance (kips)
75	477.47	428.1	905.56	84	594.79	428.1	1022.89
76	489.9	428.1	917.99	85	608.58	428.1	1036.68
77	502.48	428.1	930.58	86	622.52	428.1	1050.62
78	515.21	428.1	943.31	87	636.62	428.1	1064.72
79	528.1	428.1	956.2	88	650.86	428.1	1078.96
80	541.13	428.1	969.23	89	665.26	428.1	1093.36
81	554.32	428.1	982.42	90	679.81	428.1	1107.9
82	567.66	428.1	995.76	91	694.5	428.1	1122.6
83	581.15	428.1	1009.25	91.99	709.2	428.1	1137.3

Graphical outputs of the preliminary drivability analyses are shown in Figure D-22. The nominal driving resistance, the blow count or pile penetration resistance, and the compression driving stress are presented versus pile penetration depth for each of the five candidate pile sections. As previously noted, the recommended blow count limit is 120 blows per foot (10 blows per inch), and the recommended driving stress limit is 45 ksi. A circular reference marker is indicated on the blow count versus penetration depth plot highlighting the depth where the blow count first exceeds 120 blows per foot. This marker is also shown at the same depth on the nominal driving resistance versus depth plot indicating the nominal driving resistance achieved when practical refusal driving conditions are encountered with the selected hammer in the modeled driving conditions. Similarly, the marker is shown at the same depth on the compression driving stress versus depth plot indicating the compression driving stress when practical refusal driving conditions are encountered.

In Figure D-22, the drivability results for the HP 12x53 section stand out from the other pile sections. The drivability results for the HP 12x53 H-pile section driven with a D30-52 driving system operated at the maximum fuel setting illustrate that refusal driving conditions of 120 blows per foot are encountered near a pile penetration depth of 52 feet. Compression driving stresses for the HP 12x53 with this hammer exceed the 45 ksi compression stress limit at a pile penetration depth of 46 feet, and reach a maximum of 45.9 ksi at a depth of 49 feet.

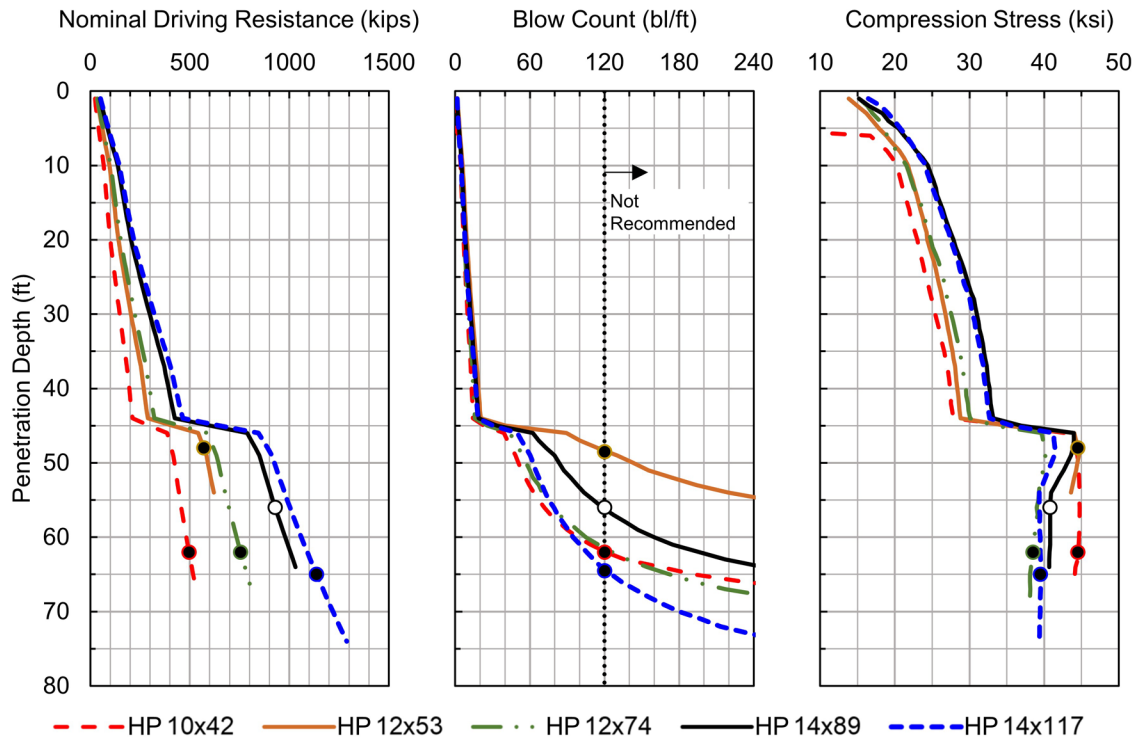


Figure D-22 Preliminary drivability results at the North Abutment.

The D30-52 hammer was also modeled at a reduced fuel setting, fuel setting 3. However, while the driving stresses can now be maintained within limits, refusal driving conditions are now encountered 4 feet shallower at a pile penetration depth of 48 feet. These results indicate the HP 12x53 pile section cannot be driven beyond 48 feet without exceeding either blow count or driving stress limits.

For the remaining candidate pile sections and selected pile hammers, the maximum pile penetration depth before achieving 120 blows per foot ranges between 57 and 64 feet. Driving stresses for some of the other candidate pile sections were close to the driving stress limit, but in all cases, the blow count limit and not the driving stress limit determined maximum drivability. A summary of the preliminary drivability results is presented in Table D-10. Once the estimated and/or minimum pile toe elevations are determined in Block 12 through Block 15 of the design process, the drivability results in Figure D-22 and Table D-10 should be reviewed to confirm that the candidate pile section can be driven to the estimated or required pile penetration depth, at reasonable blow counts, and within driving stress limits.

Table D-10 Summary of Preliminary Drivability Results at North Abutment

Pile Section	Pile Hammer	Fuel Setting	Pile Penetration Depth at Practical Refusal Limit (feet)	Nominal Driving Resistance at Practical Refusal Limit (kips)	Anticipated Nominal Resistance at Depth of Practical Refusal (kips)	Penetration Depth Exceeding Compression Driving Stress Limit (feet)	Maximum Compression Driving Stress (ksi)
HP 10x42	D25-52	4	61	489	512	> 61	43.7
HP 12x53	D30-52	4	52	605	634	46	45.9
HP 12x53	D30-52	3	48	571	600	> 48	44.5
HP 12x74	D36-52	4	61	746	780	> 61	40.1
HP 14x89	D46-52	4	57	941	985	> 57	44.5
HP 14x117	D50-52	4	64	1121	1172	> 64	41.4

As noted previously, a hammer having a ram weight of 1 to 2% of the required nominal driving resistance often provides a reasonable initial estimate of hammer size. However, if a candidate pile type does not satisfy drivability requirements with this initially selected hammer and driving stress levels are within limits, other hammers should be evaluated before eliminating the candidate pile type. For example, a D30-52 hammer was initially selected for the HP 12x74 H-pile. This driving system has a ram weight of 6.6 kips or 0.97% of the required nominal resistance, so it is slightly below the recommended trial hammer size. The blow count limit of 120 blows per foot with the D30-52 was reached at a nominal resistance of 680 kips and a pile penetration depth of 55 feet. However, compression driving stresses were 38.5 ksi or less, indicating a larger hammer could be considered.

Drivability analyses were then performed for both a D36-52 and a D46-52 pile hammer. Figure D-23 presents the drivability results for all three of these driving systems. Note that the analyses indicate that the D36-52 can drive the candidate pile section to a nominal driving resistance of 756 kips at a pile penetration depth of 62 feet before reaching the blow count limit. The maximum predicted compression driving stress of 40 ksi are also within driving stress limits. Similarly, the D46-52 can drive the same candidate pile section to a nominal driving resistance of 867 kips at a pile penetration depth of 72 feet before reaching the blow count limit. The maximum predicted compression driving stress of 44 ksi with the D46-52 is also within driving stress limits. Hence, it would have been erroneous to eliminate the HP 12x74 pile section solely based on the D30-52 drivability results. The drivability results from the two other hammers clearly indicate the candidate section can be driven to significantly higher nominal driving resistances and to greater pile penetration depths before reaching drivability limits.

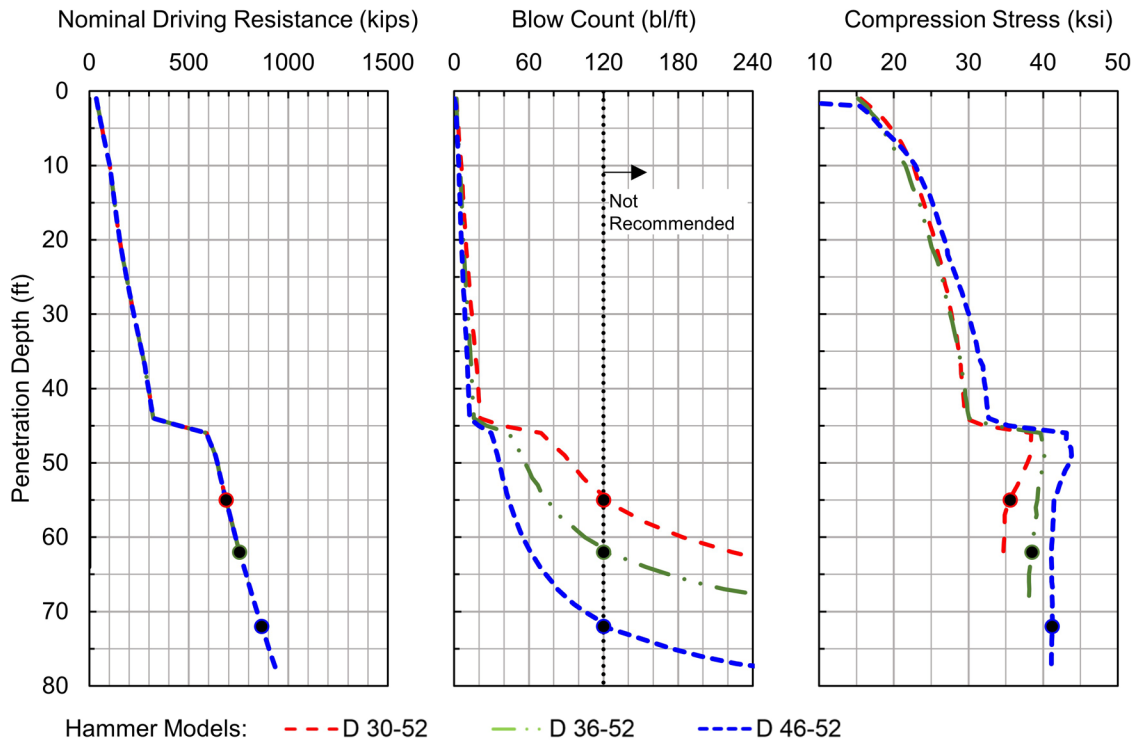


Figure D-23 Hammer size comparison to drive the HP 12x74 pile section.

At this time, the factored axial compression and tension resistances versus depth have been calculated and preliminary drivability assessments have been performed for all candidate pile sections. The factored geotechnical resistances have been evaluated for the resistance determination methods under consideration. Maximum achievable penetration depths have also been assessed with drivability analyses.

It is unreasonable to carry all possible combinations of candidate sections, resistance determination methods, and possible group configurations forward in the design process. Hence, experience, engineering judgement, and agency practice should be combined when selecting candidate pile types and sections, resistance determination methods, and potential group configurations. Otherwise, the number of possible design permutations will become unreasonable.

D.11 Block 11: North Abutment – Estimate Preliminary Number of Piles, Preliminary Pile Group Size, and Resolve Individual Pile Loads for All Limit States

The structural engineer has provided the anticipated loads for the controlling limit states at the North Abutment. These limit state loads are presented in Table D-11. The Strength I limit state loads are used to evaluate geotechnical resistance in axial

compression and tension, as well as for lateral loading. Service I limit state loads were also provided by the structural engineer without live loads. The Service I without live load (LL) includes only unfactored permanent loads such as the superstructure and wearing surface, pile cap and stem, utilities, and vertical earth pressure among others. The Service I without live load should be used for evaluating vertical deformation. There are no loads in the transverse direction at this abutment.

Table D-11 Limit State Loads on North Abutment

Limit State	Q (kips)	V_{uy} (kips)	M_{uy} (k-ft)
Strength I	2505	846	6625
Service I	1838	566	4600
Service I, without live load	1540	562	4162

Based on past experience, the agency generally utilizes 2 rows of piles at abutments with a minimum center to center pile spacing of at least 3 pile diameters. Three potential pile group configurations are therefore being considered: 2 rows of 9 piles, 2 rows of 11 piles, and 2 rows of 13 piles. These group configurations are identified as Group Configuration 1, 2, and 3, respectively in Table D-12. Because of site constraints, the pile cap length is limited to 43 feet. Furthermore, the distance from the center of any exterior pile to the pile cap edge must be at least 1.25 feet in both the transverse (x) and longitudinal (y) direction.

Table D-12 Potential Pile Group Configurations

Group Configuration	Piles per Row	Total Number of Piles	S_{bx}^* (feet)	Total Footing Length (feet)	S_{by}^* (feet)	Total Footing Width (feet)
1	9	18	5.0	42.5	4.0	6.5
2	11	22	4.0	42.5	4.0	6.5
3	13	26	3.0	38.5	4.0	6.5

* S_{bx} and S_{by} are illustrated in Figure D-24.

The following calculation is for the Group Configuration 1 and all applicable loads. For this configuration, 9 piles per row are used in two separate rows. Thus the transverse pile spacing, S_{bx} , is 5'-0" and the total footing length is 42'-6". The longitudinal pile spacing, S_{by} , is 4'-0". Figure D-24 shows the layout for the Group Configuration 1 pile cap.

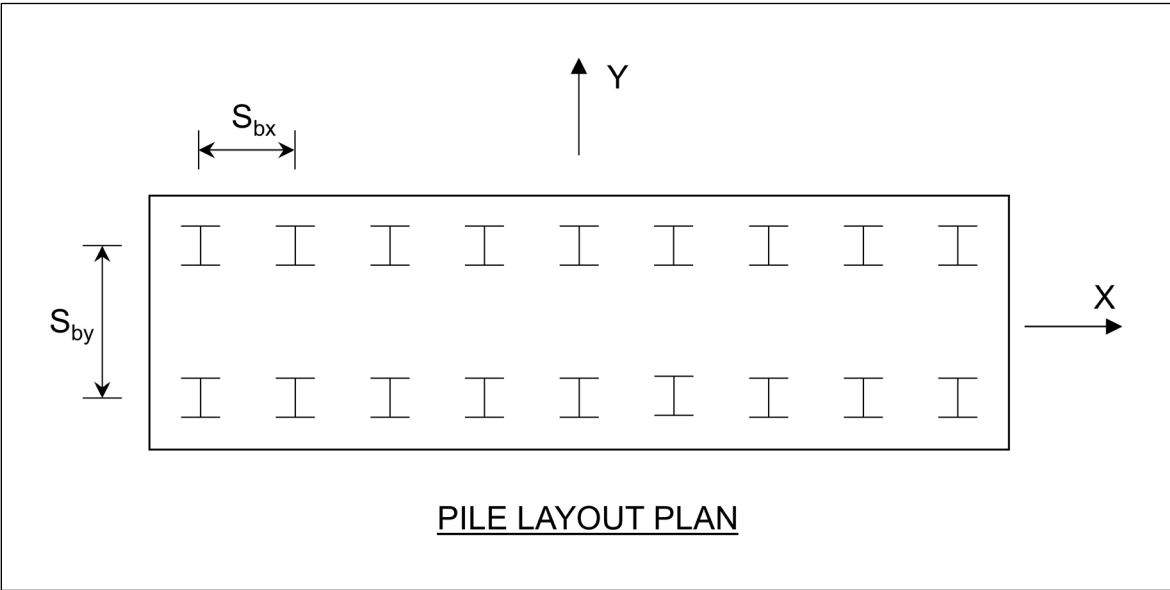


Figure D-24 Group Configuration 1 pile cap plan view.

For the established limit state loads and the trial pile group configuration depicted in Figure D-24, reactions for both the front and the back rows of piles were determined. Compression loads are taken as positive. The maximum factored load applied to each pile was subsequently calculated by dividing the reactions by the number of piles. Figure D-25 shows the free body diagram for determining the reactions and the resulting factored load per pile for both the front and the back row.

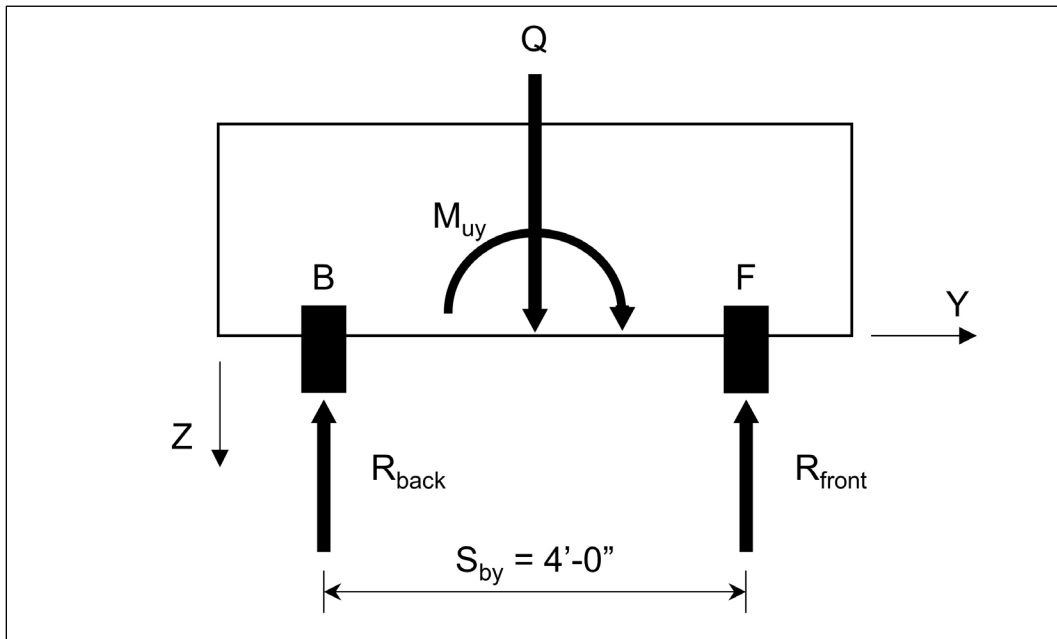


Figure D-25 Elevation view of cap free body diagram.

Table D-13 summarizes the limit state loads and the corresponding front and back row reactions. Strength I loads, Service I loads and the Service I loads without live load (LL) are provided.

Table D-13 Limit State Loads and Row Reactions for Group Configuration 1

Limit State	Q (kips)	M _{uy} (k-ft)	R _{front} (kips)	R _{back} (kips)
Strength I	2505	6625	2908	-406
Service I	1838	4600	2069	-231
Service I, without LL	1540	4162	1811	-271

Table D-14 presents the maximum factored load per pile based upon the number of piles in the group configuration. For Group Configuration 1, the Strength I limit state front-row reaction of 2908 kips divided by 9 piles yields a maximum factored load per pile of 323 kips. Table D-14 summarizes the factored load per pile for other limit states and group configurations.

Table D-14 Factored Load Per Pile for Alternative Pile Group Configurations

Group Configuration	Strength I, Q (compression) (kips)	Strength I, Q (tension) (kips)	Strength I, V _{uy} (kips)	Service I, without LL Q (kips)
1	323	-46	47	201
2	265	-37	39	165
3	224	-31	33	139

D.12 Block 12: North Abutment – Estimate Pile Penetration Depth for Maximum Axial Compression Loads; Check Group Efficiency in Axial Compression

The estimated minimum pile penetration depth necessary to obtain a factored geotechnical resistance that is equal or greater than the maximum factored load per pile is now determined. Note that the factored geotechnical resistance in axial compression and the resulting pile penetration depth is dependent upon the resistance determination method. Therefore, the influence of the field resistance determination method on the design needs to be evaluated at this point in the design process and some resistance determination methods may be eliminated from further design consideration.

The pile penetration depth necessary for the maximum factored geotechnical resistance of 323 kips on a given candidate pile section can be evaluated from Figure D-11 to Figure D-14 for the various resistance determination methods. For a factored geotechnical resistance of 323 kips in axial compression, it is estimated that pile length variation due to changes in the resistance determination method will be relatively minor. For this factored resistance, piles will also likely be driven into the gravel layer with cobbles encountered 45 feet below footing grade which increases the potential for pile damage. Therefore, it is decided that dynamic testing 2% of the production piles will be used for the resistance determination method.

Figure D-12 illustrated the factored geotechnical resistance versus penetration depth for the 5 candidate pile sections based on determination testing by dynamic testing 2% of the piles. From the static analysis results presented in this plot, the estimated penetration depth for the maximum factored geotechnical resistance of 323 kips in axial compression ranges from 43 feet for the HP 14x117 H-pile section to 59 feet for the HP 10x42 H-pile section. Similarly, the estimated pile penetration depths for all candidate pile sections and all group configurations were determined and are summarized in Table D-15.

Table D-15 Estimated Pile Penetration Depth Requirements for the Factored Geotechnical Resistance in Axial Compression at the Strength I Limit State

Group Configuration	Factored Load per Pile (kips)	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
1	323	59	45	45	45	43
2	265	46	45	45	38	35
3	224	45	45	43	32	30

Next, the group efficiency in axial compression is evaluated. Per Section 7.2.2.1, the nominal geotechnical resistance of a pile group in cohesionless soil, can be taken as the sum of the individual pile nominal geotechnical resistances. In a similar manner, the factored geotechnical resistance of the pile group in cohesionless soil is taken as the sum of the individual pile factored geotechnical resistances. This is recommended so long as 1) the bearing layer is not underlain by weak soil layers, 2) the piles are not installed at a pile spacing of less than 3 times the pile diameter, and 3) no special installation procedures are anticipated such as jetting or predrilling. Since all these conditions are met at the North Abutment, the nominal and factored group resistances are satisfactory.

D.13 Block 13: North Abutment – Establish Minimum Pile Penetration Depth for Axial Tension Loads; If Conditions Warrant, Modify Design and Return to Block 10

The factored geotechnical resistance in axial tension must also be evaluated as the back row of piles will be loaded in tension (Table D-13). In this case, the minimum required factored geotechnical resistance in axial tension is established using the Strength I limit state and is determined following the procedure outlined in Section 7.2.3.2. The analysis presented in this appendix slightly differs from the procedure outlined in Chapter 7 in that only a single row of piles is providing the tension resistance rather than the entire pile group. Two analyses are performed; one that considers the factored shaft resistances from individual piles, and one that considers the weight of a soil block acting with the piles. The lesser tension resistance determined from either method controls the design.

As noted in Table D-13, the Strength I limit state tension load on the back row of piles for the Group Configuration 1 is 406 kips. Therefore, the minimum factored geotechnical resistance in axial tension required from an individual pile is this factored load divided by the 9 piles in the rear row or 46 kips. In a similar manner, the minimum factored geotechnical resistance required from an individual pile in axial tension is 37 kips for Group Configuration 2, and 33 kips for Group Configuration 3.

As noted earlier, dynamic testing with signal matching will be used as the resistance determination method in the field. Therefore, the AASHTO resistance factor, ϕ_{up} , is 0.50 (Table 7-2 of Chapter 7). Figure D-18 provides plots of the factored geotechnical resistance in axial tension versus depth for all of the candidate pile types based on this resistance determination method. For each candidate section, this figure should be entered on the x-axis at the required factored axial tension resistance to determine the corresponding pile penetration depth.

Following this procedure, the estimated pile penetration depth to achieve the factored geotechnical resistance in axial tension for each candidate pile section and group configuration is summarized in Table D-16.

Table D-16 Estimated Minimum Pile Penetration Depth for Maximum Factored Geotechnical Resistance in Axial Tension at the Strength I Limit State

Group Configuration	Required Factored Resistance in Axial Tension (kips)	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
1	46	36	31	28	24	23
2	37	31	27	25	23	20
3	33	29	25	23	20	19

Next, the tension resistance of the pile row when considered as a soil block is calculated. For Group Configuration 1, the required factored tension resistance of the back row remains 406 kips (Table D-13) and the resistance is derived from the weight of soil as a block. The weight of soil needed in the block, W_{Block} , to resist the tension load is determined with Equation 7-39 where W_{Block} is substituted for R_{ug} .

$$R_r = \phi R_n = \phi_{ug} R_{ug} \quad \text{Eq. [7-39]}$$

Where:

- R_r = factored uplift resistance (kips).
- ϕ_{ug} = resistance factor for group uplift per Table 7-1, 0.50.
- R_{ug} = nominal uplift resistance of the pile group (kips).

and

$$R_{ug} = W_{Block}$$

Solving for the minimum W_{Block} :

$$W_{Block} = \frac{R_r}{\phi_{ug}} \quad \text{Eq. [7-39 modified]}$$

$$W_{Block} = \frac{(406 \text{ kips})}{(0.50)}$$

$$W_{Block} = 812 \text{ kips}$$

The length of the pile group will change depending upon the group configuration. For Group Configurations 1 and 2, the pile group length is 41 feet from exterior pile

edge to exterior pile edge assuming a 12 inch pile width/diameter. For Group Configuration 3, the pile group length is 37 feet using the same pile dimension. The effect of a smaller pile size on the pile group length is negligible (e.g. for HP 10x42 section, the pile group length is 40.83 feet).

The soil profile consists of three layers each with different total and effective unit weights. Therefore, the weight contribution from each soil layer was used to calculate the soil block weight at a given pile toe elevation. The following calculation is performed for Group Configuration 1 and 2 with a pile group length of 41 feet and an embedded pile length of 27 feet. Table D-17 presents a summary of the effective soil unit weight and geometry for each layer comprising the soil block. Figure D-26 provides a visual representation of the soil volume used to determine the soil block weight.

Table D-17 Geometry of Soil Block Layers for Nominal Group Tension Resistance Computation with Pile Toe at 27 Feet

Soil Block Layer	γ' (pcf)	Layer Thickness (feet)	Depth to Bottom (feet)	Z_{bottom} (feet)	Z_{top} (feet)	B_{bottom} (feet)	B_{top} (feet)
1	105.0	10.0	10.0	49.5	54.5	9.5	14.5
2	42.6	10.0	20.0	44.5	49.5	4.5	49.5
3	49.6	20.0	27.0	41.0	44.5	1.0	4.5

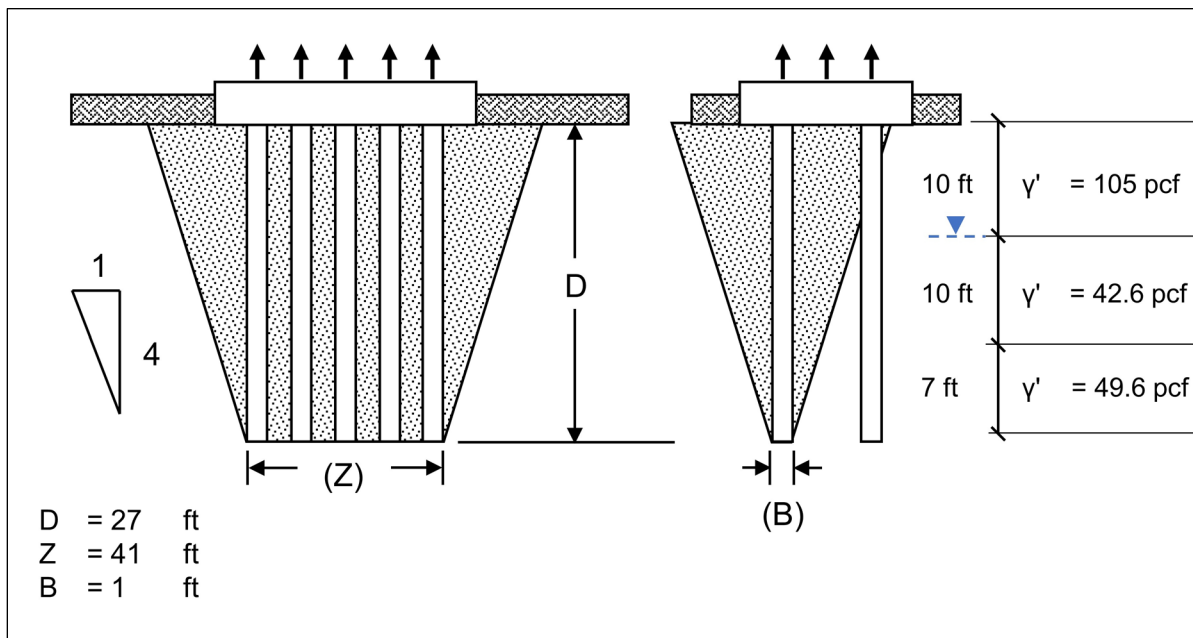


Figure D-26 Nominal tension resistance of pile group based on soil block weight.

To determine the volume of soil for each layer, the geometry presented in Table D-17 was used to calculate the volume of each layer's inverted obelisk using Equation D-1. The volume of each block layer was then multiplied by the respective soil unit weight to determine the soil block weight. All three block layers were then summed to calculate the total weight of the block. An example calculation is shown for Layer 1 of the soil block. Completed calculations are summarized in Table D-18.

Determine the volume of soil in Block Layer 1

$$V = \frac{h}{6} [Z_{bottom}B_{top} + Z_{top}B_{bottom} + 2 * (Z_{top}B_{top} + Z_{bottom}B_{bottom})] \quad \text{Eq. D-1}$$

$$V = \frac{(10 \text{ feet})}{6} [(49.5 \text{ feet}) * (14.5 \text{ feet}) + (54.5 \text{ feet}) * (9.5 \text{ feet}) + 2 * ((54.5 \text{ feet}) * (14.5 \text{ feet}) + (49.5 \text{ feet}) * (9.5 \text{ feet}))]$$

$$V_{Block \text{ Layer } 1} = 6260.8 \text{ ft}^3$$

Calculate the effective soil weight in Block Layer 1

$$W_{Block \text{ Layer } 1} = V_{Block \text{ Layer } 1} * \gamma'$$

$$W_{Block \text{ Layer } 1} = (6260.8 \text{ ft}^3) * (105 \text{ pcf}) \left(\frac{1 \text{ kip}}{1000 \text{ lbs}}\right)$$

$$W_{Block \text{ Layer } 1} = 657.4 \text{ kips}$$

Table D-18 Calculation of Soil Volume and Soil Block Weight for Nominal Group Tension Resistance

Block Layer	Depth to Bottom (feet)	Volume of Soil (ft ³)	γ' (pcf)	Weight of Soil (lbs)	Weight of Soil (kips)
1	10	6,260.8	105.0	657,387.5	657.4
2	20	3,310.8	42.6	141,041.5	141.0
3	27	830.1	49.6	41,172.1	41.2
					839.6

The required soil block weight of 812 kips is achieved at approximately 27 feet of pile penetration. Hence any pile penetration depth of 27 feet or greater would satisfy the required nominal group tension resistance. For Group Configuration 3 with a pile group length of 37 feet, the minimum pile penetration depth to achieve a soil block weight in excess of the required nominal tension resistance of 812 kips is 28 feet.

The required minimum pile penetration depth to satisfy the nominal geotechnical resistance in axial tension is the greater depth of the above calculated individual or group resistances.

The minimum pile penetration depth necessary for the HP 12x74 section in Group Configuration 1 is determined as follows:

$D_{individual}$ = minimum penetration depth based on sum of individual pile resistance, 28 feet (Table D-16).

D_{block} = minimum penetration depth based on weight of soil block, 27 feet.

Therefore, the minimum pile penetration depth needed to satisfy the nominal geotechnical resistance in axial tension, $D_{min-tension}$, is 28 feet.

In a similar manner, this check was performed for all candidate pile sections and all group configurations. The resulting minimum pile penetration depth necessary to achieve the nominal geotechnical resistance requirements for each candidate pile section within the specified group configuration is summarized in Table D-19.

Table D-19 Established Minimum Required Pile Penetration Depth for Factored Geotechnical Resistance in Axial Tension at North Abutment

Group Configuration	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
1	36	31	28	27*	27*
2	31	27*	27*	27*	27*
3	29	28*	28*	28*	28*

* Indicates axial tension resistance governed by soil block weight.

D.14 Block 14: North Abutment – Establish Minimum Pile Penetration Depth for Lateral Loads. Determine p-y Models, Required Geomaterial Parameters, and Perform Lateral Load Analysis; If Conditions Warrant, Modify Design and Return to Block 10

Next, lateral analyses are performed to establish the required minimum pile penetration depth for lateral loading and to evaluate pile deflection and structural resistance for the applied limit state loads. A minimum required pile penetration depth was established to satisfy the nominal geotechnical resistance requirements in axial tension in Block 13. A deeper minimum required pile penetration depth for lateral loading can result based on the combination of factored lateral loads and structural resistances, or deflection limits. Excessive deflections and moments develop at relatively short pile lengths, where a depth to fixity is not achieved. Furthermore, the structural resistance of pile sections must be evaluated based upon the axial, lateral and moment loads. Factored structural resistances were presented in Table D-3. A lateral deformation limit of 1 inch was established as a global performance requirement in Block 1 and confirmed in Block 4 as the design progressed.

The soil profile at the North Abutment was presented in Figure D-7. For lateral load analyses, appropriate p-y models must be selected for each soil layer. The input parameters necessary for lateral load analysis using the LPILE computer program are included in the North Abutment soil profile in Figure D-7.

As discussed in Section 7.3.7.6, p-multipliers are applied to the p-y curves to model pile group behavior. The p-multipliers depend on the center to center pile spacing within the pile group. For all group configurations at the North Abutment, the pile spacing in the longitudinal direction is 4 feet. Therefore, per Section 7.3.7.6 and AASHTO (2014) design specifications, interpolation was used to determine p-multipliers for a pile spacing of 4b. In this case, the front row p-multiplier is 0.90, while the second row is 0.625.

Cyclic loading was analyzed for both rows using LPILE's Load Type 2 option, which uses Shear and Slope to model a fixed head condition. Using the limit state loads at this abutment, lateral analyses in the longitudinal (y-direction) were performed about the pile section's strong axis. Figure D-24 shows the pile orientation within the trial pile cap design.

The following calculation is presented for the HP 12x74 H-pile section using a range of factored axial and lateral loads and the Group Configuration 1. Tables D-20 and D-21 provide LPILE output summaries at the pile head for both rows considering a

pile penetration depth for lateral loading of 45 feet. The pile head is assumed to terminate at the ground surface (i.e., no stickup).

Table D-20 LPILE Summary Output at Pile Head for Front Row, $p_m=0.90$

Load Case	Load Type No.	Pile-Head Condition 1 V (kips)	Pile-Head Condition 2 S (rad)	Axial Load (kips)	Pile-Head Deflection (inches)	Maximum Moment in Pile (kip-ft)	Maximum Shear in Pile (kips)	Pile-Head Rotation (radians)
1	2	0	0	323	0.000	0.0	0	0
2	2	20	0	323	0.249	-95.5	20	0
3	2	30	0	323	0.396	-148.4	30	0
4	2	40	0	323	0.553	-203.1	40	0
5	2	45	0	323	0.640	-232.2	45	0
6	2	47	0	323	0.677	-244.3	47	0
7	2	50	0	323	0.736	-262.7	50	0
8	2	55	0	323	0.841	-294.5	55	0
9	2	60	0	323	0.953	-327.3	60	0

Table D-21 LPILE Summary Output at Pile Head for Second Row, $p_m=0.625$

Load Case	Load Type No.	Pile-Head Condition 1 V (kips)	Pile-Head Condition 2 S (rad)	Axial Load (kips)	Pile-Head Deflection (inches)	Maximum Moment in Pile (kip-ft)	Maximum Shear in Pile (kips)	Pile-Head Rotation (radians)
1	2	0	0	323	0.000	0.0	0	0
2	2	20	0	323	0.320	-104.7	20	0
3	2	30	0	323	0.508	-162.3	30	0
4	2	40	0	323	0.727	-225.1	40	0
5	2	45	0	323	0.857	-259.3	45	0
6	2	47	0	323	0.910	-273.2	47	0
7	2	50	0	323	0.998	-294.7	50	0
8	2	55	0	323	1.151	-331.4	55	0
9	2	60	0	323	1.315	-369.3	60	0

The pile group deflection can be estimated from the above LPILE's deflection results for the front and second rows. The factored load versus pile head deflection for each row is plotted in Figure D-27 along with the group average. The average lateral load per pile for a given group deflection is shown. A step by step discussion of this procedure is provided in Section 7.3.7.6.

The rigid cap method assumes piles move together, and therefore experience the same shear and lateral load. Accordingly, at the resulting factored lateral load per pile, V_y , of 47 kips (Table D-14), the estimated lateral group deflection at the pile head is determined as 0.80 inches. This lateral deflection is less than the 1 inch tolerance based upon project design requirements.

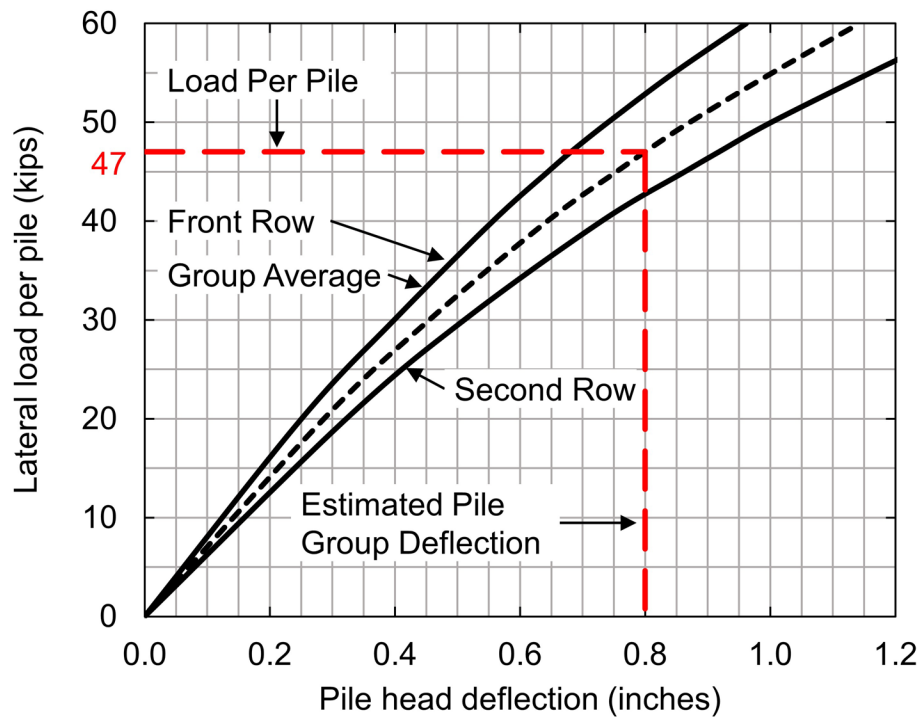


Figure D-27 Factored load versus deflection in longitudinal direction for HP 12x74 at North Abutment.

In addition to the lateral deflection limit, stresses from the resulting bending moment and shear must be evaluated to check that the pile section does not fail structurally. Using the LPILE tabular results, Figure D-28 plots the front row bending moment versus depth for the front row deflection of 0.8 inches.

Figure D-29 plots the maximum bending moment versus pile head deflection for both the front and back rows, however only the maximum bending moment for the front row is used as a “worst case” evaluation of the structural resistance in combined axial compression and flexure. As illustrated in Figure D-29, at the estimated pile head deflection of 0.80 inches, the maximum bending moment, M_{ly} , is 280 kip-ft.

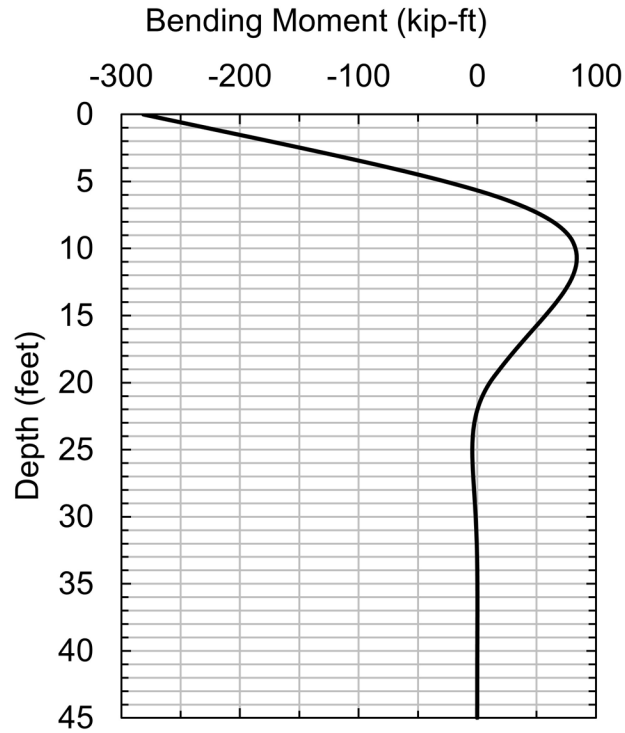


Figure D-28 Front row bending moment versus depth in longitudinal direction.

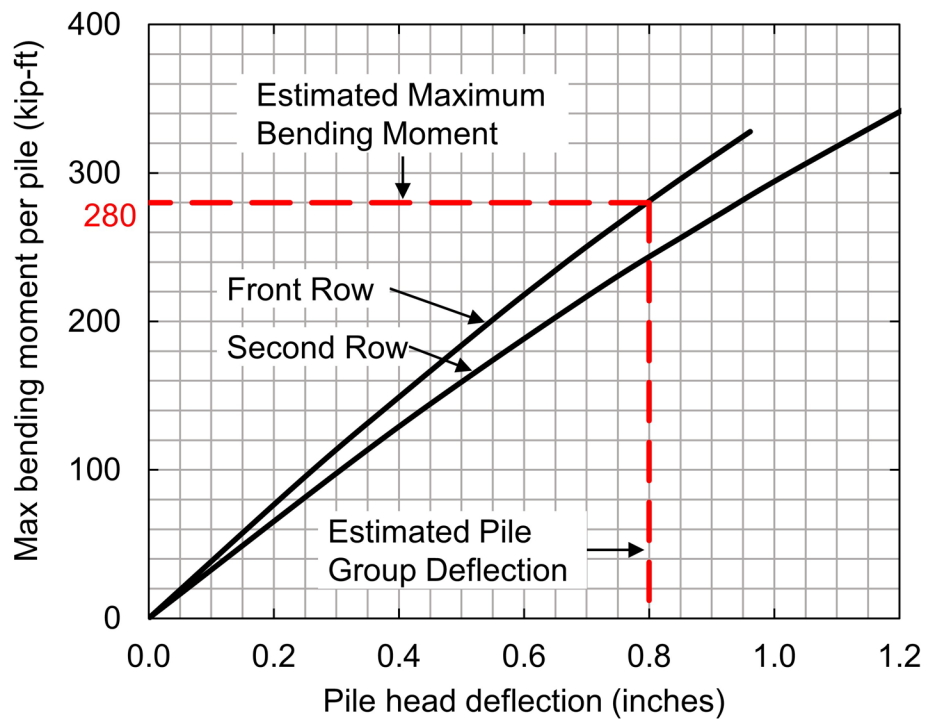


Figure D-29 Bending moment versus deflection in longitudinal direction for HP 12x74 at North Abutment.

Equation 8-58 must be satisfied for the factored axial compression load and moment in the pile. Using results of the lateral analysis, the factored structural resistance was evaluated at the pile head using the factored axial compression load and maximum bending moment (determined using factored loads). The factored structural resistances were determined and are summarized in Table D-3.

Equation 8-58 must be satisfied for the pile section to be acceptable.

P_u = factored axial load, 323 kips (Table D-14).

P_r = factored axial resistance, 762 kips (Table D-3).

M_{ux} = factored moment about x-axis, 0 kip-ft (Block 11).

M_{rx} = factored flexural resistance about x-axis, 118 kip-ft (Table D-3).

M_{uy} = factored moment about y-axis, 280 kip-ft (Figure D-29).

M_{ry} = factored flexural resistance about y-axis, 433 kip-ft (Table D-3).

$$\frac{P_u}{P_r} + \frac{8.0}{9.0} \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \right) \leq 1.0 \quad [\text{Eq. 8-58}]$$

$$\frac{323 \text{ kips}}{762 \text{ kips}} + \frac{8.0}{9.0} \left(\frac{0}{118} + \frac{280}{433} \right) \leq 1.0$$

$$0.998 \leq 1.0$$

The maximum shear from factored lateral loading was then compared to the factored shear resistance from Table D-3. Based on the factored loads, the factored shear resistance is acceptable.

V_r = factored shear resistance, 214 kips (HP 12x74, Table D-3).

V_{uy} = factored shear load, 47 kips.

$$V_{uy} < V_r$$

$$47 \text{ kips} < 214 \text{ kips}$$

The lateral analysis was also performed for each candidate pile section and the deflection and factored structural resistance was subsequently evaluated considering the factored loads for the group configurations shown in Table D-14.

Pile head deflection must be limited to 1 inch, and based upon the applied loads and pile section, the factored structural resistance of the pile must also satisfy the

structural resistance interaction equation presented as Equation 8-58. Pile sections satisfying both criteria were deemed acceptable. Furthermore, as presented in Table D-22, a minimum pile penetration depth was established based on the lateral load analyses. The minimum penetration depth is identified as “- -” for candidate pile sections not meeting the lateral deformation or structural resistance requirements.

Several of the larger pile sections provided sufficient stiffness to resist the applied loads, while smaller, less stiff sections did not (they failed the structural resistance check in Equation 8-58). Factored axial compression loads, in combination with moments caused by factored lateral loads, resulted in some sections' factored structural resistance being exceeded. As a direct result of this analysis, the HP10x42 pile section is unsuitable and eliminated as a candidate pile section for final design. However for the remainder of this example problem, this pile section will still be carried forward for illustrative purposes. Likewise, the HP 12x53 pile section is unsuitable and eliminated as a candidate pile section for Group Configurations 1 and 2.

Table D-22 Established Minimum Pile Penetration Depth Required for Lateral Loading at the North Abutment

Group Configuration	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
1	- - -	- - -	25	10	10
2	- - -	- - -	10	10	10
3	- - -	20	10	10	10

D.15 Block 15: North Abutment – Establish Pile Penetration Depths that Satisfy Tolerable Deformations; Estimate Group Settlement over the Minimum and Maximum Range of Pile Penetration Depths From Blocks 12 through 14 and Identify All Pile Toe Elevations Which Result in Intolerable Deformations; If Conditions Warrant, Modify Design and Return to Block 10

For the cohesionless soils at the North Abutment, pile group settlement was estimated using two methods, the Meyerhof (1976) Method and the Janbu tangent modulus method. Ideally, the settlement method chosen by the designer is one that has shown good correlation with observed results. The pile group settlement at the North Abutment was first calculated using the Meyerhof method. The Meyerhof approach is a traditional settlement estimation method for pile groups in cohesionless soils and one that is contained in the AASHTO (2014) design specifications. However, the soil conditions across the bridge substructure locations are quite variable and a settlement method that could be used at all substructure locations was also desired. Therefore, group settlement was also computed with the Janbu tangent modulus approach using an equivalent footing placed at the neutral plane.

The Meyerhof (1976) approach as presented in Section 7.3.5.2.1 was used to estimate group settlement. The settlement calculations were performed using only unfactored permanent loads. Therefore the loads from the Service I limit state without live load were used to estimate settlement (load factor of 1.0 on permanent loads). The average contact stress for the trial pile group was calculated using the vertical load, Q , and pile group area $Z \times B$. The length of the pile group in Group Configurations 1 and 2 is 41 feet. Therefore, the following calculation is only suitable for these two group configurations. Both the group length, Z , and width, B , were calculated from exterior pile edge to exterior pile edge.

Calculate the average contact stress, p_f , from the trial pile group.

- B = pile group width in longitudinal direction, 5 feet.
- Z = pile group length in transverse direction, 41 feet.
- Q = unfactored permanent load, 1540 kips.

$$p_f = \frac{Q}{B*Z}$$

$$p_f = \frac{1540 \text{ kips}}{(5 \text{ feet})*(41 \text{ feet})} = 7.512 \text{ ksf}$$

$$p_f = 7.512 \text{ ksf}$$

The group embedment influence factor, I_f , is determined based upon the pile group width and estimated pile depth using Equation 7-52.

D_B = pile embedded length into bearing stratum. In this calculation, 5 feet into dense gravel with sand (embedded length $D = 50$ feet).

D' = effective depth, $2/3 * D_B = 3.33$ feet.

$$I_f = 1 - \frac{D'}{8B} \geq 0.5 \quad [\text{Eq. 7-52}]$$

$$I_f = 1 - \frac{3.33 \text{ feet}}{8*(5 \text{ feet})} \geq 0.5$$

$$I_f = 0.92 \geq 0.5$$

$$I_f = 0.92$$

The total settlement is conservatively estimated using Equation 7-50.

$N_{1(60)}$ = average corrected SPT N value within a depth B below pile toe, 59 bpf.

$$S = \frac{4 p_f I_f \sqrt{B}}{N_{1(60)}} \quad [\text{Eq. 7-50}]$$

$$S = \frac{4*(7.512 \text{ ksf})*(0.92)*\sqrt{5 \text{ feet}}}{(59 \text{ bpf})}$$

$$S = 1.04 \text{ inches}$$

The above analysis was performed for additional pile penetration depths and for the pile group dimensions of all three group configurations. Table D-23 summarizes the analysis results for the Meyerhof (1976) estimated group settlement. The estimated group settlement depends upon the $N_{1(60)}$ value of the soil into which the pile group is embedded. In particular, at the contact of the medium dense sand and dense gravel with sand layers, this effect is magnified. This is a limitation of the Meyerhof settlement method, in which soil layers below a depth of B, the pile group width,

below the pile toe are effectively disregarded. The Engineer should consider this when assessing the minimum pile penetration depth for settlement considerations.

Table D-23 Estimated Pile Group Settlement Using Meyerhof (1976) Method For All Pile Group Configurations.

Pile Toe Elevation (feet)	Pile Penetration Depth (feet)	D_B (feet)	$N_{1(60)}$ for Bearing Stratum (bpf)	Estimated Settlement Group Configuration 1 & 2 (inches)	Estimated Settlement Group Configuration 3 (inches)
270.0	40.0	20.0	17	2.63	2.92
267.5	42.5	22.5	38	1.11	1.22
264.5	45.5	0.5	59	1.13	1.25
260.0	50.0	5.0	59	1.04	1.16
255.0	55.0	10.0	59	0.95	1.05

Note: D_B = embedded pile length into bearing stratum.

The Meyerhof (1976) approach does not include or consider the increase in vertical effective stress from embankment loading. Depending on the embankment construction and pile installation timeline, consideration of the stress increase from embankment construction on pile group settlement estimates may or may not be appropriate. Embankment loading effects were included in the second settlement computation method performed using the neutral plane method and Janbu tangent modulus. For simplification, the vertical effective stress increase is determined by treating the embankment surcharge as a strip load. Figure D-30 demonstrates this concept and defines symbols, while the change in vertical effective stress with depth is determined using Equations D-2 through D-4.

$$\Delta\sigma_v = \frac{q}{\pi} [\beta + \sin(\beta) * \cos(\beta + 2\delta)] \quad \text{Eq. D-2}$$

Where:

$$\beta = \tan^{-1} \left(\frac{x+b}{z} \right) - \tan^{-1} \left(\frac{x-b}{z} \right) \quad \text{Eq. D-3}$$

and

$$\delta = \tan^{-1} \left(\frac{x-b}{z} \right) \quad \text{Eq. D-4}$$

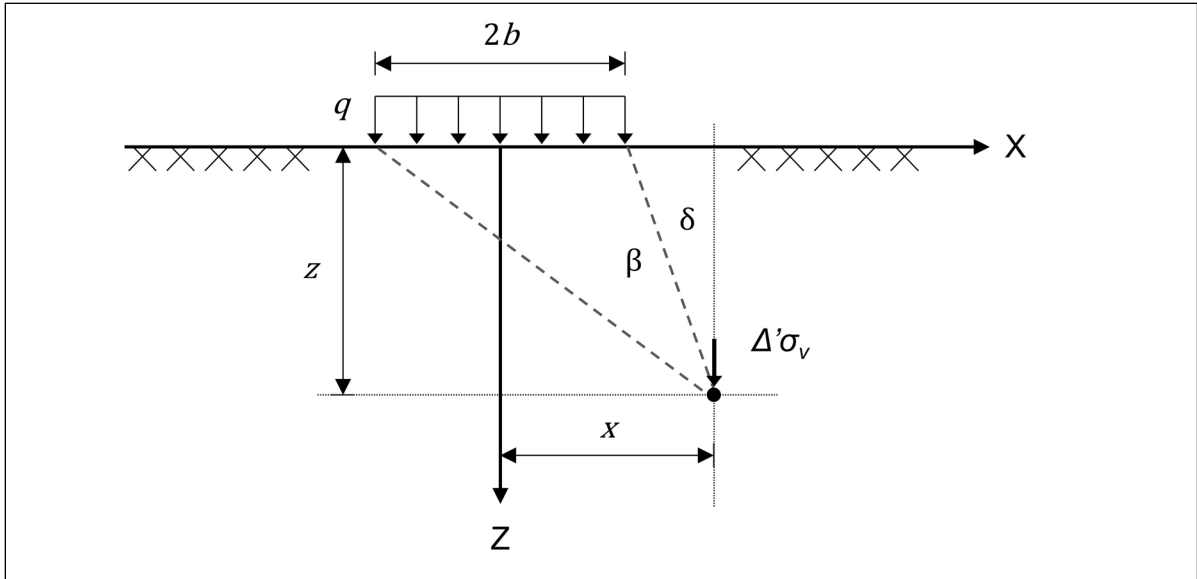


Figure D-30 Vertical effective stress increase due to strip load.

The 10 foot high embankment, with a soil unit weight 120 pcf, results in a surcharge stress at the embankment base of 1.2 ksf, and is assumed to extend 100 feet behind the abutment. Fill directly above the footing is already included in design as a permanent vertical load, EV, and therefore the embankment surcharge is assumed to act as a strip load beginning at the footing edge. The change in vertical effective stress from the embankment surcharge, $\Delta\sigma'_{v(e)}$, is determined under the footing centerline as depicted in Figure D-31.

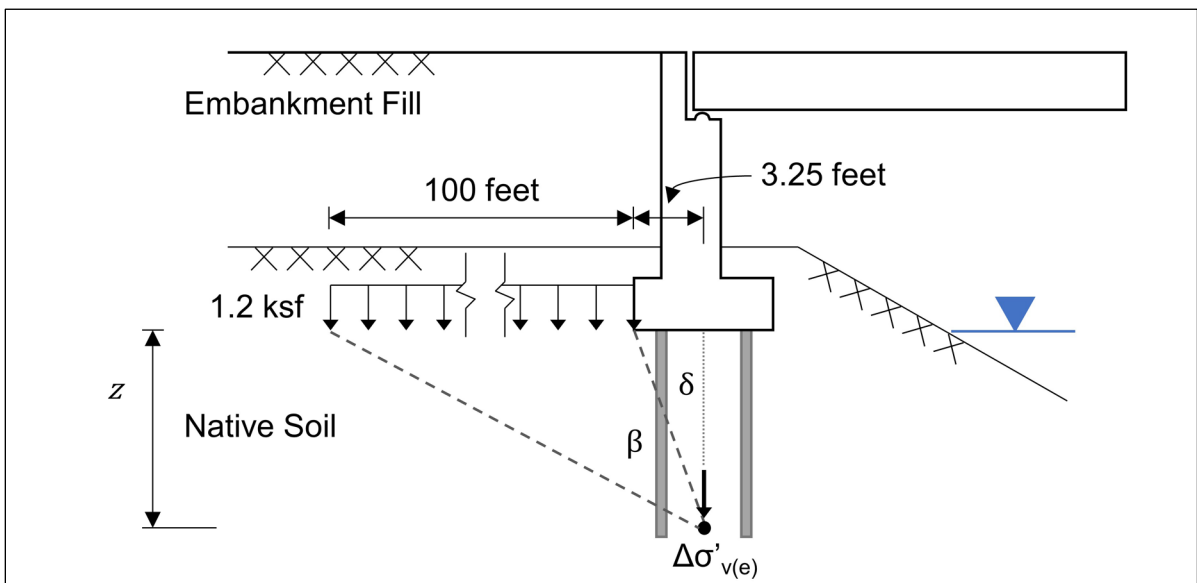


Figure D-31 Vertical effective stress increase due to embankment surcharge.

An example calculation is shown for a depth below footing, z , of 2.5 feet. Complete calculations were performed using a spreadsheet and are summarized in Table D-24. The bottom of the North Abutment footing is at Elevation 310 feet and therefore although the value of z in the following series of analyses may vary based upon the equivalent footing or neutral plane location. Elevation is used to provide a better comparison of effective stress change with depth.

Determine geometry of profile.

$$x = 3.25 \text{ feet} + \frac{100 \text{ feet}}{2} = 53.25 \text{ feet}$$

$$b = \frac{100 \text{ feet}}{2} = 50 \text{ feet}$$

Determine angle β .

$$\beta = \tan^{-1}\left(\frac{x+b}{z}\right) - \tan^{-1}\left(\frac{x-b}{z}\right) \quad [\text{Eq. D-3}]$$

$$\beta = \tan^{-1}\left(\frac{53.25 \text{ feet} + 50 \text{ feet}}{2.1 \text{ feet}}\right) - \tan^{-1}\left(\frac{53.25 \text{ feet} - 50 \text{ feet}}{2.1 \text{ feet}}\right)$$

$$\beta = 0.63$$

Determine angle δ .

$$\delta = \tan^{-1}\left(\frac{x-b}{z}\right) \quad [\text{Eq. D-4}]$$

$$\delta = \tan^{-1}\left(\frac{53.25 \text{ feet} - 50 \text{ feet}}{2.5 \text{ feet}}\right)$$

$$\delta = 0.92$$

Calculate the change in vertical effective stress due to embankment surcharge.

q = stress per unit length, 1.2 ksf.

$$\Delta\sigma'_{v(e)} = \frac{q}{\pi} [\beta + \sin(\beta) * \cos(\beta + 2\delta)] \quad [\text{Eq. D-2}]$$

$$\Delta\sigma'_{v(e)} = \frac{1.2 \text{ ksf}}{\pi} [0.63 + \sin(0.63) * \cos(0.63 + 2 * (0.92))]$$

$$\Delta\sigma'_{v(e)} = 0.07 \text{ ksf}$$

Table D-24 Vertical Effective Stress Increase from Embankment Surcharge

Elevation (feet)	Soil Layer	z (feet)	δ	β	$\Delta\sigma'_{v(e)}$ (ksf)
309.99	1	0.01	1.57	0.00	0.00
307.5	1	2.5	0.92	0.63	0.07
302.5	1	7.5	0.41	1.09	0.30
297.5	1	12.5	0.25	1.20	0.41
292.5	1	17.5	0.18	1.22	0.46
287.5	2	22.5	0.14	1.21	0.49
282.5	2	27.5	0.12	1.19	0.51
277.5	2	32.5	0.10	1.17	0.52
272.5	2	37.5	0.09	1.14	0.52
267.5	2	42.5	0.08	1.10	0.53
262.5	3	47.5	0.07	1.07	0.53
257.5	3	52.5	0.06	1.04	0.53
252.5	3	57.5	0.06	1.01	0.53
247.5	3	62.5	0.05	0.97	0.52
242.5	3	67.5	0.05	0.94	0.52
237.5	3	72.5	0.04	0.91	0.51
232.5	3	77.5	0.04	0.88	0.51
227.5	3	82.5	0.04	0.86	0.50
222.5	3	87.5	0.04	0.83	0.49
219.0	3	91.0	0.04	0.81	0.49

Note, the designer should determine if the construction schedule can accommodate the required time for embankment induced settlements to occur before the start of pile driving and superstructure construction. This example calculation assumes construction cannot be delayed, and therefore, the stress increase from embankment construction and foundations loads are applied concurrently.

Pile group settlement was calculated using the neutral plane method and Janbu tangent modulus as discussed in Section 7.3.5.6. An equivalent footing, with plan dimensions equal to those of the pile group, was evaluated at increasing pile penetration depths, and the resulting pile group settlement computed using the Janbu tangent modulus. This procedure allowed the shallowest depth of an equivalent footing to be determined that met vertical deformation requirements. The required minimum pile penetration depth was then determined by the pile toe elevation that would place the neutral plane at this same equivalent footing depth where vertical deformation requirements were satisfied.

The following example calculation is performed for an equivalent footing located at Elevation 270 feet corresponding to a pile penetration depth of 60 feet. The length of the pile group in Group Configurations 1 and 2 is 41 feet from exterior pile edge to exterior pile edge. Since Group Configuration 3 has different pile group plan dimensions, this example is suitable for only Configurations 1 and 2. Similar to the vertical effective stress increase calculations from embankment loading, the soil profile was again divided into 5 foot thick layers. The elevation shown in Table D-25 references the midpoint of each respective 5 foot thick soil layer, while z is the depth below the equivalent footing to the midpoint of each respective 5-foot-thick soil layer. Tabulated values for this analysis are recorded in Table D-25.

Considering only the unfactored permanent load acting on the superstructure, Q , calculate the change in vertical effective stress below the equivalent footing, $\Delta\sigma'_{v(ss)}$, at Elev. 267.5 feet.

- Q = unfactored permanent load, 1540 kips (Service I, no LL, Table D-11).
- B = pile group width, 5 feet.
- Z = pile group length, 41 feet (Group Configurations 1 and 2 only).
- z = depth below equivalent footing, 2.5 feet (Elev.267.5 feet).

$$\Delta\sigma'_{v(ss)} = \frac{Q}{(B+z)*(Z+z)} \quad [\text{Eq. 7-55}]$$

$$\Delta\sigma'_{v(ss)} = \frac{(1540 \text{ kips})}{((5 \text{ feet})+(2.5 \text{ feet}))*((41 \text{ feet})+(2.5 \text{ feet}))}$$

$$\Delta\sigma'_{v(ss)} = 4.72 \text{ ksf}$$

Including the change in effective stress from the embankment and superstructure, calculate the new effective stress below the equivalent footing at Elev. 267.5 feet.

- σ'_{v0} = initial vertical effective stress at depth z below the equivalent footing, 3.15 ksf.
- $\Delta\sigma'_{v(e)}$ = change in effective stress at depth z below the equivalent footing from embankment loading, 0.53 ksf (Table D-24).
- $\Delta\sigma'_{v(ss)}$ = change in effective stress at depth z below the equivalent footing from superstructure loading, 4.72 ksf.

$$\sigma'_{1(e+ss)} = \sigma'_{v0} + \Delta\sigma'_{v(ss)} + \Delta\sigma'_{v(e)}$$

$$\sigma'_{1(e+ss)} = (3.15 \text{ ksf}) + (0.53 \text{ ksf}) + (4.72 \text{ ksf})$$

$$\sigma'_{1(e+ss)} = 8.40 \text{ ksf}$$

Determine the stress increase by comparing σ'_{vo} and $\sigma'_{1(ss+e)}$.

$$\text{Stress Increase} = \frac{\sigma'_{1(ss+e)} - \sigma'_{vo}}{\sigma'_{vo}} * 100\% \quad \text{Eq. D-5}$$

$$\text{Stress Increase} = \frac{(8.40 \text{ ksf}) - (3.15 \text{ ksf})}{(3.15 \text{ ksf})} * 100\%$$

$$\text{Stress Increase} = 167\%$$

The stress increase is greater than or equal to 10%. Deformation for this depth increment should be estimated and included in the sum of all depth increments in which the stress increase is not less than 10%. A rock layer, which is considered incompressible, is located at Elev. 218 feet at this abutment, and thus it is assumed that no settlement or compression occurs below this depth.

For the dense coarse grained soil at Elev. 267.5 feet, $z = 2.5$ (stress exponent of $j = 1.0$), determine the strain in the layer from the increase in vertical effective stress, $\Delta\varepsilon$, with Equation 7-61.

- E_s = elastic modulus of soil, 1416 ksf (Figure D-7).
- σ'_{vo} = initial vertical effective stress at depth z below the equivalent footing, 3.15 ksf.
- $\sigma'_{1(ss+e)}$ = new vertical effective stress below the equivalent footing, 8.40 ksf.

$$\Delta\varepsilon = \frac{1}{E_s} [\sigma'_{1(ss+e)} - \sigma'_{vo}] \quad \text{[Eq. 7-61]}$$

$$\Delta\varepsilon = \frac{1}{(1416 \text{ ksf})} [(8.40 \text{ ksf}) - (3.15 \text{ ksf})]$$

$$\Delta\varepsilon = 0.0037$$

Calculate the layer compression denoted, S , with the initial height of the layer, H_o .

$$S = \Delta\varepsilon * H_o$$

$$S = 0.0037 * 5 \text{ feet} * \left(\frac{12 \text{ inches}}{1 \text{ foot}}\right)$$

$$S = 0.22 \text{ inches}$$

Table D-25 Calculation of Settlement by Janbu Tangent Modulus for Equivalent Footing at Elevation 270 feet

Elev. (feet)	z (feet)	H _o (feet)	E _s (ksf)	σ' _{vo} (ksf)	B (feet)	Z (feet)	Δσ' _{v(ss)} (ksf)	Δσ' _{v(e)} (ksf)	σ' _{1(ss+e)} (ksf)	Stress Increase (%)	Δε	S (in)
267.5	2.5	5	1416	3.15	7.5	43.5	4.72	0.53	8.40	167	0.0037	0.22
262.5	7.5	5	1416	3.46	12.5	48.5	2.54	0.53	6.52	89	0.0022	0.13
257.5	12.5	5	1416	3.77	17.5	53.5	1.64	0.53	5.94	58	0.0015	0.09
252.5	17.5	5	1416	4.08	22.5	58.5	1.17	0.53	5.78	42	0.0012	0.07
247.5	22.5	5	1416	4.40	27.5	63.5	0.88	0.52	5.80	32	0.0010	0.06
242.5	27.5	5	1416	4.71	32.5	68.5	0.69	0.52	5.92	26	0.0009	0.05
237.5	32.5	5	1416	5.02	37.5	73.5	0.56	0.51	6.09	21	0.0008	0.05
232.5	37.5	5	1416	5.33	42.5	78.5	0.46	0.51	6.30	18	0.0007	0.04
227.5	42.5	5	1416	5.65	47.5	83.5	0.39	0.50	6.53	16	0.0006	0.04
222.5	47.5	5	1416	5.96	52.5	88.5	0.33	0.49	6.78	14	0.0006	0.03
219.0	51.0	2	1416	6.27	56.0	92.0	0.30	0.49	7.06	13	0.0006	0.01
											Total:	0.80

In a similar manner, the above analysis was performed for additional equivalent footing locations for all three trial group configurations. Table D-26 summarizes the results for the neutral plane method and Janbu tangent modulus estimated settlement. As established by global project performance requirements, vertical deformation (including settlement and elastic pile compression) should be limited to 1.5 inches at each substructure location. Therefore, Table D-26 indicates that this requires that the equivalent footing be located at Elevation 270 feet, a minimum of 40 feet below the bottom of pile cap (Elev. 310).

It is assumed that the equivalent footing acts at the same location as the neutral plane. Accordingly, an analysis was performed to determine the pile toe elevation

necessary to locate the neutral plane at Elev. 270.0, thereby establishing the minimum required pile penetration depth to satisfy tolerable deformations.

Table D-26 Estimated Pile Group Settlement Using Janbu Tangent Modulus with Neutral Plane Method For All Pile Group Configurations.

Equivalent Footing Elevation (feet)	Equivalent Footing Depth (feet)	Estimated Settlement Group Configurations 1 and 2 (inches)	Estimated Settlement Group Configuration 3 (inches)
285.0	25	2.30	2.46
280.0	30	1.98	2.12
275.0	35	1.54	1.65
270.0	40	0.80	0.85
265.0	45	0.76	0.81

The location of the neutral plane and the magnitude of drag force are evaluated following the procedure outlined in Section 7.3.6, using unfactored permanent loads and nominal geotechnical resistance. Because load factors for the Service limit state are 1.0, applicable loads at this limit state may be considered unfactored. The Service I, without LL limit state loads are therefore used for evaluation for the neutral plane. This example again utilizes the load for Group Configuration 1 (Q= 201 kips, Table D-14). Figure D-32 presents a graphical interpretation of the neutral plane for the HP12x74 pile driven to a penetration depth of 60 feet.

First, the sustained load plus the cumulative shaft resistance versus depth is plotted. Next, the mobilized toe resistance minus the cumulative shaft resistance versus pile penetration depth is plotted. The exact percentage of toe mobilization is unknown at this stage of design, therefore multiple toe mobilization curves should be evaluated to determine the neutral plane location. The 0% toe mobilization curve is the most conservative location to evaluate pile settlement since it locates the neutral plane at the highest elevation. The 100% toe mobilization curve should be used to check the pile section's structural strength since it results in the greatest axial force in the pile. The structural strength check is performed in Block 17.

At the North Abutment, it is expected that piles will be supported partially by toe resistance. Therefore the 0% toe mobilization curve presents an unreasonable baseline to evaluate settlement in this case. With some toe resistance likely mobilized, the 50 percent toe mobilization curve is used to evaluate settlement.

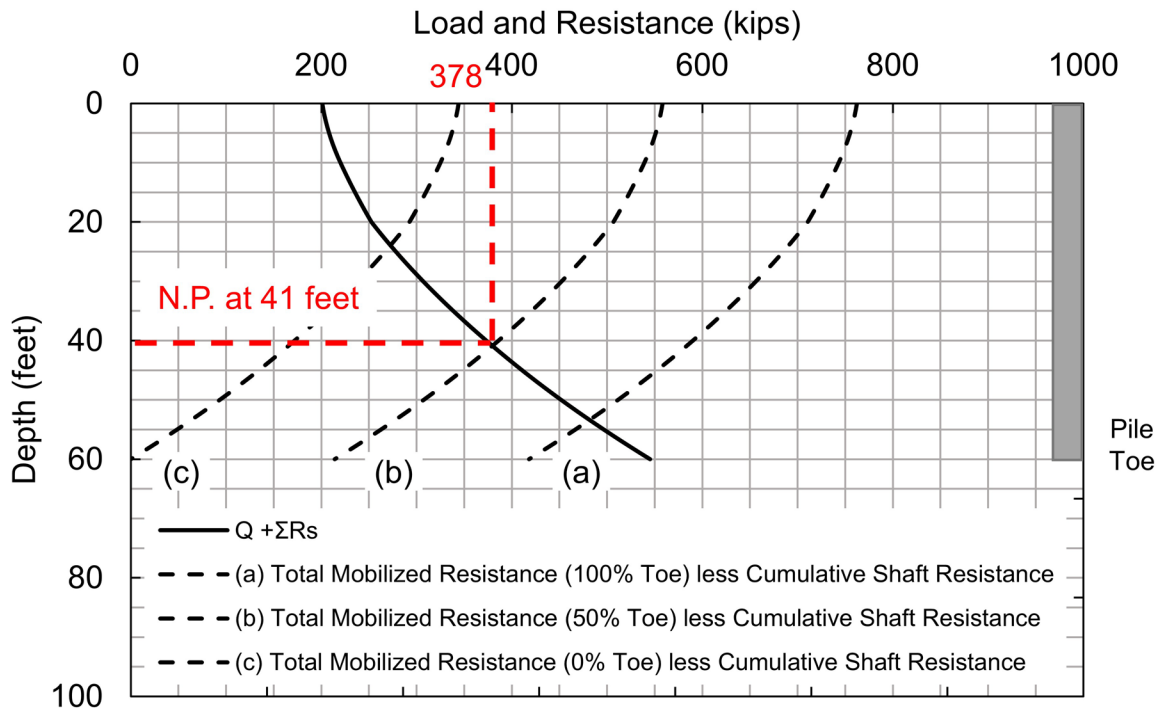


Figure D-32 Neutral plane at 50% toe mobilization for HP 12x74 at the North Abutment.

This analysis procedure was also performed for the remaining candidate pile sections and trial group configurations to determine the pile penetration depth required for the neutral plane to be located below a depth of 40 feet (Elev. 270.0 feet). Table D-27 summarizes the analysis results. The load per pile for each group configuration was previously presented in Table D-14.

Table D-27 Pile Penetration Depth Required to Locate Neutral Plane at Elev. 270.0 feet

Group Configuration	Load per Pile Q (kips)	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
1	201	69	60	60	54	53
2	165	64	55	55	50	50
3	139	60	52	52	48	48

A comparison of the required pile penetration depth for both settlement estimation methods yielded dissimilar results. To limit total settlement and elastic compression to less than 1.5 inches, the Meyerhof Method requires a pile penetration depth of 42.5 feet (Elev. 267.5 feet) for a HP 12x74 in Group Configuration 1. The Janbu tangent modulus approach requires a 17 foot deeper pile to satisfy the same settlement constraints with the same pile and group configuration. Table D-23 presents the minimum pile penetration depth based on group settlement estimation using the Meyerhof (1976) approach. Table D-27 presents the minimum pile penetration depth based on group settlement estimation using the Janbu Tangent modulus Method with the equivalent footing located at the neutral plane. Based on placing the neutral plane at or below EL 270, all candidate pile sections and group configurations require the piles to be driven into Soil Layer 3 (dense gravel with sand). After considering both settlement estimates, the results of the neutral plane and Janbu tangent modulus method were used to establish the minimum pile penetration depth for tolerable deformations.

A review of the soil profile at the North Abutment in Figure D-7 indicates no compressible soil layers below Soil Layer 3. Therefore, no effort is necessary to identify a maximum pile penetration depth to prevent punching through a dense layer into an unsuitable soil layer, or to causing a stress increase on a lower compressible layer causing excessive settlement. Table D-28 presents the established minimum pile penetration depths to satisfy tolerable deformations at the North Abutment using the Janbu tangent modulus approach.

Table D-28 Established Minimum Pile Penetration Depths to Satisfy Tolerable Deformations at the North Abutment

Group Configuration	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
1	69	60	60	54	53
2	64	55	55	50	50
3	60	52	52	48	48

Elastic shortening of the pile should be considered along with settlement. For elastic compression, the load per pile from the Service I, without LL limit state is applied at the pile head. As shown in Table D-14, this load is 201 kips.

Note that the drag force from negative shaft resistance increases the axial compression force in the pile. Negative shaft resistance above the neutral plane acts to increase axial compression force in the pile, whereas below the neutral

plane, positive shaft resistance reduces the axial compression force in the pile. This effect must be accounted for in the elastic compression calculation. Accordingly, the unfactored axial load used to compute elastic compression, Q , changes for each pile segment length, increasing equal to the unfactored permanent load plus the shaft resistance down to the neutral plane. In this example, at and below the neutral plane location of 54 feet, the unfactored axial load is equal to the resistance distribution from 100% toe mobilization. A review of Figure D-29 indicates that the maximum drag force magnitude results from 100% toe mobilization.

Equation 7-48 is used to illustrate this example for the first 12 inch increment of the HP 12x74 pile section. The average shaft resistance and average load for each respective depth interval is used to estimate the elastic compression. For each 12 inch segment, the elastic modulus remains constant, and was evaluated as 29,000 ksi. The pile cross sectional area likewise remains constant as 21.8 in². Remaining calculations were performed using a spreadsheet; Table D-29 summarizes the elastic compression with depth.

Determine the unfactored axial load in segment.

- Q_d = unfactored permanent load, 201 kips.
- R_s^- = average (negative) shaft resistance, 0.5 kips.

$$Q = Q_d + R_s^- = 201 \text{ kips} + 0.5 \text{ kips}$$

$$Q = 201.5 \text{ kips}$$

Calculate elastic compression of segment with unfactored axial load from combined unfactored permanent load and negative shaft resistance.

- L = segment length, 12 inches.
- A = cross sectional area of pile material, 21.8 in².
- E = elastic modulus of pile, 29,000 ksi.

$$\Delta = \frac{QL}{AE} \quad \text{[Eq. 7-48]}$$

$$\Delta = \frac{(201.5 \text{ kips}) * (12 \text{ inches})}{(21.8 \text{ in}^2) * (29,000 \text{ ksi})}$$

$$\Delta = 0.00383 \text{ inches}$$

Table D-29 Elastic Compression Calculation

Depth Below Pile Head (feet)	Average Shaft Resistance (kips)	Average Unfactored Axial Load (kips)	Segment Compression, Δ (inches)
0	0.0	201.0	0.00000
0-1	0.5	201.5	0.00383
1-2	1.7	202.7	0.00385
2-3	3.0	204.0	0.00387
3-4	4.5	205.5	0.00390
4-5	6.3	207.3	0.00393
5-6	8.2	209.2	0.00397
6-7	10.3	211.3	0.00401
7-8	12.5	213.5	0.00405
8-9	15.0	216.0	0.00410
9-10	17.7	218.7	0.00415
10-11	20.5	221.5	0.00421
11-12	23.5	224.5	0.00426
12-13	26.5	227.5	0.00432
13-14	29.6	230.6	0.00438
14-15	32.8	233.8	0.00444
15-16	36.0	237.0	0.00450
16-17	39.3	240.3	0.00456
17-18	42.7	243.7	0.00463
18-19	46.2	247.2	0.00469
19-20	49.8	250.8	0.00476
20-21	54.0	255.0	0.00484
21-22	59.0	260.0	0.00493
22-23	64.0	265.0	0.00503
23-24	69.2	270.2	0.00513
24-25	74.5	275.5	0.00523
25-26	80.0	281.0	0.00533
26-27	85.5	286.5	0.00544
27-28	91.2	292.2	0.00555
28-29	97.0	298.0	0.00566
29-30	102.9	303.9	0.00577
30-31	108.9	309.9	0.00588
31-32	115.0	316.0	0.00600
32-33	121.3	322.3	0.00612
33-34	127.7	328.7	0.00624
34-35	134.2	335.2	0.00636
35-36	140.8	341.8	0.00649
36-37	147.6	348.6	0.00662
37-38	154.4	355.4	0.00675
38-39	161.4	362.4	0.00688

Table D-29 Elastic Compression Calculation (continued)

Depth Below Pile Head (feet)	Average Shaft Resistance (kips)	Average Unfactored Axial Load (kips)	Segment Compression, Δ (inches)
39-40	168.5	369.5	0.00701
40-41	175.8	376.8	0.00715
41-42	183.1	384.1	0.00729
42-43	190.6	391.6	0.00743
43-44	198.2	399.2	0.00758
44-45	205.9	406.9	0.00772
45-46	213.7	414.7	0.00787
46-47	221.7	422.7	0.00802
47-48	229.8	430.8	0.00818
48-49	238.1	439.1	0.00833
49-50	246.5	447.5	0.00849
50-51	255.1	456.1	0.00866
51-52	263.8	464.8	0.00882
52-53	272.7	473.7	0.00899
53-54	281.7	482.7	0.00916
54-55	290.9	482.0	0.00915
55-56	300.2	472.0	0.00896
56-57	309.7	462.5	0.00878
57-58	319.3	452.8	0.00860
58-59	329.1	443.1	0.00841
59-60	339.1	433.1	0.00822
		Total	0.37

For the pile head load of 201 kips (Group Configuration 1 loads), estimated elastic compression of the HP 12x74 pile section driven to 60 feet is 0.37 inches (pile toe at Elev. 250 feet). Combined with 0.80 inches of deformation from settlement (Table D-28), it is estimated that total vertical deformation at the North Abutment is 1.17 inches.

D.16 Block 16: North Abutment – Check pile drivability to maximum pile penetration depth requirements established in Blocks 12 through 15

Preliminary pile drivability analyses were performed for the 5 candidate pile sections in Block 10. Plots of nominal driving resistance, blow count, and compression stress versus depth were presented in Figure D-22. This figure should now be reviewed considering the established or estimated minimum pile penetration depths. A candidate pile section must be capable of being driven to the penetration depth

necessary to achieve the nominal geotechnical resistance in axial compression and tension, and to a penetration depth necessary to satisfy lateral load demands as well as axial and lateral deformation requirements. Estimated or minimum pile penetration depths were previously established in Blocks 12, 13, 14, and 15 and summarized in Tables D-15, D-19, D-22 and D-28.

Although a minimum pile penetration depth is not typically established for nominal geotechnical resistance in axial compression, the pile should also be capable of being driven reasonably close to the estimated pile penetration depth where the nominal resistance is expected to develop. If the pile cannot be driven to the required depth within driving stress limits and at reasonable blow counts, a larger pile hammer, a pile section with greater impedance, or pile installation aids such as predrilling or jetting may be required to satisfy or improve drivability. Alternatively, substructure design modifications should be considered.

For the candidate HP 12x74 H-pile section in the Group Configuration 1, the pile penetration depth for axial compression loading was estimated at 45 feet (Table D-15), the minimum penetration depth for axial tension loading was 28 feet (Table D-19), and the minimum penetration depth for lateral loading was 25 feet (Table D-22). The minimum pile penetration depth for group settlement was 60 feet (Table D-28) based on embankment and foundation loads occurring simultaneously. Accordingly, this candidate pile section must have sufficient drivability to the maximum of these depths (i.e., 60 feet).

A review of Figure D-22 indicates that the HP 12x74 pile section can be driven to 61 feet with a D36-52 hammer before encountering practical refusal. The preliminary drivability evaluation (with soil and hammer model assumptions described in Block 10), estimated that the blow count will not exceed 120 blows per foot or 10 blows per inch before this pile penetration depth. Compression driving stresses are estimated to remain below driving stress limits. Drivability results for a D46-52 pile hammer were also presented in Figure D-22 and indicated greater pile penetration depths were possible. Therefore, it is concluded that the HP 12x74 pile section can be driven to the 60 foot penetration depth.

D.17 Block 17: North Abutment – Determine the Neutral Plane Location and Resulting Drag Force; Check Structural Strength Limit State for Pile Penetration Depth From Block 16

Previously in Block 15, the neutral plane was evaluated as part of the settlement calculations. The neutral plane and resulting drag force are now evaluated to check

the structural strength limit state for candidate pile sections. Section 7.3.6 provides guidance for evaluating the neutral plane location and the magnitude of the drag force. The Service I, without live load limit state was used for the applied pile head load. This example again utilizes the load for Group Configuration 1 ($Q = 201$ kips, Table D-14).

At 100 percent toe mobilization, the neutral plane is at its lowest potential location, and thus the highest drag force magnitude results. Accordingly, this is the toe mobilization curve that should be used to check the pile section's structural strength. Figure D-33 presents a graphical interpretation of the neutral plane for the HP12x74 pile section driven to the estimated pile penetration depth of 60 feet. In this case the neutral plane is located 54 feet below the pile head with a resulting maximum axial compression force in the pile of 486 kips.

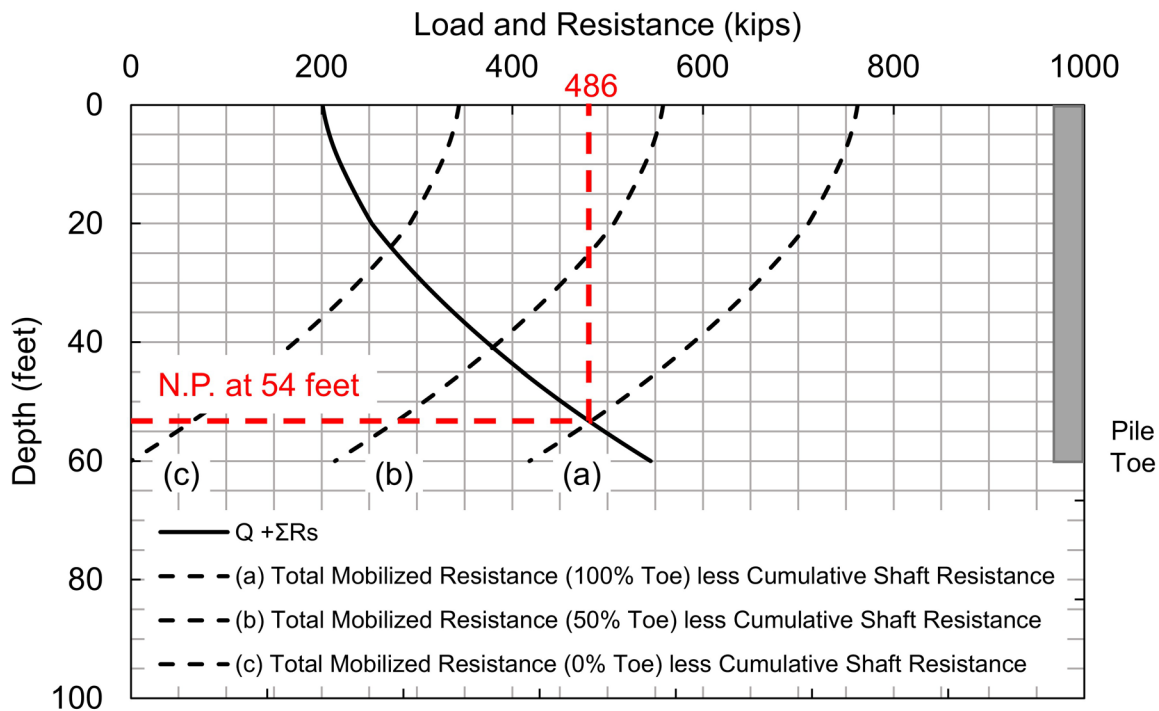


Figure D-33 Neutral plane at 100% toe mobilization for HP 12x74 at the North Abutment.

The resulting unfactored drag force, DF , is the difference between the maximum unfactored axial compression force in the pile, Q_{max} , minus the unfactored permanent load (Q). In this case, the drag force is evaluated for 100 percent toe mobilization.

$$DF = Q_{max} - Q$$

$$DF = (486 \text{ kips}) - (201 \text{ kips}) = 285 \text{ kips}$$

Following this calculation, the factored structural load was determined using Equation 7-70. As discussed in Section 7.3.6, a load factor of 1.25 is applied to the permanent load while a load factor of 1.1 is applied to the drag force. Because the pile is driven into dense gravel with cobbles, the pile toe may be subject to damage during driving, and therefore as recommended by the AASHTO (2104) code, a structural resistance factor for axial compression ϕ_c of 0.50 would be applied to the nominal structural resistance of 1088 kips (Table D-2). Accordingly, the factored structural resistance, P_r , for the HP 12x74 pile section is 544 kips.

$$1.25 (Q) + \gamma_p(DF) < P_r \quad [\text{Eq. 7-70}]$$

$$1.25 (201 \text{ kips}) + 1.1(285 \text{ kips}) = 565 \text{ kips}$$

$$565 \text{ kips} > 544 \text{ kips}$$

In this case, the factored structural resistance is less than the factored loads, and therefore the pile section would be considered unacceptable. This evaluation of the structural resistance does not however consider the location of the neutral plane. In this case, the neutral plane is 6 feet above the pile toe for 100% toe mobilization, and this location on the pile may not be damaged.

The AASHTO (2014) design specifications do not specifically address drag force considerations relative to the pile structural resistance. For example, the factored structural resistance of an H-pile is the nominal structural resistance multiplied by the resistance factor for axial compression, ϕ_c . This resistance factor is 0.70 for combined axial and flexural resistance of undamaged piles, 0.60 for the axial resistance of piles in compression under good driving conditions, and 0.50 for the axial resistance of piles in compression subject to damage due to severe driving conditions. While not equivocally stated, it follows that if the neutral plane is located below the point of fixity and above the depth where H-piles are subject to potential damage during driving, then the sum of the sustained load plus drag force at the neutral plane is limited to 0.70 of the nominal structural resistance.

Hence for piles at the North Abutment, if the neutral plane is located at a depth where the pile section is likely to be straight and undamaged, a structural resistance factor for axial compression, ϕ_c , of 0.70 could be applied to the nominal structural resistance, P_n , of 1088 kips (Table D-2). Accordingly, the factored structural resistance, P_r , for the HP 12x74 pile section is 762 kips. Equation 7-68 is used to evaluate the structural resistance.

$$1.25 (Q) + \gamma_p(DF) < P_r \quad [\text{Eq. 7-70}]$$

$$1.25 (201 \text{ kips}) + 1.1(285 \text{ kips}) = 565 \text{ kips}$$

$$565 \text{ kips} < 762 \text{ kips}$$

In this case, the factored structural resistance is larger than the factored load, and therefore the pile section is acceptable.

It may be beneficial to review the ratio of factored load to nominal structural resistance or, P_u / P_n . The following evaluation serves to back calculate the minimum required structural resistance factor, $\phi_{c(\min)}$, for the section to be acceptable considering the factored load.

$$\phi_{c(\min)} = \frac{P_u}{P_n}$$

$$\phi_{c(\min)} = \frac{(565 \text{ kips})}{(1088 \text{ kips})}$$

$$\phi_{c(\min)} = 0.52$$

To further evaluate the drag force on the HP 12x74 pile section, the ratio of factored load to nominal structural resistance or, P_u / P_n was performed at 1 foot increments over the portion of the pile. Table D-30 presents a calculation of unfactored load in the pile for 50 percent toe mobilization, the drag force, and the factored load at 1 foot increments over the lower 20 feet. The final column shows the ratio of factored load to nominal structural resistance, P_u / P_n . This comparison provides the Engineer with the minimum structural resistance factor required for the pile section to be acceptable. For example, at the pile toe (Elev. 248 feet), the pile section would be acceptable based on a structural resistance factor for axial compression, ϕ_c , of 0.50.

Table D-30 Calculation of Load in Pile, Factored Load and Comparison to Nominal Structural Resistance

Elevation (feet)	Depth on Pile (feet)	Nominal Shaft Resistance (kips)	$Q + \Sigma$ R_s (kips)	Unfactored Load in Pile (kips)	Drag Force DF (kips)	Factored Load P_u (kips)	P_u/P_n $\phi_{c(\min)}$
268.0	40	172	373	373	172	441	0.40
267.0	41	179	380	380	179	449	0.41
266.0	42	187	388	388	187	457	0.42
265.0	43	194	395	395	194	465	0.43
264.0	44	202	403	403	202	473	0.44
263.0	45	210	411	411	210	482	0.44
262.0	46	218	419	419	218	491	0.45
261.0	47	226	427	427	226	499	0.46
260.0	48	234	435	435	234	509	0.47
259.0	49	242	443	443	242	518	0.48
258.0	50	251	452	452	251	527	0.48
257.0	51	259	460	460	259	537	0.49
256.0	52	268	469	469	268	546	0.50
255.0	53	277	478	478	277	556	0.51
254.0	54	286	487	487	286	565	0.52
253.0	55	296	497	477	276	554	0.51
252.0	56	305	506	467	266	544	0.50
251.0	57	314	515	458	257	534	0.49
250.0	58	324	525	448	247	523	0.48
249.0	59	334	535	438	237	512	0.47
248.0	60	344	545	428	227	501	0.46

*Neutral plane located at Depth 54 feet (EL 254.0 feet) for 50 percent toe mobilization.

An analysis of the drag force was performed for each candidate pile section. The pile penetration depth utilized for the drag force structural resistance check was the required minimum penetration depth presented in Table D-28. Table D-31 presents the ratio of the factored load to nominal structural resistance, at the pile toe, for the all the candidate piles and group configurations. For H-piles which may be subject to damage during driving and require pile toe protection (i.e., as for piles driven to bedrock or through dense gravel, cobbles, etc.), the structural resistance factor in axial compression, ϕ_c , is 0.50. Considering 50 percent toe mobilization, only the HP 12x74 and HP 14x117 pile section are acceptable for all group configurations.

Table D-31 Ratio of Factored Load to Nominal Structural Resistance in Axial Compression, $\phi_{c(\min)}$, at the Pile Toe

Group Configuration	HP 10x42 $\phi_{c(\min)}$	HP 12x53 $\phi_{c(\min)}$	HP 12x74 $\phi_{c(\min)}$	HP 14x89 $\phi_{c(\min)}$	HP 14x117 $\phi_{c(\min)}$
1	0.55	0.63	0.46	0.50	0.41
2	0.54	0.63	0.46	0.43	0.35
3	0.54	0.55	0.43	0.38	0.32

The ratio of the factored load to nominal structural resistance was also evaluated at the neutral plane. In this case however, if the neutral plane was located at a depth where the pile section was assumed to be straight and undamaged, a higher resistance factor, ϕ_c , of up to 0.70 was applied. Table D-32 presents the ratio of the factored load to nominal structural resistance at the neutral plane, along with the respective neutral plane depth. For example, on the HP 12x74 pile section in Group Configuration 1, the minimum required structural resistance in axial compression $\phi_{c(\min)}$ is 0.52, at a neutral plane depth of 54 feet. This location is 6 feet above the pile toe, and is assumed undamaged, therefore the higher resistance factor of 0.70 is applied. Accordingly, the pile section is considered acceptable.

Conversely, for the HP 10x42 pile section in Group Configuration 1, the minimum required structural resistance in axial compression $\phi_{c(\min)}$ is 0.74, at a neutral plane depth of 54 feet. This location is 15 feet above the pile toe (69 feet, Table D-32), and is also assumed undamaged. After applying the higher resistance factor of 0.70, the pile section is considered unacceptable (and was also structurally unacceptable based upon the load at the pile toe, Table D-32).

Table D-32 Ratio of Factored Load to Nominal Structural Resistance in Axial Compression, $\phi_{c(\min)}$, at the Neutral Plane

Group Configuration	HP 10x42 $\phi_{c(\min)}$ / NP depth (feet)	HP 12x53 $\phi_{c(\min)}$ / NP depth (feet)	HP 12x74 $\phi_{c(\min)}$ / NP depth (feet)	HP 14x89 $\phi_{c(\min)}$ / NP depth (feet)	HP 14x117 $\phi_{c(\min)}$ / NP depth (feet)
1	0.74 / 54	0.68 / 55	0.52 / 54	0.50 / 54	0.41 / 53
2	0.67 / 54	0.63 / 55	0.47 / 53	0.43 / 50	0.35 / 50
3	0.62 / 54	0.55 / 55	0.43 / 52	0.38 / 48	0.32 / 48

Based on results of the drag force analysis, a candidate pile section may be eliminated from consideration if the factored loads are higher than the factored structural resistance. As recorded in Table D-33, the larger candidate pile sections remained acceptable considering drag force.

Table D-33 Does Candidate Pile Section Meet Structural Resistance Requirement Considering Drag Force Associated with Minimum Pile Penetration Depth?

Group Configuration	HP 10x42	HP 12x53	HP 12x74	HP 14x89	HP 14x117
1	No	No	Yes	Yes	Yes
2	No	No	Yes	Yes	Yes
3	No	No	Yes	Yes	Yes

D.18 Decision Block 18: Does Estimated Total Settlement and Differential Settlement Between Adjacent Substructure Locations Satisfy Requirements and Angular Distortion Limits?

This design step cannot yet be completed. The total settlement at the North Abutment has been estimated and it is within the established deformation limits. However, the foundation design and settlement estimates for the adjacent pier have yet to be performed. Therefore, differential settlement and angular distortion cannot be assessed at this time. This step will be revisited once design computations at the pier are performed.

D.19 Block 19: North Abutment – Evaluate Economics of Candidate Piles, Preliminary Group Configurations, and Other Factors

Until now, the design process has served to compare strength and service limits for several candidate pile types within trial group configurations. Some candidate pile types have not met all of the strength, service, or drivability requirements. It is useful to quickly review the suitable and unsuitable pile types and group configurations and then assess the cost of the viable foundation solutions.

Table D-34 summarizes the established minimum pile penetration depth based on analysis results from Blocks 12 through 15. For all candidate pile sections and group configurations at the North Abutment, the established minimum pile penetration depth was based on meeting tolerable vertical deformations.

Several candidate sections did not meet structural resistance requirements for axial loading, lateral loading or both. The candidate pile sections and/or group configurations not meeting design requirements are identified with an asterisk in Table D-34. For all three group configurations, the HP 10x42 and HP 12x52 pile sections did not meet all structural resistance requirements. These candidate pile sections and group configuration will therefore be eliminated for the final design.

Table D-34 Established Minimum Pile Penetration Depth at the North Abutment

Group Configuration	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
1	69*	60*	60	54	53
2	64*	55*	55	50	50
3	60*	52*	52	48	48

*Did not meet structural resistance requirement.

Table D-35 presents the estimated minimum penetration depth for each candidate section to meet the factored geotechnical resistance requirements at the Strength I limit state. The larger pile sections require less pile penetration depth than the smaller pile sections to provide the same geotechnical resistance. In addition, the factored load per pile decreases from Group Configuration 1 to Group Configuration 3. However, from the analyses in Blocks 13 through 15, the established minimum penetration depth to preclude unacceptable vertical deformation requires all piles to be driven deeper than the depth needed solely for their factored geotechnical resistance. The minimum penetration depth requirement results in the additional geotechnical resistance gained by further pile embedment to be essentially wasted and therefore uneconomical.

Table D-35 Estimated Minimum Penetration Depth for Factored Geotechnical Resistance at Strength I Limit State

Group Configuration	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
1	59*	45*	45**	45**	43**
2	46*	45*	45**	38**	35**
3	45*	45*	43**	32**	30**

*Did not meet structural resistance requirement.

**Must be driven deeper to meet deformation requirements (Table D-28).

The unit cost per foot of pile installed should be estimated. Past pricing information is generally the best guide. However due to fluctuations in the market price of material and other factors, pile costs are subject to change. Section 6.14 provides recent piling cost information from several state agency databases. In the calculation below, the cost of steel of \$0.90 per pound is used as a baseline to determine pile cost. Equation D-5 shows the cost per linear foot calculation in which the cost is determined for the HP12x74 pile section. Table D-36 presents the cost per linear foot for each of the 5 candidate pile sections and shows reasonable agreement with the installed cost per foot of pile provided in Section 6.14.

Determine price per linear foot for HP12x74 pile section.

- \$ = cost per pound of steel, \$0.90/lb.
- w = weight per linear foot of pile section, 74 lbs/ft.

$$\text{Cost} = \frac{\$}{\text{lbs}} * \frac{\text{lbs}}{\text{ft}} \quad \text{Eq. D-6}$$

$$\text{Cost} = \left(\frac{\$0.90}{\text{lbs}} \right) * \left(\frac{74\text{lbs}}{\text{ft}} \right)$$

$$\text{Cost} = \frac{\$66.60}{\text{ft}}$$

Table D-36 Estimated Cost Per Linear Foot for 5 Candidate Pile Sections

HP 10x42 (\$ / ft)	HP 12x53 (\$ / ft)	HP 12x74 (\$ / ft)	HP 14x89 (\$ / ft)	HP 14x117 (\$ / ft)
37.80	47.70	66.60	80.10	105.30

The pile cost versus pile penetration depth is plotted for each of the 5 candidate pile sections in Figure D-34. In this calculation, the cost per linear foot is multiplied by the pile length, and due to the price difference per foot, the cost difference between pile sections becomes more pronounced with depth. Table D-37 shows the price per pile at the established minimum pile penetration depth. For example, for the HP 12x 74 pile section in Group Configuration 1, the individual cost is determined by multiplying the cost per foot by the penetration depth. At its minimum pile penetration depth of 60 feet, the cost per pile is \$3,996. There will be additional pile length embedded in the cap. However that length is currently undetermined, and for pile cost estimation and decision purposes, it can be considered negligible. Table D-38 shows the pile group cost reflecting the cost per pile and number of piles in the group.

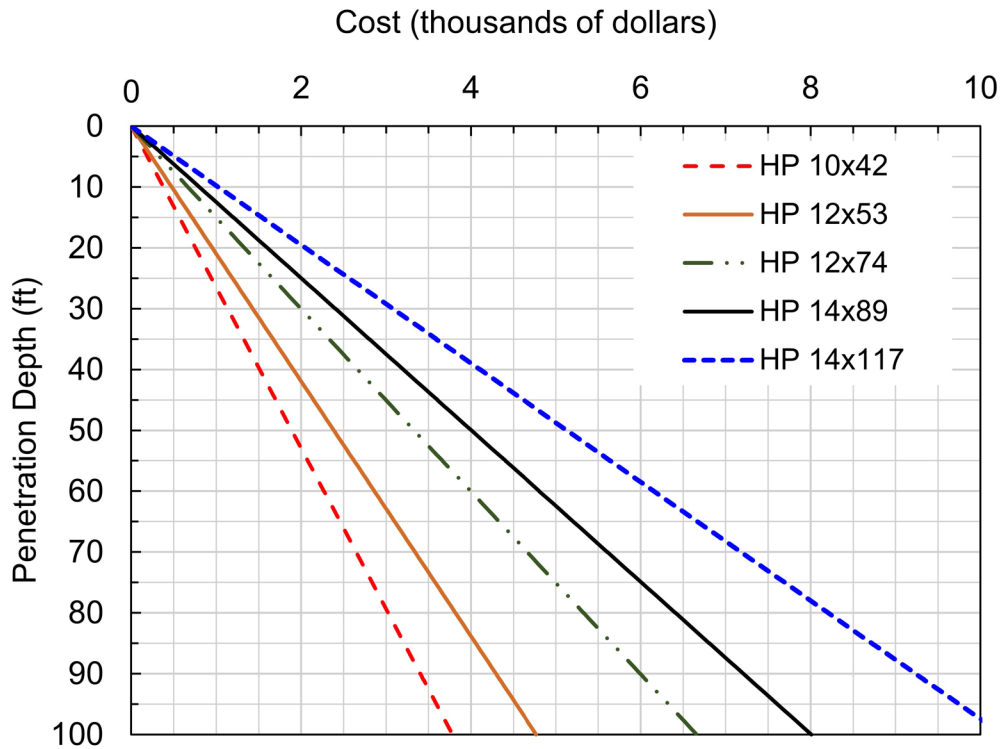


Figure D-34 Pile cost versus penetration depth.

Table D-37 Cost per Pile at Established Minimum Penetration Depth for Piles Meeting Structural Requirements

Group Configuration	HP 12x74	HP 14x89	HP 14x117
1	\$3,996	\$4,325	\$5,438
2	3,663	4,005	5,130
3	3,463	3,845	4,925

Table D-38 Pile Group Cost at Established Minimum Penetration Depth for Piles Meeting Structural Requirements

Group Configuration	HP 12x74	HP 14x89	HP 14x117
1	\$71,928	\$77,857	\$97,880
2	80,586	88,110	112,860
3	90,043	99,965	128,045

Based upon this comparison, the HP 14x117 pile section proves to be least economical followed by the HP 14x89 section. Conversely, the HP 12x74 pile

section in Group Configuration 1 appears to be the most economical. The cost of the pile cap must also be considered before selecting the lowest cost.

The cost of the pile cap is estimated and factored into the total foundation cost. Section 8.9 outlines a procedure to estimate the total pile cap thickness. Equation 8-80 is used along with the factored geotechnical resistance, per pile, to estimate this value. Table D-39 summarizes the factored geotechnical resistance for each pile section based upon the estimated minimum pile toe elevation.

Table D-39 Factored Geotechnical Resistance, R_r , at Estimated Minimum Pile Penetration Depth

Group Configuration	HP 12x74 (kips)	HP 14x89 (kips)	HP 14x117 (kips)
1	323	323	323
2	265	265	265
3	224	224	224

A sample calculation for determining the cap thickness is shown below for the HP 12x74 H-pile in Group Configuration 1. Table D-40 summarizes the estimated cap thickness for each pile section and pile group permutation calculated using this procedure.

Estimate the total pile cap thickness for a HP12x74 H-pile in Group Configuration 1.

P_{ui} = maximum single pile factored axial load, $Q = 323$ kips (Table D-14).

$$t_{cap} = \frac{P_{ui}}{12} + 30 \quad [\text{Eq. 8-80}]$$

$$t_{cap} = \frac{(323 \text{ kips})}{12} + 30$$

$$t_{cap} = 57 \text{ inches}$$

Table D-40 Estimated Total Pile Cap Thickness

Group Configuration	HP 12x74 (inches)	HP 14x89 (inches)	HP 14x117 (inches)
1	57	57	57
2	52	52	52
3	49	49	49

Next, the volume of reinforce concrete required to construct the pile cap is determined from the estimated total pile cap thickness. The total pile cap width and length values were previously provided in Table D-12. The resulting volume of reinforced concrete in the pile cap for each pile section and pile group permutation is presented in Table D-41. Note that pile cap volume is shown in cubic yards (CY).

Table D-41 Estimated Volume of Reinforced Concrete in Pile Cap

Group Configuration	HP 12x74 (CY)	HP 14x89 (CY)	HP 14x117 (CY)
1	48.5	48.5	48.5
2	44.4	44.4	44.4
3	37.6	37.6	37.6

To estimate the pile cap cost, past pricing information is generally the best guide. However, similar to estimating the pile cost, due to fluctuations in the market price of material and other factors, pile cap costs are subject to change. The cost of the reinforced concrete pile cap, furnished and constructed, is estimated to be \$500 /CY. Using this estimated value, the volumes presented in Table D-41 were used to estimate the cost of the various reinforced concrete pile caps. The pile cap cost for each candidate section and group configuration permutation is shown in Table D-42.

Table D-42 Estimated Cost of Reinforced Concrete Pile Cap

Group Configuration	HP 12x74	HP 14x89	HP 14x117
1	\$24,264	\$24,264	\$24,264
2	22,204	22,204	22,204
3	18,794	18,794	18,795

By adding the cost of the pile cap and piles for each permutation, the estimated total foundation cost is determined as presented in Table D-43. Considering both the pile and the pile cap costs, the HP 12x74 in Group Configuration 1 is the most economical option.

With field resistance determination tests yet to be performed, only geotechnical correlations have been considered to estimate nominal resistance vs. depth relationships. Dynamic testing results may require additional (or less) pile embedment to satisfy resistance requirements. The potential for such variations between estimated and installed lengths becomes more-pronounced with friction

Table D-43 Estimated Foundation Cost Including Piles and Pile Cap at North Abutment

Group Configuration	HP 12x74	HP 14x89	HP 14x117
1	\$96,192	\$102,121	\$122,145
2	102,790	110,314	135,064
3	108,838	118,759	146,839

piles than with end-bearing piles. To provide insight into how such potential length variations might affect costs, an additional comparison of pile cost versus depth from the estimated depth was completed for the final three candidate pile sections.

In this comparison, Table D-44 shows the cost per pile and cost per pile group for the three lowest cost permutations of group configuration and candidate sections from Table D-43 (i.e., the HP 12x74, the HP 14x89 section, and the HP14x117 section all in Group Configuration 1). Group Configuration 1 contains 18 piles. Although the pile sections have different established minimum penetration depth requirements, the depth provided in Table D-44 will be used as a baseline to evaluate the cost risk associated with pile overrun or underrun (“over/underrun”) that may result from field determination testing. At the North Abutment, the established minimum pile penetration depth is greater than the estimated pile penetration depth for factored geotechnical resistance in axial compression, therefore underrun is provided for demonstration purposes only.

If the estimated soil resistance is underpredicted and the 14x89 pile section in Group Configuration 1 is determined to have sufficient factored resistance upon field determination testing at 49 feet (underrun of 5 feet), the cost per pile would be reduced by \$481 and thus a reduction of \$8,651 relative to the original \$102,121 estimate for the total foundation cost would result. A graphical representation of this effect is presented in Figure D-35 (Underrun is shown for demonstration purposes only at the North Abutment as the established minimum pile penetration depth is 60 feet for this pile section). Conversely, if for example, the HP 12x74 section must be driven 15 feet deeper (overrun of 15 feet) to achieve sufficient factored resistance; an additional cost of \$999 per pile would result. Based upon the number of piles and estimated pile cap cost, this would increase the total foundation cost by \$17,982 from the original estimate of \$96,192.

If it is assumed that the same pile over/underrun length will result for the pile sections after field determination testing, a cost comparison can be made for a range

of over/underrun lengths for the final candidate pile sections and group configurations. This comparison can compare the sensitivity of foundation cost to over/underrun lengths among candidate pile sections, and therefore may aid selection among the sections.

Table D-44 Comparison of Pile Over/Underrun Costs for Three Lowest Cost Alternatives at the North Abutment

Over/Underun (feet)	HP 12x74 Change in Cost per Pile	HP 12x74 Change in Cost of Foundation	HP 14x89 Change in Cost per Pile	HP 14x89 Change in Cost of Foundation	HP 14x117 Change in Cost per Pile	HP 14x117 Change in Cost of Foundation
-5	(\$400)	\$(7,193)	\$(481)	\$(8,651)	\$(616)	\$(11,081)
0	-	-	-	-	-	-
5	333	5,994.00	401	7,209	\$513	9,234
10	666	11,98	801	14,418	1,026	18,468
15	99	17,98	1,202	21,627	1,539	27,702

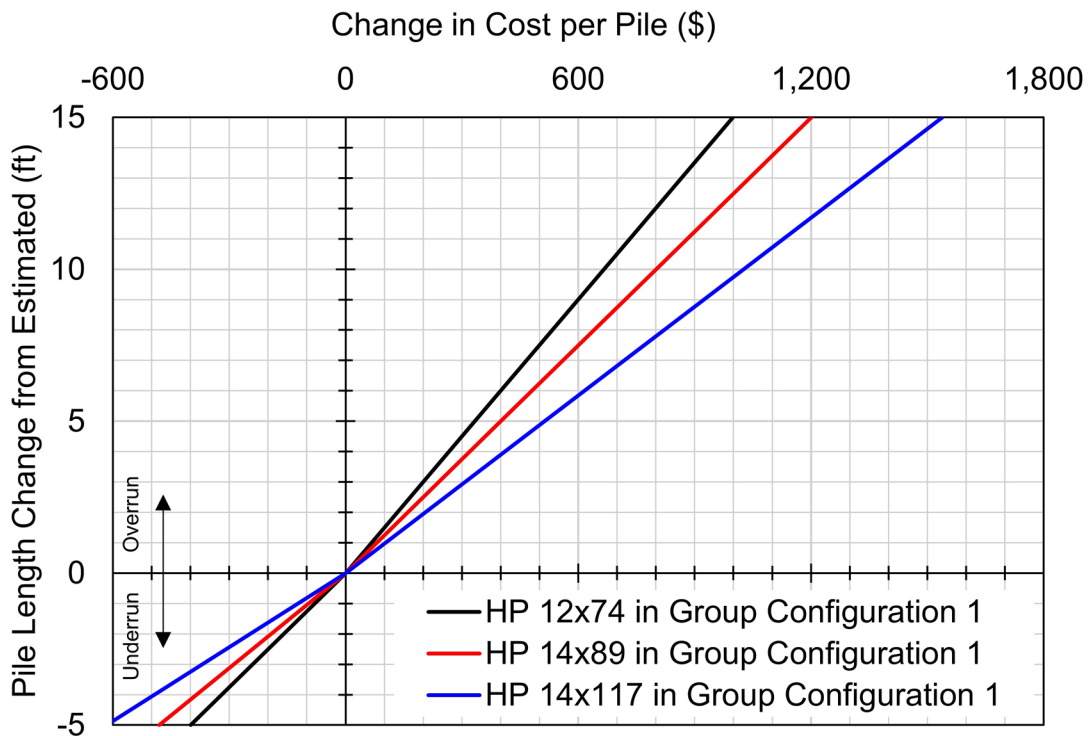


Figure D-35 Change in cost per pile from over/ underrun.

Table D-45 compares the overall foundation cost change associated with over/underrun lengths for the final three candidate pile sections and pile group configuration permutations; graphical results of this interpretation are presented in Figure D-36. The change of foundation cost is again presented along with the relative difference based upon the same assumed pile over/underrun for the pile

sections. The difference in foundation cost is referenced from the HP12x74 pile section in Group Configuration 1. For example if 15 feet of overrun occurs for each permutation, the resulting cost difference between the HP 12x74 section in Group Configuration 1 and the HP 12x74 section in Group Configuration 1 is \$17,982 minus \$21,627 or -\$3,645.

Table D-45 Comparison of Pile Group Over/Underrun Costs for Three Lowest Cost Alternatives at the North Abutment

Over/ Underun (feet)	HP 12x53 Change in Cost of Foundation	HP 14x89 Change in Cost of Foundation	Difference in Cost of Foundation	HP 14x117 Change in Cost of Foundation	Difference in Cost of Foundation
-5	\$(7,193)	\$(8,651)	\$1,458	\$(11,081)	\$3,888
0	-	-	-	-	-
5	5,994	7,209	(1,215)	9,234	(3,240)
10	11,988	14,418	(2,430)	18,468	(6,480)
15	17,982	21,627	(3,645)	27,702	(9,720)

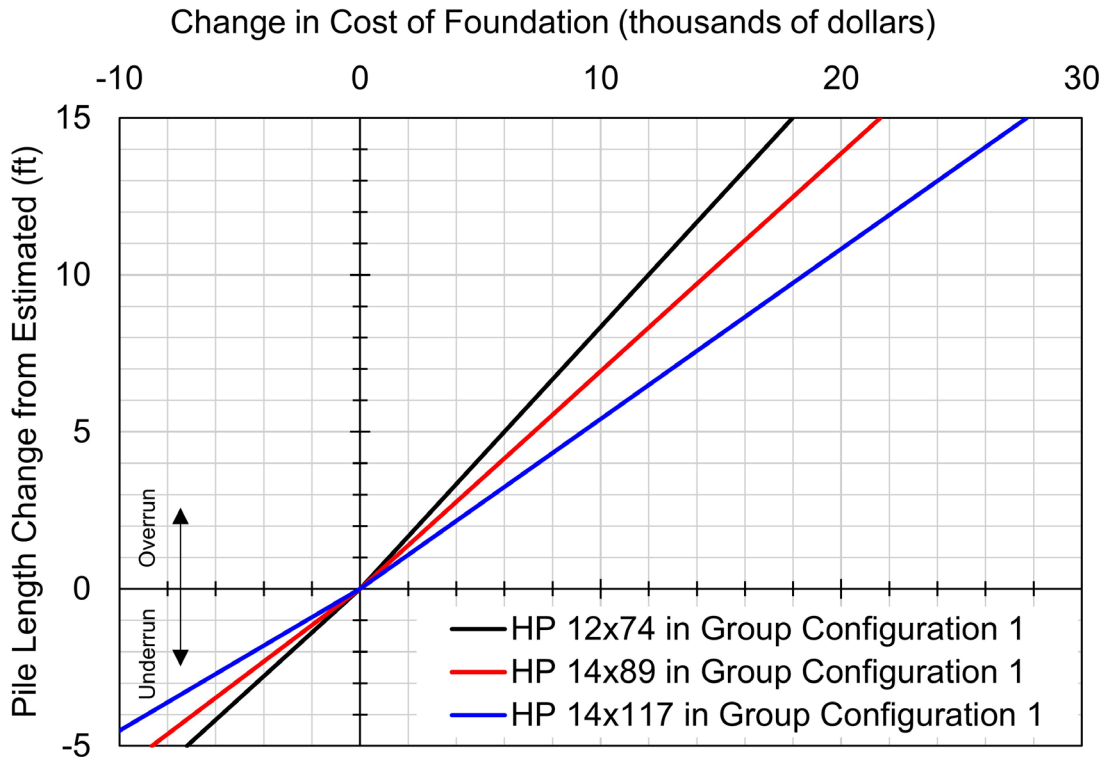


Figure D-36 Change in total cost of foundation from overrun/ underrun.

In the absence of a test pile program, a definitive cost comparison cannot be made; however, the above described cost analysis can assist to reduce the number of options and estimate the relative risk of cost overruns or savings associated with a

given candidate pile section and group configuration. At this juncture, a preliminary design of the Pier 2 and the South Abutment should be performed before making a final decision on pile type and size. This offers flexibility for the pile group design at other locations if the agency practice is to use the same pile sections at all foundation locations. If, for example, larger loads are required at Pier 2 or the South Abutment, the use of a larger pile section may be reason to further eliminate one or more of the candidate pile sections or group configurations.

D.20 Decision 20: Is the Preliminary Design of All Substructure Foundations Complete?

No. The preliminary foundation design has been completed for only the North Abutment. The preliminary design needs to be completed for the Pier and for the South Abutment. Return to Block 9 and begin the preliminary design for the next substructure location.

D.21 Block 9: Pier 2 – Calculate Nominal and Factored Structural Resistances for all Candidate Piles

The nominal structural resistance is now re-evaluated for the five candidate H-pile sections at Pier 2. The H-pile sections selected for evaluation again include a HP 10x42, a HP 12x53, a HP 12x74, a HP 14x89 and a HP 14x117. A detailed step by step example for calculation of the nominal structural resistance was previously presented in Section 8.5.3 for an HP 14x117 H-pile section. Therefore, this process is not repeated for the five candidate pile sections.

Due to channel degradation and local scour, the hydraulic engineer estimated scour depth is up to 5 feet below the bottom of pile cap for the design flood. The nominal structural resistance in axial compression must therefore account for a longer unbraced length. At the Strength limit state, the design flood event scour results in an unbraced length of 5 feet. Furthermore, for the check flood at the Extreme Event Limit State, the hydraulic engineer estimated scour potential up to 7 feet below the bottom of the pile cap. In this case, 7 feet of unbraced length is factored into the nominal structural resistance calculation. Table D-46 presents the nominal structural resistances in axial compression, flexure, and shear for the five candidate pile sections.

Table D-46 Nominal Structural Resistance in Axial Compression, Flexure and Shear

H-pile Section	HP 10x42	HP 12x53	HP 12x74	HP 14x89	HP 14x117
P_n , Nominal Resistance in Axial Compression (kips) (Strength V)	581	734	1043	1266	1670
P_n , Nominal Resistance in Axial Compression (kips) (Extreme Event II)	546	702	999	1229	1624
M_{ny} , Nominal Resistance in Weak Axis Flexure (kip-ft)	82	114	118	257	380
M_{nx} , Nominal Resistance in Strong Axis Flexure (kip-ft)	176	295	433	592	807
V_n , Nominal Resistance in Shear (kips)	118	149	214	246	331

It is anticipated that the piles at Pier 2 will be driven into the dense gravel with sand deposit, or possibly to the underlying bedrock. The dense gravel with sand deposit contains occasional boulders. In these conditions, pile shoes are recommended for

use with the H-piles to reduce the risk of damage. Therefore, the applicable structural resistance factors, ϕ_c , are 0.5 for axial compression resistance and 0.7 for combined axial compression and flexural resistance. A resistance factor of $\phi_v=1.0$ is used for shear, and a resistance factor of $\phi_f=1.0$ is applicable for flexure only. Table D-47 summarizes the calculated factored structural resistances in axial compression, combined axial and flexure, flexure, and shear.

Table D-47 Factored Structural Resistance in Axial Compression, Flexure and Shear

H-pile Section	HP 10x42	HP 12x53	HP 12x74	HP 14x89	HP 14x117
P_r , Factored Resistance in Axial Compression, $\phi_c = 0.5$ (kips) (Strength V)	290	367	521	633	835
P_r , Factored Axial and Flexural Resistance, $\phi_c = 0.70$ (kips) (Strength V)	407	514	730	886	1169
P_n , Factored Resistance in Axial Compression, $\phi_c = 0.5$ (kips) (Extreme Event II)	273	351	500	615	812
P_r , Factored Axial and Flexural Resistance, $\phi_c = 0.70$ (kips) (Extreme Event II)	382	491	699	861	1137
M_{ry} , Factored Resistance in Weak Axis Flexure, $\phi_f=1.0$ (kip-ft)	82	114	118	257	380
M_{rx} , Factored Resistance in Strong Axis Flexure, $\phi_f=1.0$ (kip-ft)	176	295	433	592	807
V_r , Factored Resistance in Shear $\phi_v=1.0$ (kips)	118	149	214	246	331

D.22 Block 10: Pier 2 – Calculate Nominal and Factored Geotechnical Resistances in Axial Compression and Tension versus Depth for all Candidate Piles; Perform Preliminary Pile Drivability Analyses

The engineering properties for the soil conditions encountered at Pier 2 were determined in Block 5. The results of the boring program and laboratory tests are now used to develop a design profile for Pier 2. Engineering judgement was used in developing the design profile to delineate the subsurface conditions into layers with similar properties. An effective stress diagram, depicted in Figure D-37, was also made for this soil profile. This diagram includes the total stress, porewater pressure and effective stress versus depth. Figure D-37 also presents the basic soil profile for quick reference to the relevant soil layers. The effective stress diagram was

computed using Equations 5-7 through 5-9 from Chapter 5. These equations are repeated below. For Figure D-37 the total stress, porewater pressure and effective stress were calculated at 1 foot increments using a spreadsheet.

$$\sigma_{vo} = \sum_i^n (\gamma_i h_i) \quad [\text{Eq. 5-7}]$$

$$u = \gamma_w h_w \quad [\text{Eq. 5-8}]$$

$$\sigma'_{vo} = \sum_i^n (\gamma_i h_i) - \gamma_w h_w \quad [\text{Eq. 5-9}]$$

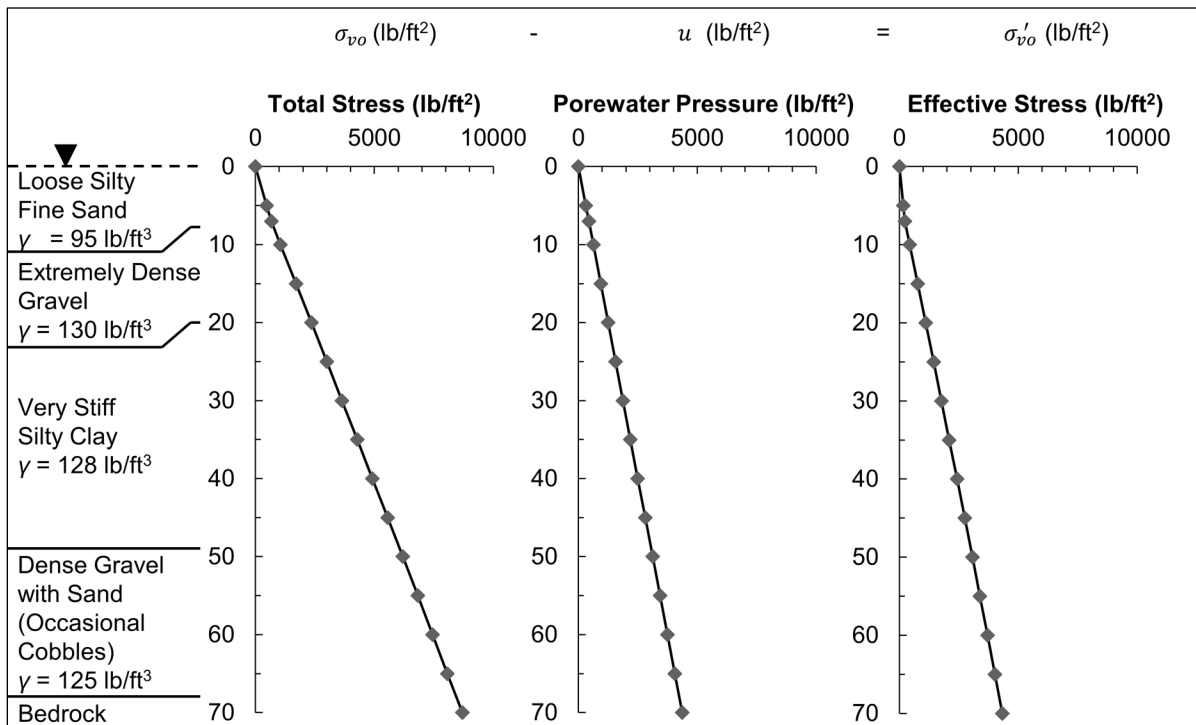


Figure D-37 Effective stress diagram for Pier 2 using Boring S-2.

To continue development of an idealized soil profile, field SPT N values were corrected for hammer energy transfer and vertical effective stress. The field SPT N values were first corrected for energy transfer using Equation 5-1. These results are presented in Table D-48. Typical correlations for SPT hammer type and energy transfer are provided in Section 5.1.1 (e.g., typical energy transfer of 80% for automatic hammer). The SPT hammer on the drill rig used for this project's soil exploration program was calibrated in accordance with ASTM D4633. Results of this calibration indicated an average energy transfer of 75%. Therefore, the *ER* value in Equation 5-1 is equal to 75.

$$N_{60} = N \left(\frac{ER}{60} \right) \quad [\text{Eq. 5-1}]$$

The SPT N value was then corrected for vertical effective stress as presented in Equation 5-2, using the Peck et al. (1974) correction factor, C_n . The depth value for this calculation was taken from the middle depth of the SPT sampling event after the 6 inch seating interval (e.g., the SPT N value recorded from 0.5-1.5 feet was corrected using the vertical effective stress at 1 foot).

$$(N_1)_{60} = C_n N_{60} \quad [\text{Eq. 5-2}]$$

$$C_n = 0.77 \log \left(\frac{20}{\sigma'_{vo}} \right) \quad [\text{Eq. 5-3}]$$

After correcting the SPT N values for energy transfer and vertical effective stress, an average corrected N value was determined for each respective cohesionless soil layer. The N value from the first SPT sample in Layer 1 was not included in this procedure for Layer 1 since the bottom of the footing at Pier 2 is below this sample depth.

Table D-49 presents the average $(N_1)_{60}$ value for each cohesionless layer at Pier 2, the coefficient of variation, COV, within the layer, and the effective stress friction angle chosen for the layer. The coefficient of variation for each layer was less than 25% indicating low variability within each of the identified layers. Higher variability would have necessitated separating the soil profile into additional soil layers.

Table D-48 Correction of Field SPT N Value for Energy and Vertical Effective Stress at Pier 2 using Boring S-2

Soil Layer	Depth (ft)	σ'_{vo} (ksf)	Field N value	N_{60}	C_n	$(N_1)_{60}$
1	1	0.033	3	4	2.00	8
1	6	0.101	7	9	2.00	18
2	8	0.166	51	64	1.83	117
2	11	0.369	52	65	1.57	102
2	16	0.707	55	69	1.35	93
4	51	3.013	42	53	0.86	45
4	56	3.326	46	58	0.83	48
4	61	3.639	41	51	0.80	41
4	66	3.952	50	63	0.77	48

The $(N_1)_{60}$ values were used to estimate the effective stress friction angle of each soil layer in accordance with Table 5-5 of Chapter 5. These friction angle

correlations are presented in Table D-49. Layer 2 consists of extremely dense gravel and Layer 4 consists of hard angular gravel with sand. Therefore, as discussed in Section 5.5.1, a design friction angle of 36 degrees was used for estimating the shaft resistance in both of these layers while the friction angle noted in Table D-49 was used to estimate the toe resistance.

Table D-49 Soil Layer Effective Stress Friction Angle Correlations at Pier 2

Soil Layer	Average $(N_1)_{60}$	COV	ϕ' shaft (degrees)	ϕ' toe (degrees)
1	18	0.0%	32	32
2	104	9.6%	36	43
4	46	6.3%	36	38

Soil Layer 3 consists of very stiff silty clay. Unconfined compression strength tests were performed on each undisturbed cohesive soil sample. The undrained shear strength was then calculated by dividing the unconfined compression strength by 2.

Figure D-38 presents the undrained shear strength versus depth for Soil Layer 3. The undrained shear strength increases approximately linearly from 20 to 45 feet. Layer 3 was refined into 6 sublayer increments of 5 feet, each using the respective undrained shear strength value determined from laboratory testing. Table D-50 presents the shear strength values for Soil Layer 3 as well as sublayers a through f.

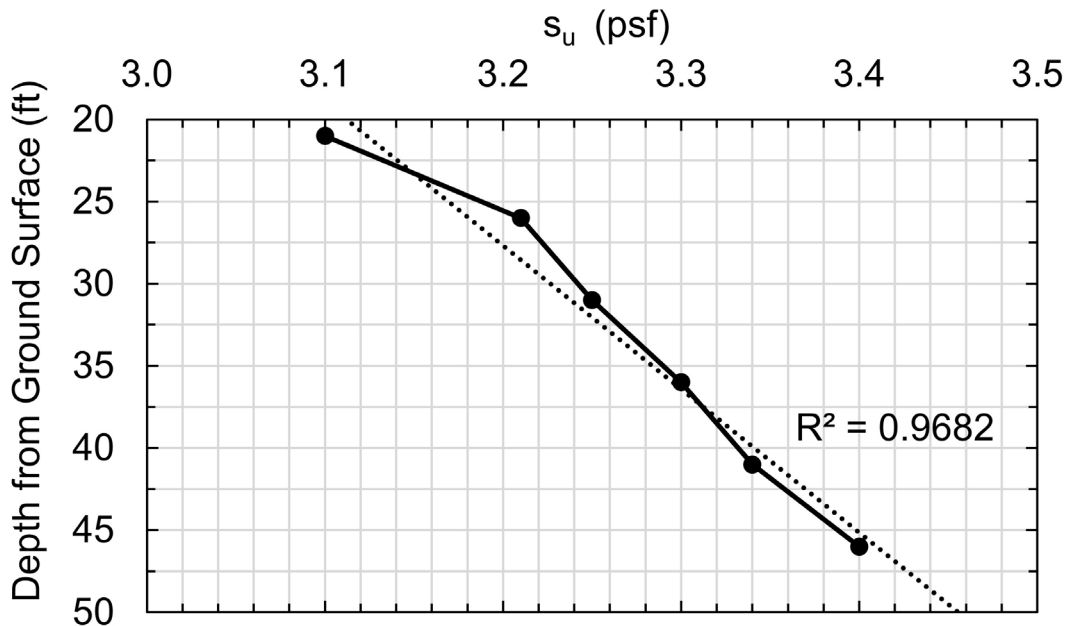


Figure D-38 Undrained shear strength, s_u , versus depth for Soil Layer 3 at Pier 2.

Table D-50 Undrained Shear Strength, s_u , for Soil Layer 3 at Pier 2

Soil Layer	Depth (ft)	σ'_{vo} (ksf)	q_u (tsf)	s_u (ksf)
3a	21	1.045	3.10	3.10
3b	26	1.373	3.21	3.21
3c	31	1.701	3.25	3.25
3d	36	2.029	3.30	3.30
3e	41	2.357	3.34	3.34
3f	46	2.685	3.40	3.40

The design soil profile for Pier 2 is presented in Figure D-39, with the bottom of footing elevation noted 10 feet below ground surface. Some soil properties in the design soil profile such as the elastic moduli, E_s , and the initial cyclic modulus of subgrade reaction, k_c , have been selected based on published correlations in the absence of laboratory and field testing. For this particular soil profile, the elastic moduli were determined from the SPT correlation in Table 5-11 and the initial cyclic moduli of subgrade reaction were selected based on representative values shown in Table 7-22. The average cohesive soil strength for Layer 3 was used to select the 50% strain factor, ϵ_{50} , from Table 7-21.

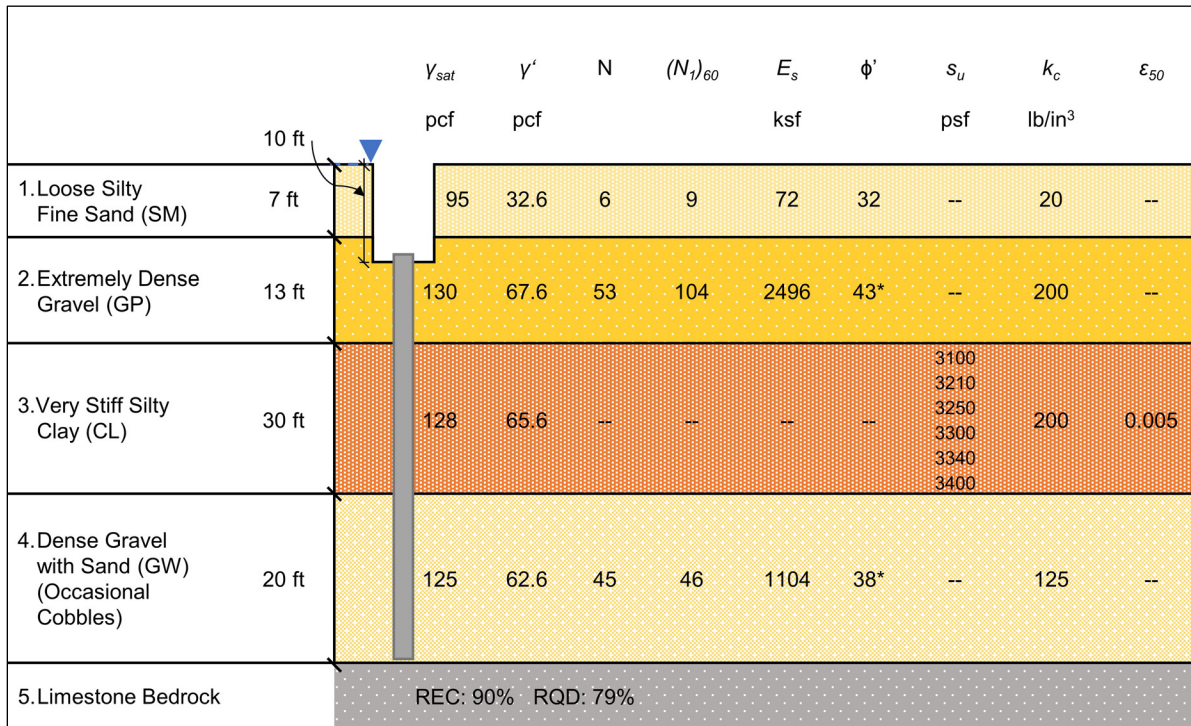


Figure D-39 Design soil profile at Pier 2.

D.22.1 Geotechnical Resistance in Axial Compression

The nominal geotechnical resistance in axial compression is now determined for the selected candidate H-pile sections. The nominal geotechnical resistance can be calculated by hand or with computer software using an appropriate analysis method based upon soil and pile type. Chapter 7 describes appropriate methods for this purpose as well as available computer programs.

For this example, the DrivenPiles computer program was used to calculate the nominal resistance, shaft resistance, and toe resistance as a function of depth for each of the five candidate H-piles. This software program was selected since it uses the FHWA recommended Nordlund method and alpha method to estimate the nominal geotechnical resistance of H-piles in cohesionless and cohesive soils, respectively. The DrivenPiles program also allows the analyst to select a different friction angle for shaft and toe resistance calculations in any layer. This option was utilized for the extremely dense gravel in Layer 2 and the gravel with sand comprising Layer 4. The computer program was also used to evaluate the nominal geotechnical resistance at Pier 2 during the design flood and at the check flood.

A summary of the nominal shaft, nominal toe, and nominal geotechnical resistance is presented for the HP 12x74 H-pile section in Table D-51. A graphical interpretation of the estimated resistance is presented in Figure D-40. The indicated pile penetration depth is referenced from the bottom of pile cap (Elev. 275.0 feet), which is 10 feet below the original ground surface (Elev. 285.0 feet).

Table D-51 Nominal Shaft, Nominal Toe and Nominal Geotechnical Resistance for
HP 12x74 at Pier 2 (pre-scour)

Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Geotechnical Resistance (kips)	Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Geotechnical Resistance (kips)
0.01	0.0	106.5	106.5	29.99	109.9	30.5	140.4
1	1.1	123.0	124.2	30.01	110.1	30.8	140.9
2	2.4	139.7	142.1	31	115.1	30.8	145.9
3	3.9	156.4	160.2	32	120.2	30.8	151.0
4	5.5	173.1	178.5	33	125.3	30.8	156.1
4.99	7.2	189.6	196.8	34	130.4	30.8	161.2
5.01	7.3	189.9	197.2	34.99	135.5	30.8	166.3
6	9.2	206.4	215.6	35.01	135.6	31.4	166.9
7	11.3	223.1	234.4	36	140.6	31.4	172.0
8	13.5	239.8	253.3	37	145.7	31.4	177.1
9	16.0	256.5	272.4	38	150.8	31.4	182.2
9.99	18.5	273.0	291.5	39	155.9	31.4	187.3
10.01	18.6	28.6	47.2	39.99	161.0	31.4	192.3
11	22.4	28.6	51.0	40.01	161.1	251.3	412.4
12	26.3	28.6	54.9	41	168.5	256.4	424.9
13	30.4	28.6	59.0	42	176.2	261.5	437.7
14	34.6	28.6	63.2	43	184.0	266.6	450.6
14.99	38.9	28.6	67.5	44	191.9	271.7	463.7
15.01	39.0	29.6	68.6	45	200.0	275.4	475.4
16	43.1	29.6	72.8	46	208.3	275.4	483.6
17	47.4	29.6	77.0	47	216.7	275.4	492.0
18	51.8	29.6	81.4	48	225.2	275.4	500.6
19	56.4	29.6	86.0	49	234.0	275.4	509.3
19.99	61.0	29.6	90.6	50	242.8	275.4	518.2
20.01	61.1	30.0	91.1	51	251.8	275.4	527.2
21	65.5	30.0	95.5	52	261.0	275.4	536.3
22	70.1	30.0	100.1	53	270.3	275.4	545.6
23	74.8	30.0	104.8	54	279.8	275.4	555.1
24	79.7	30.0	109.7	55	289.4	275.4	564.7
24.99	84.7	30.0	114.6	56	299.1	275.4	574.5
25.01	84.7	30.5	115.2	57	309.0	275.4	584.4
26	89.5	30.5	119.9	58	319.1	275.4	594.5
27	94.4	30.5	124.9	59	329.3	275.4	604.7
28	99.5	30.5	129.9	59.99	339.6	275.4	614.9
29	104.7	30.5	135.1	60.01	339.7	1042.6	1382.3

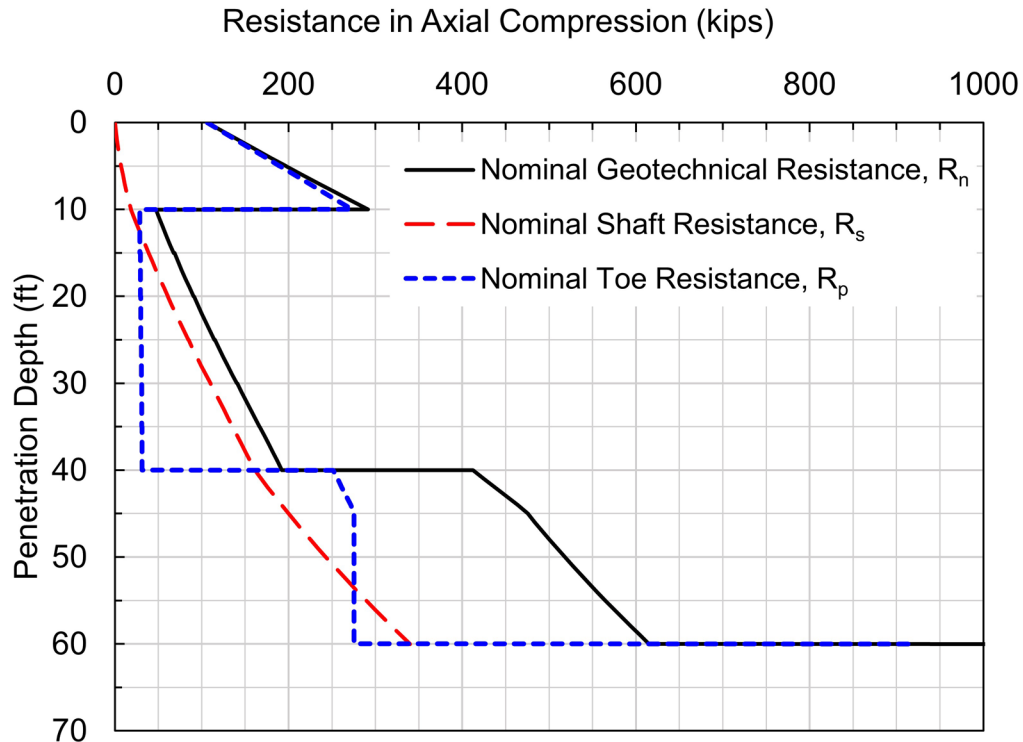


Figure D-40 Nominal geotechnical resistance in axial compression versus pile penetration depth for HP 12x74 at Pier 2 (pre scour).

D.22.1.1 Geotechnical Resistance in Axial Compression at the Design Flood

The nominal geotechnical resistance in axial compression at the design flood must also be evaluated. In the design flood, 5 feet of channel degradation scour is anticipated. The channel degradation scour results in a change in vertical effective stress from the removal of 5 feet of channel materials. The channel degradation scour occurs above the bottom of the pile cap. Hence the channel degradation scour results in no loss of nominal resistance other than that caused by the reduction in overburden stress.

In addition to the channel degradation scour, 10 feet of local scour occurs during the design flood. This 10 feet of local scour occurring below the channel degradation scour results in the loss of frictional resistance on the upper 5 feet of the pile in the design flood event. Hence, these two scour mechanisms require a more rigorous analysis of the nominal resistance beyond simple subtraction of the shaft resistance within the scour prism. Therefore, the DrivenPiles program was used to calculate the shaft, toe and nominal resistance as a function of depth for each of the five candidate H-piles in the design flood.

Figure D-41 presents plots of the nominal shaft resistance, R_s , the nominal toe resistance, R_p , and nominal resistance, R_n for the HP 12x74 pile section versus pile penetration depth. These results are also presented numerically in Table D-52. The depth indicated in both Figure D-41 and Table D-52 is referenced from the bottom of pile cap (EL 275 feet), which is 10 feet below the original ground surface elevation (EL 285 feet).

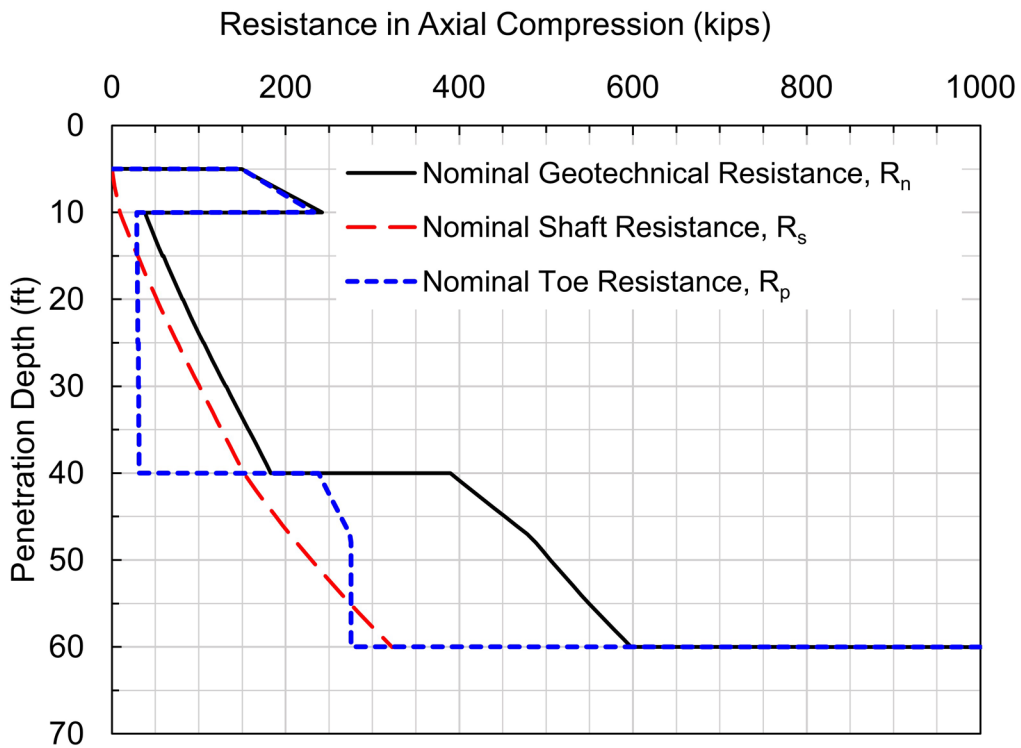


Figure D-41 Nominal geotechnical resistance in axial compression versus pile penetration depth for HP 12x74 at Pier 2 (design flood).

The nominal resistance versus depth at the design flood was calculated for all of the candidate pile sections in a similar manner. These results are presented in Figure D-42.

Table D-52 Nominal Shaft, Nominal Toe and Nominal Geotechnical Resistance for
HP 12x74 at Pier 2 (design flood)

Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Geotechnical Resistance (kips)	Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Geotechnical Resistance (kips)
0.01	0.0	0.0	0.0	29.99	100.8	30.8	131.7
1	0.0	0.0	0.0	30.01	105.9	30.8	136.7
2	0.0	0.0	0.0	31	111.0	30.8	141.8
3	0.0	0.0	0.0	32	116.1	30.8	146.9
4	0.0	0.0	0.0	33	121.2	30.8	152.0
4.99	0.0	149.7	149.7	34	126.3	30.8	157.1
5.01	1.5	166.2	167.8	34.99	126.4	31.4	157.7
6	3.3	182.9	186.1	35.01	131.4	31.4	162.8
7	5.1	199.6	204.7	36	136.5	31.4	167.9
8	7.2	216.2	223.4	37	141.6	31.4	173.0
9	9.3	232.8	242.1	38	146.7	31.4	178.1
9.99	9.4	28.6	38.0	39	151.8	31.4	183.1
10.01	13.2	28.6	41.8	39.99	151.9	238.0	389.9
11	17.1	28.6	45.7	40.01	158.9	243.1	402.0
12	21.2	28.6	49.8	41	166.2	248.2	414.4
13	25.4	28.6	54.0	42	173.6	253.3	426.9
14	29.7	28.6	58.3	43	181.1	258.4	439.6
14.99	29.8	29.6	59.4	44	188.9	263.5	452.4
15.01	33.9	29.6	63.5	45	196.7	268.6	465.4
16	38.2	29.6	67.8	46	204.7	273.8	478.5
17	42.6	29.6	72.2	47	212.9	275.4	488.2
18	47.2	29.6	76.8	48	221.2	275.4	496.6
19	51.8	29.6	81.4	49	229.7	275.4	505.0
19.99	51.9	30.0	81.9	50	238.3	275.4	513.6
20.01	56.3	30.0	86.3	51	247.0	275.4	522.4
21	60.9	30.0	90.9	52	256.0	275.4	531.3
22	65.6	30.0	95.6	53	265.0	275.4	540.4
23	70.5	30.0	100.5	54	274.3	275.4	549.6
24	75.4	30.0	105.4	55	283.6	275.4	559.0
24.99	75.5	30.5	106.0	56	293.1	275.4	568.5
25.01	80.3	30.5	110.7	57	302.8	275.4	578.2
26	85.2	30.5	115.7	58	312.6	275.4	588.0
27	90.3	30.5	120.7	59	322.5	275.4	597.9
28	95.5	30.5	125.9	59.99	322.7	922.6	1245.3
29	100.7	30.5	131.2	60.01	327.7	1042.6	1370.3

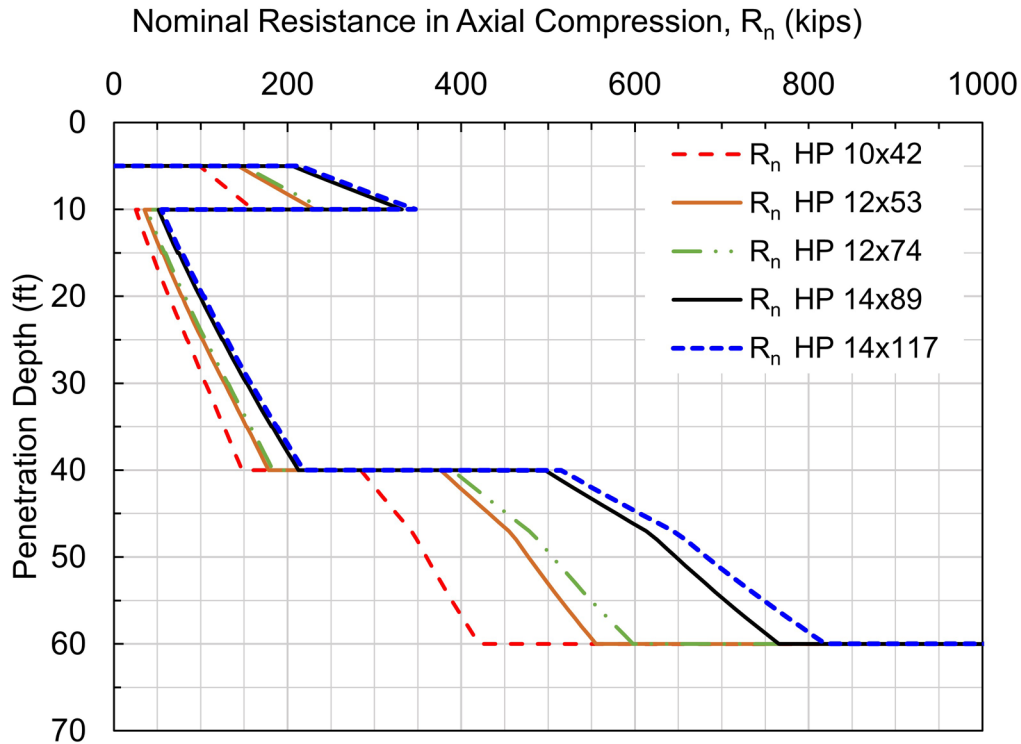


Figure D-42 Nominal geotechnical resistance in axial compression versus penetration depth for all candidate pile sections at Pier 2 (design flood).

Figure D-43 presents a design chart of the nominal and factored geotechnical resistance for the HP 12x74 pile section in axial compression during the design flood event. The design chart includes the nominal geotechnical resistance as well as the factored geotechnical resistance based on several resistance determination methods. The factored geotechnical resistance versus penetration depth is plotted for resistance determination by a static load test with dynamic testing of 2% of the piles ($\phi_{dyn}=0.80$), by dynamic testing of at least two piles per site condition, but no less than 2% of the production piles, with signal matching ($\phi_{dyn}=0.65$), and by wave equation analysis ($\phi_{dyn}=0.50$). Also included is the factored geotechnical resistance based on the static analysis method used in this design example. For this, ($\phi_{stat}=0.45$) was applied to the cohesionless soils as a result of using the Nordlund method while ($\phi_{stat}=0.35$) applied to the cohesive soils as a result of using the alpha method. For a single pile section (HP 12x74), this figure illustrates the effects the various resistance determination methods have on the pile length required for a given factored resistance, the factored resistance available from a given pile section, and the potential impact of these factors on the number of piles needed to resist axial compression loads.

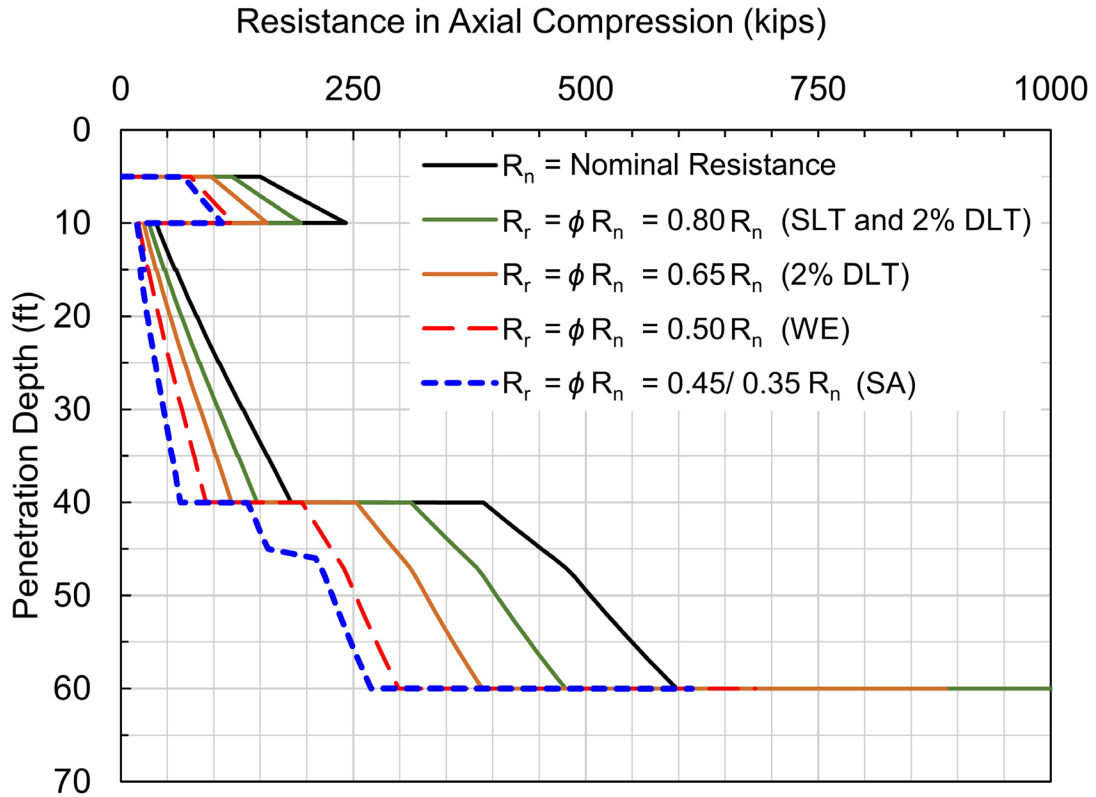


Figure D-43 Design chart of nominal and factored geotechnical resistance in axial compression versus pile penetration depth for HP 12x74 at the Pier 2 (design flood).

The factored geotechnical resistance in axial compression for all candidate pile sections is presented in Figures D-44 through D-47 during the design flood event. The presented factored geotechnical resistances are based upon the field determination method or static analysis method used. Accordingly, these figures can be used to assess the effects of the various determination methods on pile length, pile section selection, and the required number of piles to resist axial compression loads.

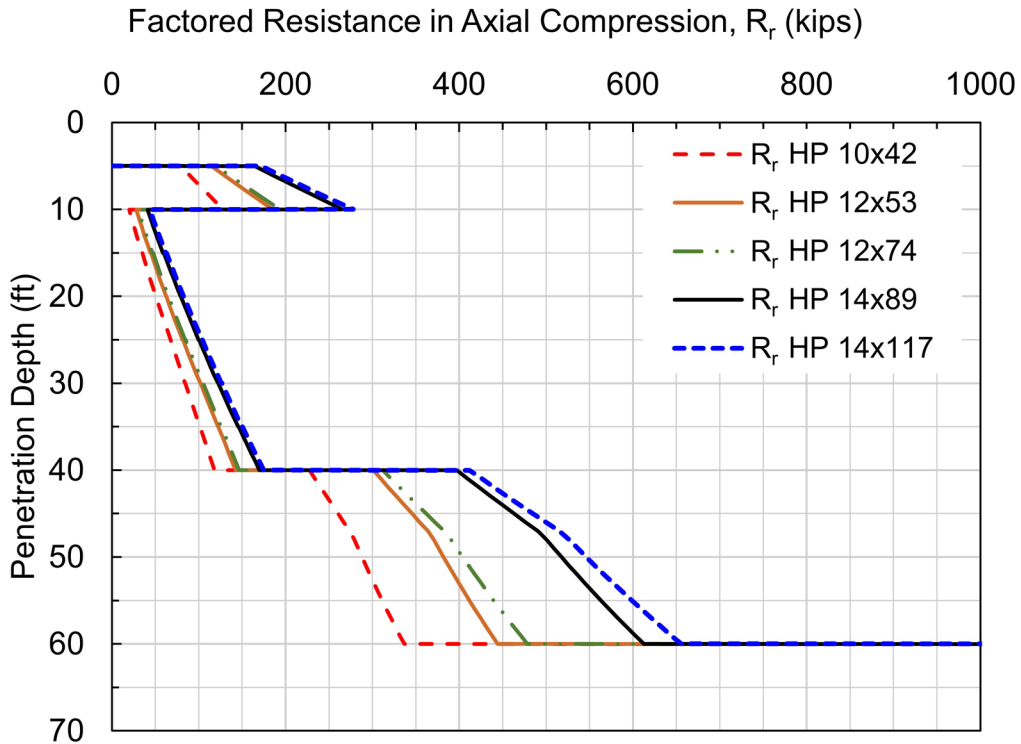


Figure D-44 Factored geotechnical resistance, R_r , in axial compression based on field determination by static load test and dynamic testing 2% of the piles, $\phi_{dyn}=0.80$.

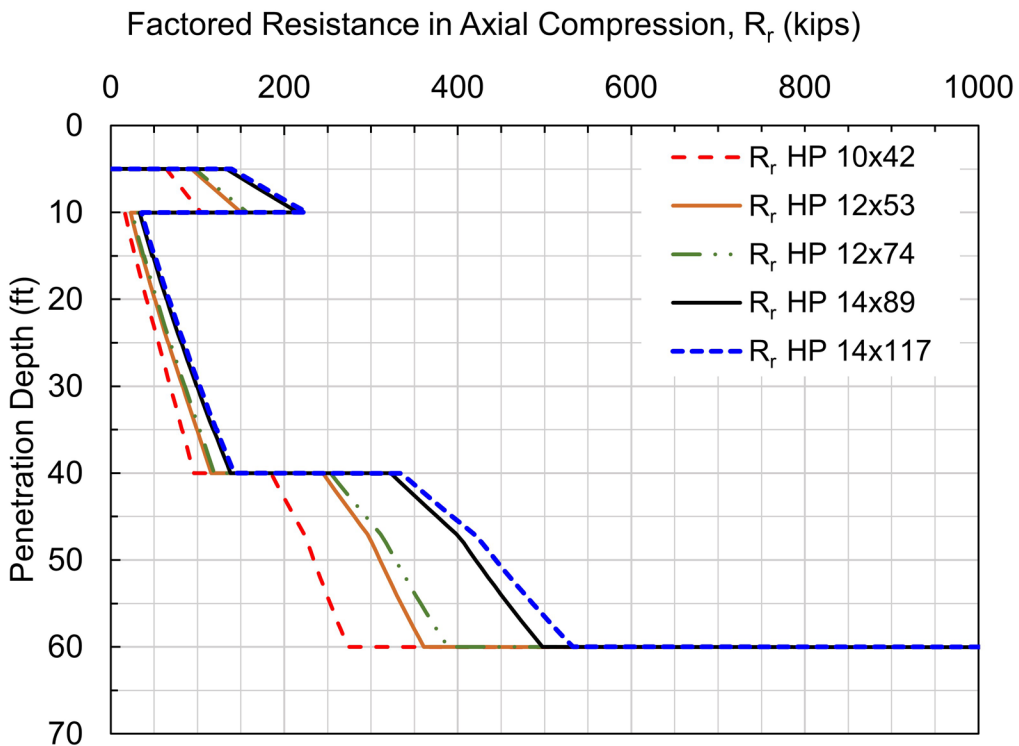


Figure D-45 Factored geotechnical resistance, R_r , in axial compression based on field determination by dynamic testing 2% of the piles, $\phi_{dyn}=0.65$.

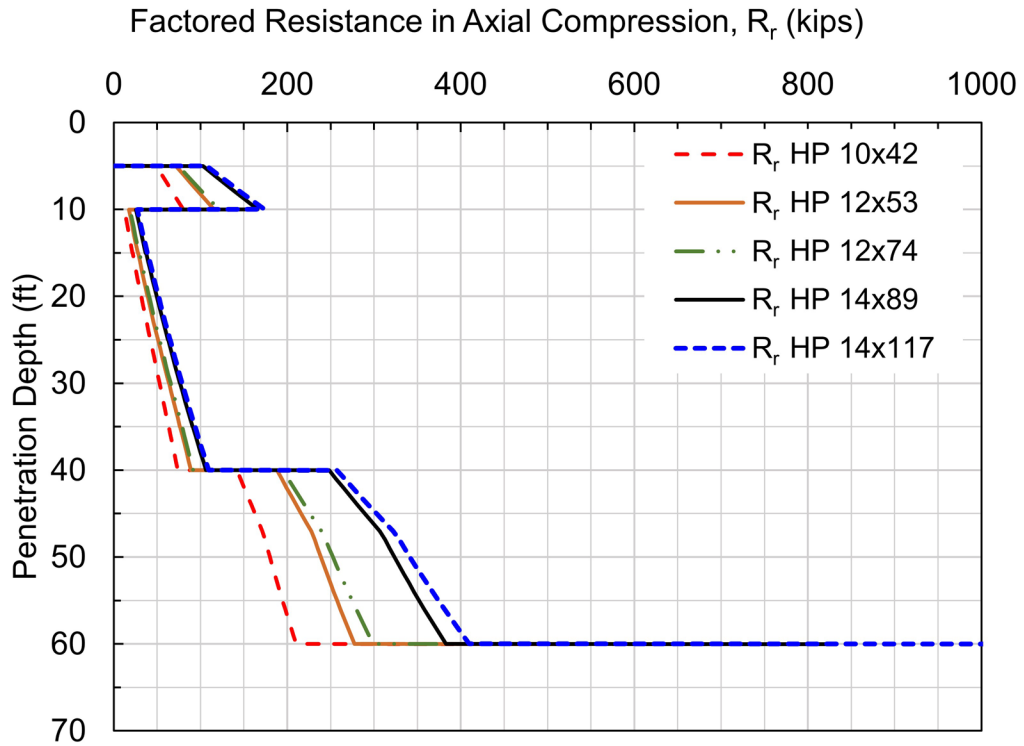


Figure D-46 Factored geotechnical resistance, R_r , in axial compression based on field determination by wave equation analysis, $\phi_{dyn}=0.50$.

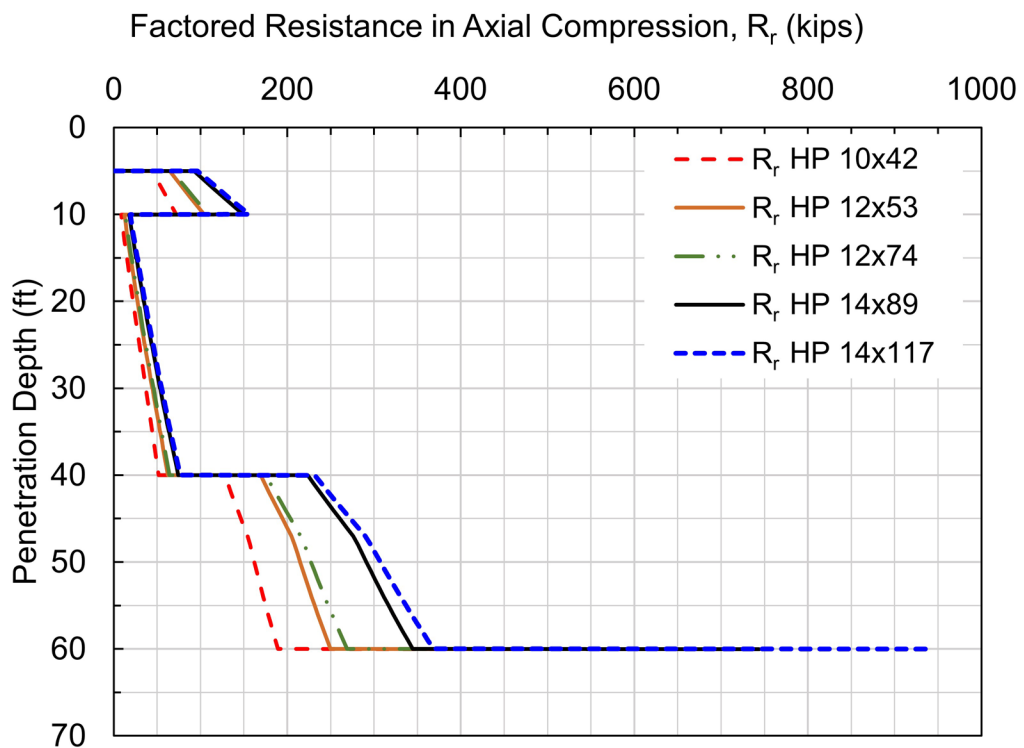


Figure D-47 Factored geotechnical resistance, R_r , in axial compression based on determination using Nordlund Method static analysis, $\phi_{stat}=0.45$.

D.22.1.2 Geotechnical Resistance in Axial Compression at the Check Flood

The nominal geotechnical resistance in axial compression at the Extreme Event check flood must also be evaluated. In the check flood, 5 feet of channel degradation scour is anticipated. The channel degradation scour results in a change in vertical effective stress from the removal of 5 feet of channel materials. The channel degradation scour occurs above the bottom of the pile cap. Hence the channel degradation scour results in no loss of nominal resistance other than that caused by the reduction in overburden stress.

In addition to the channel degradation scour, 12 feet of local scour occurs during the design flood. This 12 feet of local scour occurring below the channel degradation scour results in the loss of frictional resistance on the upper 7 feet of the pile in the Extreme Event check flood. Hence, these two scour mechanisms require a more rigorous analysis of the nominal resistance beyond simple subtraction of the shaft resistance within the scour prism. The DrivenPiles program was once again used to calculate the shaft, toe and nominal resistance as a function of depth for each of the five candidate H-piles in the check flood.

Table D-53 presents a summary of the estimated nominal geotechnical resistance considering scour at the check flood for the HP 12x74 pile section. These results are also presented graphically in Figure D-48. The depth indicated in both Table D-53 and Figure D-48 is referenced from the bottom of footing (EL 275 feet), which is 10 feet below the original ground surface elevation (EL 285 feet).

The nominal shaft, nominal toe, and nominal geotechnical resistance considering scour at the check flood were calculated for all of the candidate pile sections in a similar manner. These results are presented in Figure D-49.

Table D-53 Nominal Shaft, Nominal Toe and Nominal Geotechnical Resistance for
HP 12x74 at Pier 2 (check flood)

Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Geotechnical Resistance (kips)	Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Geotechnical Resistance (kips)
0.01	0.0	0.0	0.0	29.99	97.5	30.5	127.9
1	0.0	0.0	0.0	30.01	97.6	30.8	128.4
2	0.0	0.0	0.0	31	102.6	30.8	133.5
3	0.0	0.0	0.0	32	107.7	30.8	138.6
4	0.0	0.0	0.0	33	112.8	30.8	143.7
5	0.0	0.0	0.0	34	118.0	30.8	148.8
6	0.0	0.0	0.0	34.99	123.0	30.8	153.8
6.99	0.0	0.0	0.0	35.01	123.1	31.4	154.5
7.01	0.0	183.1	183.1	36	128.2	31.4	159.5
8	1.9	199.6	201.4	37	133.3	31.4	164.6
9	3.9	216.2	220.2	38	138.4	31.4	169.7
9.99	6.1	232.8	238.8	39	143.5	31.4	174.8
10.01	6.1	28.6	34.7	39.99	148.5	31.4	179.9
11	9.9	28.6	38.5	40.01	148.6	238.0	386.6
12	13.9	28.6	42.5	41	155.7	243.1	398.7
13	18.0	28.6	46.6	42	162.9	248.2	411.1
14	22.2	28.6	50.8	43	170.3	253.3	423.6
14.99	26.5	28.6	55.1	44	177.9	258.4	436.3
15.01	26.6	29.6	56.2	45	185.6	263.5	449.1
16	30.7	29.6	60.3	46	193.5	268.6	462.1
17	35.0	29.6	64.6	47	201.5	273.8	475.2
18	39.4	29.6	69.0	48	209.6	275.4	485.0
19	43.9	29.6	73.5	49	217.9	275.4	493.3
19.99	48.5	29.6	78.1	50	226.4	275.4	501.8
20.01	48.6	30.0	78.6	51	235.0	275.4	510.4
21	53.0	30.0	83.0	52	243.8	275.4	519.1
22	57.6	30.0	87.6	53	252.7	275.4	528.1
23	62.4	30.0	92.4	54	261.8	275.4	537.1
24	67.2	30.0	97.2	55	271.0	275.4	546.4
24.99	72.2	30.0	102.2	56	280.4	275.4	555.7
25.01	72.3	30.5	102.7	57	289.9	275.4	565.2
26	77.0	30.5	107.5	58	299.6	275.4	574.9
27	82.0	30.5	112.4	59	309.4	275.4	584.7
28	87.0	30.5	117.5	59.99	319.3	275.4	594.6
29	92.2	30.5	122.7	60.01	319.4	1042.6	1362.0

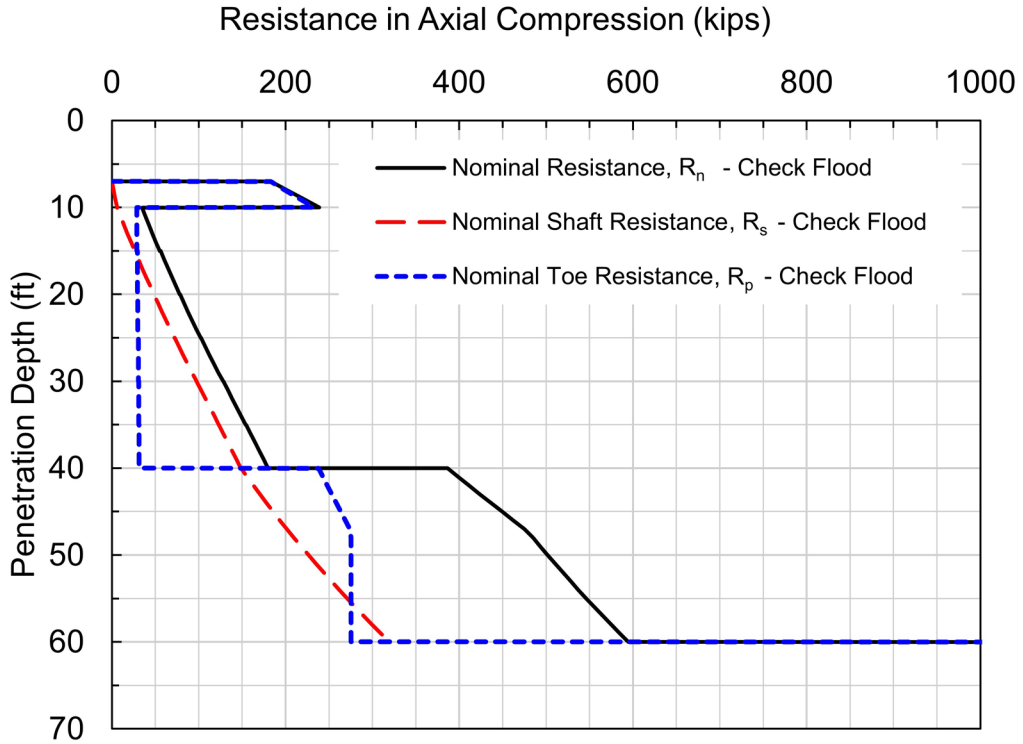


Figure D-48 Nominal geotechnical resistance in axial compression versus pile penetration depth for HP 12x74 at Pier 2 (check flood).

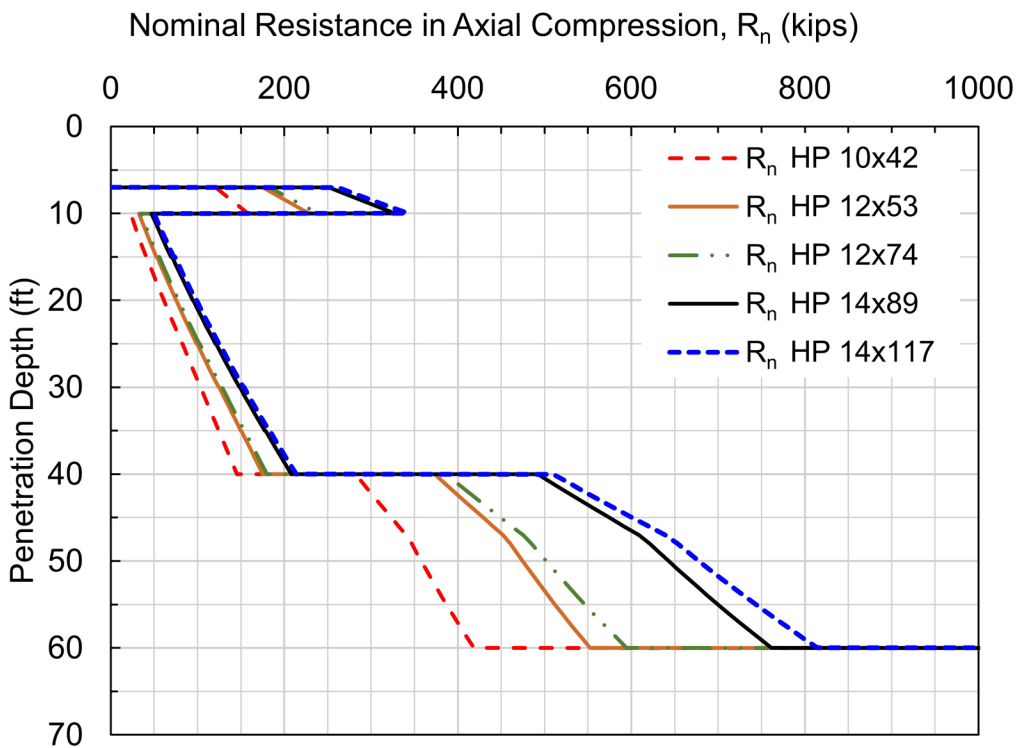


Figure D-49 Nominal geotechnical resistance in axial compression versus penetration depth for all candidate pile sections at Pier 2 (check flood).

Figure D-50 presents a design chart of the nominal and factored geotechnical resistance in axial compression versus depth during the check flood for the HP 12x74 candidate H-pile section. The design chart includes the nominal geotechnical resistance as well as the factored geotechnical resistance based on several resistance determination methods. The factored geotechnical resistance versus penetration depth is plotted for resistance determination by a static load test with dynamic testing of 2% of the piles ($\phi_{dyn}=0.80$), by dynamic testing of at least two piles per site condition, but no less than 2% of the production piles, with signal matching ($\phi_{dyn}=0.65$), and by wave equation analysis ($\phi_{dyn}=0.50$). Also included is the factored geotechnical resistance based on the static analysis method used in this design example. For this, ($\phi_{stat}=0.45$) was applied to the cohesionless soils as a result of using the Nordlund method while ($\phi_{stat}=0.35$) was applied to the cohesive soils as a result of using the alpha method. For a single pile section (HP 12x74), this figure illustrates the effects the various resistance determination methods have on the pile length required for a given factored resistance, the factored resistance available from a given pile section, and the potential impact of these factors on the number of piles needed to resist axial compression loads.

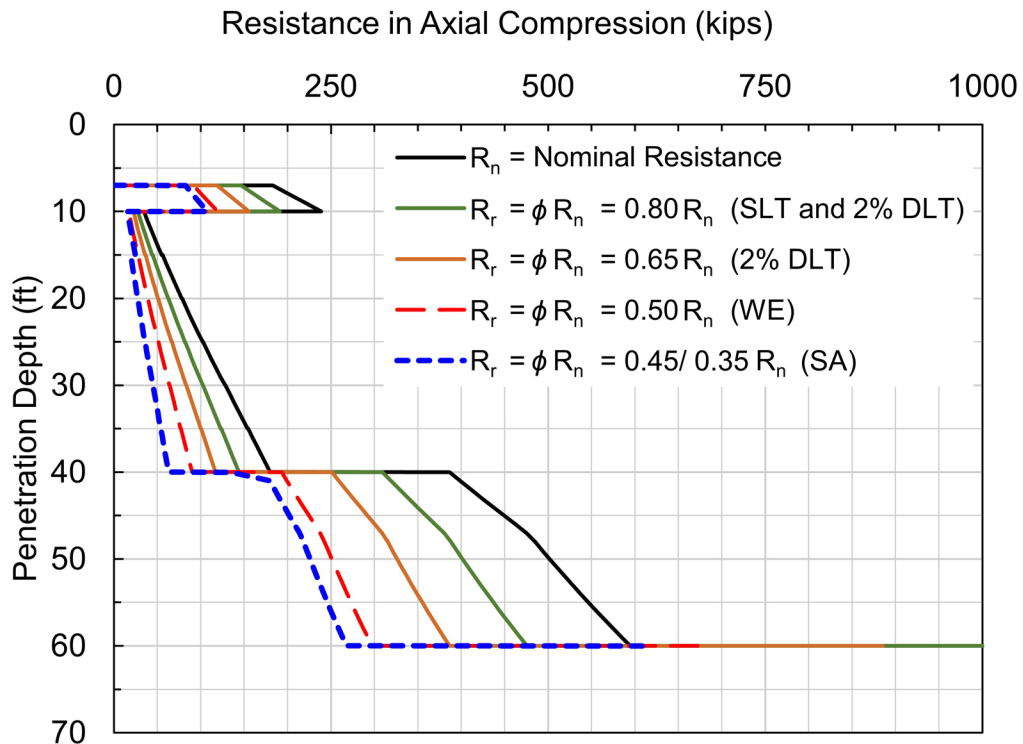


Figure D-50 Design chart of nominal and factored geotechnical resistance in axial compression versus pile penetration depth for HP 12x74 at the Pier 2 (check flood).

The factored geotechnical resistance in axial compression at the check flood for all candidate pile sections is presented in Figures D-51 through D-54. The presented factored geotechnical resistances are based upon on the field determination method or static analysis method used. Accordingly, these figures will be used to assess the effects of the various determination methods on pile length, pile section selection, and the required number of piles to resist axial compression loads.

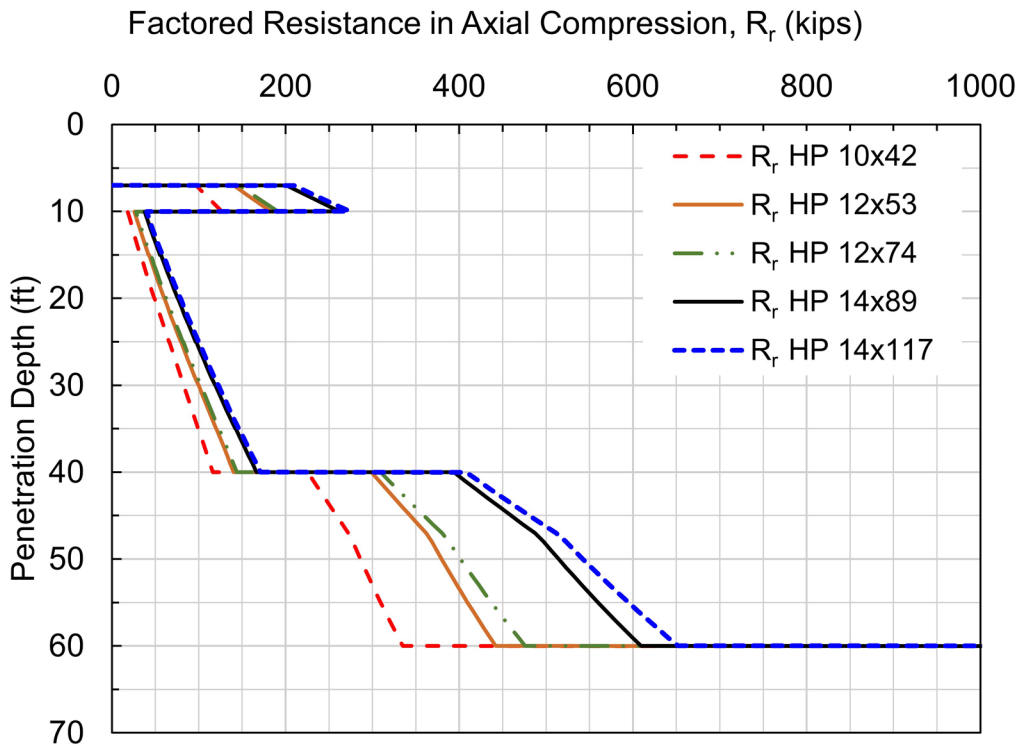


Figure D-51 Factored geotechnical resistance, R_r , in axial compression based on field determination by static load test and dynamic testing 2% of the piles, $\phi_{dyn}=0.80$.

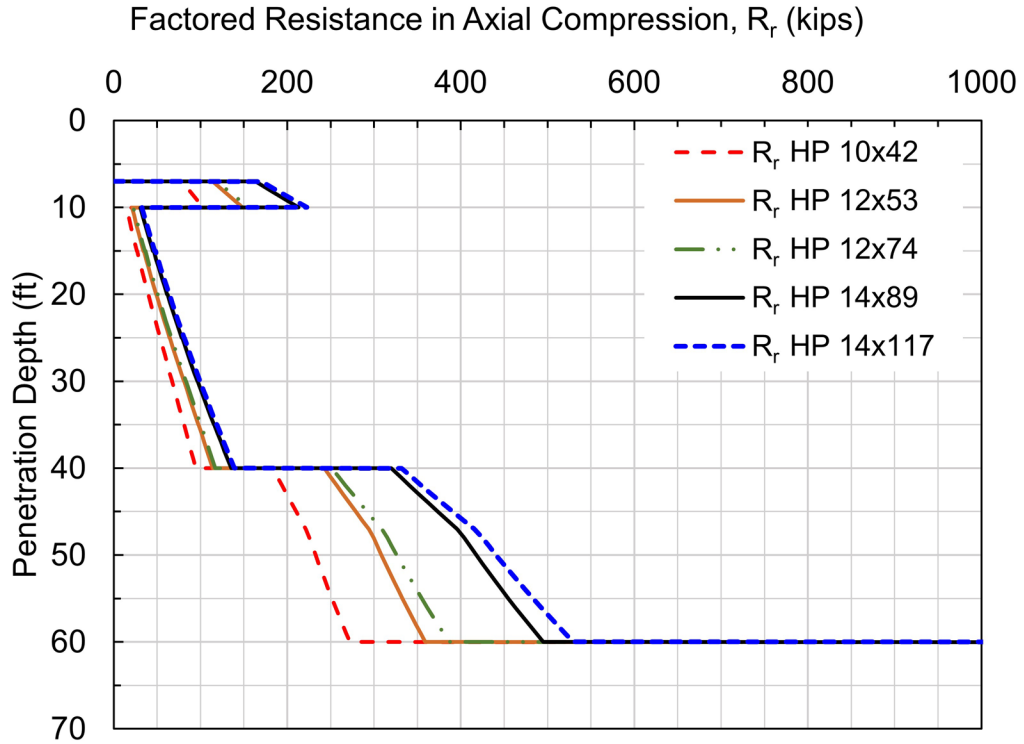


Figure D-52 Factored geotechnical resistance, R_r , in axial compression based on field determination by dynamic testing 2% of the piles, $\phi_{dyn}=0.65$.

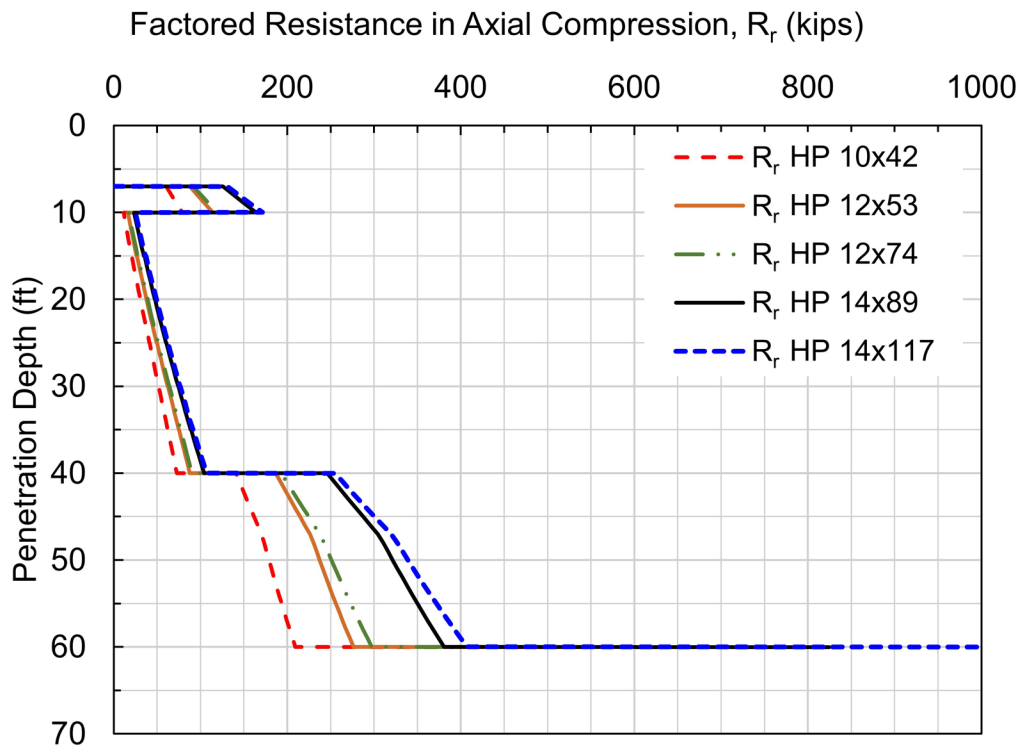


Figure D-53 Factored geotechnical resistance, R_r , in axial compression based on field determination by wave equation analysis, $\phi_{dyn}=0.5$.

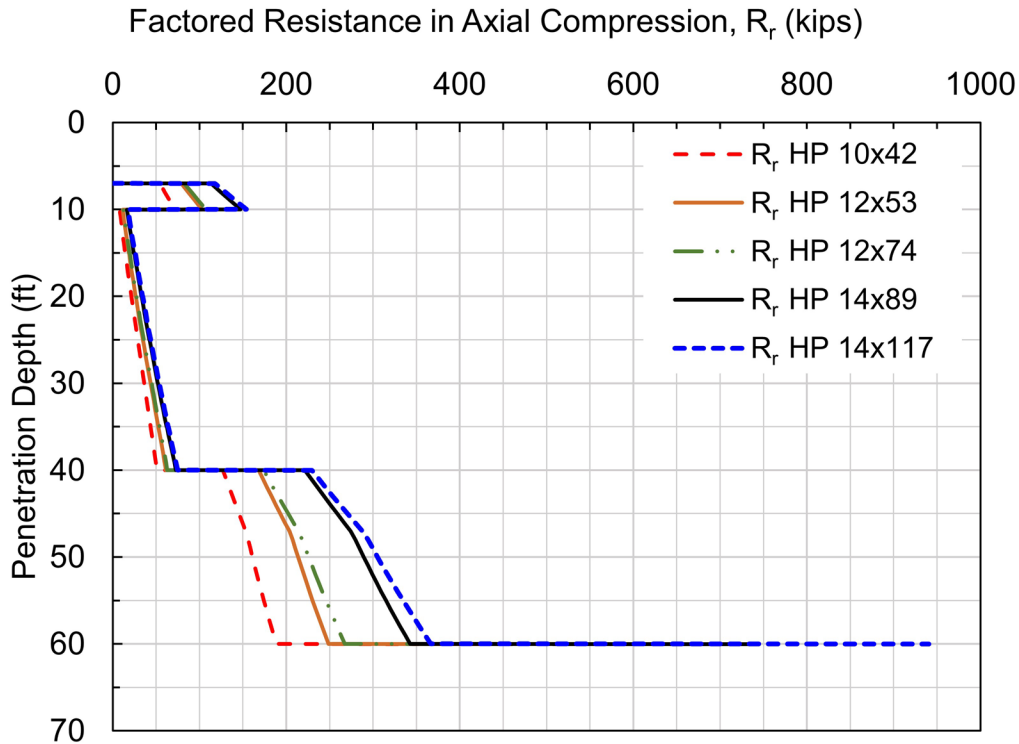


Figure D-54 Factored geotechnical resistance, R_r , in axial compression based on determination using static analysis, $\phi_{stat}=0.45$ or $\phi_{stat}=0.35$.

D.22.2 Geotechnical Resistance in Axial Tension

The nominal and factored geotechnical resistances in axial tension (uplift) are now calculated. The calculations were performed using the DrivenPiles program which utilized the Nordlund method in cohesionless soil layers and the alpha method in cohesive soil layers. For geotechnical resistance in axial tension, only the shaft resistance is considered. Therefore, the shaft resistance results calculated for the geotechnical resistance in axial compression can be re-used for axial tension. These tabular outputs of the nominal shaft resistance versus depth were previously presented in Table D-52 for the design flood and Table D-53 for the check flood.

D.22.2.1 Geotechnical Resistance in Axial Tension at the Design Flood

The nominal geotechnical resistance in axial tension at the design flood must be evaluated. In the design flood, 5 feet of channel degradation scour is anticipated. The channel degradation scour results in a change in vertical effective stress from the removal of 5 feet of channel materials. The channel degradation scour occurs above the bottom of the pile cap. Hence the channel degradation scour results in no loss of nominal resistance other than that caused by the reduction in overburden stress.

In addition to the channel degradation scour, 10 feet of local scour occurs during the design flood. This 10 feet of local scour occurring below the channel degradation scour results in the loss of frictional resistance on the upper 5 feet of the pile in the design flood event. Hence, these two scour mechanisms require a more rigorous analysis of the nominal resistance beyond simple subtraction of the shaft resistance within the scour prism.

The nominal geotechnical resistance in axial tension at the design flood (and associated factored geotechnical resistance based upon resistance determination method) will therefore be used to evaluate axial tension at the Strength limit state.

Figure D-55 presents plots of the nominal shaft resistance versus penetration depth for all the candidate pile sections during the design flood. As outlined in Section 7.2.3.2.1, the factored uplift resistance for a single pile is the shaft resistance multiplied by the appropriate resistance determination resistance factor, ϕ_{up} . Figure D-56 presents a design chart of the nominal and factored geotechnical resistance in axial tension for the HP 12x74 H-pile section. A resistance factor of 0.60 is used when the uplift resistance is determined by a static load test and 0.50 is used when determined by a dynamic test with signal matching. If the axial tension resistance is evaluated using static analysis methods, a resistance factor of 0.35 is applied to the nominal shaft resistance determined by the Nordlund static analysis method in cohesionless soil layers, and a resistance factor of 0.25 is applied to the nominal shaft resistance determined by the alpha static analysis method in cohesive soil layers.

For a single pile section (HP 12x74), Figure D-56 illustrates the effects the various resistance determination methods have on the pile length required for a given factored resistance, the factored resistance available from a given pile section, and the potential impact of these factors on the number of piles needed to resist axial tension loads.

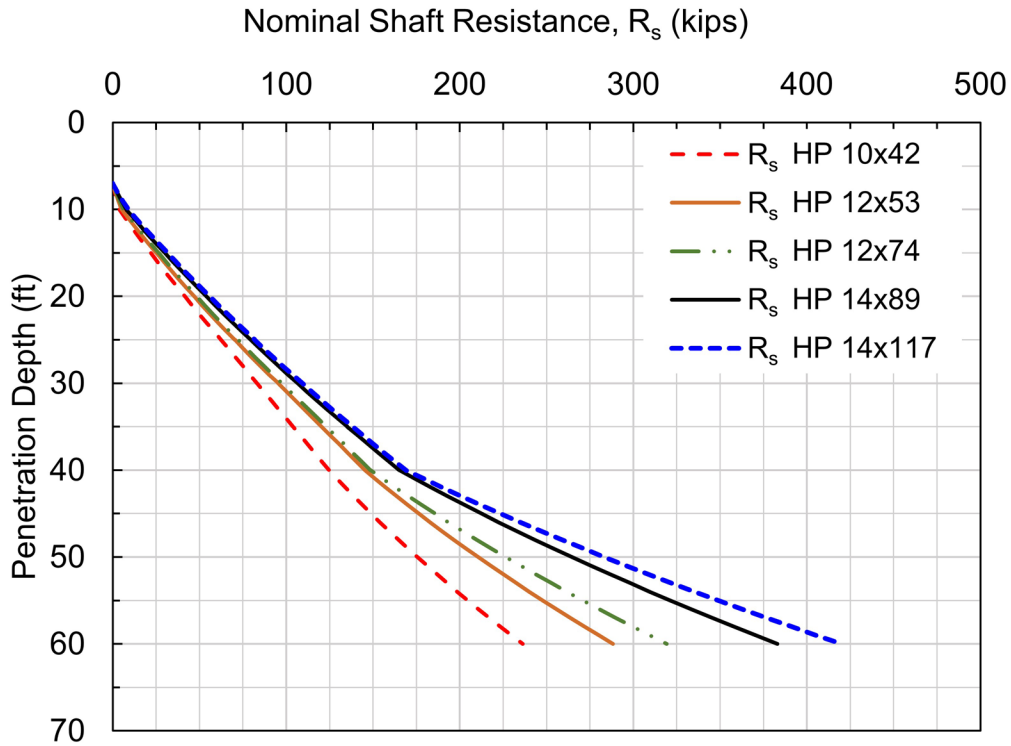


Figure D-55 Nominal shaft resistance versus pile penetration depth for all candidate pile sections at Pier 2 (design flood).

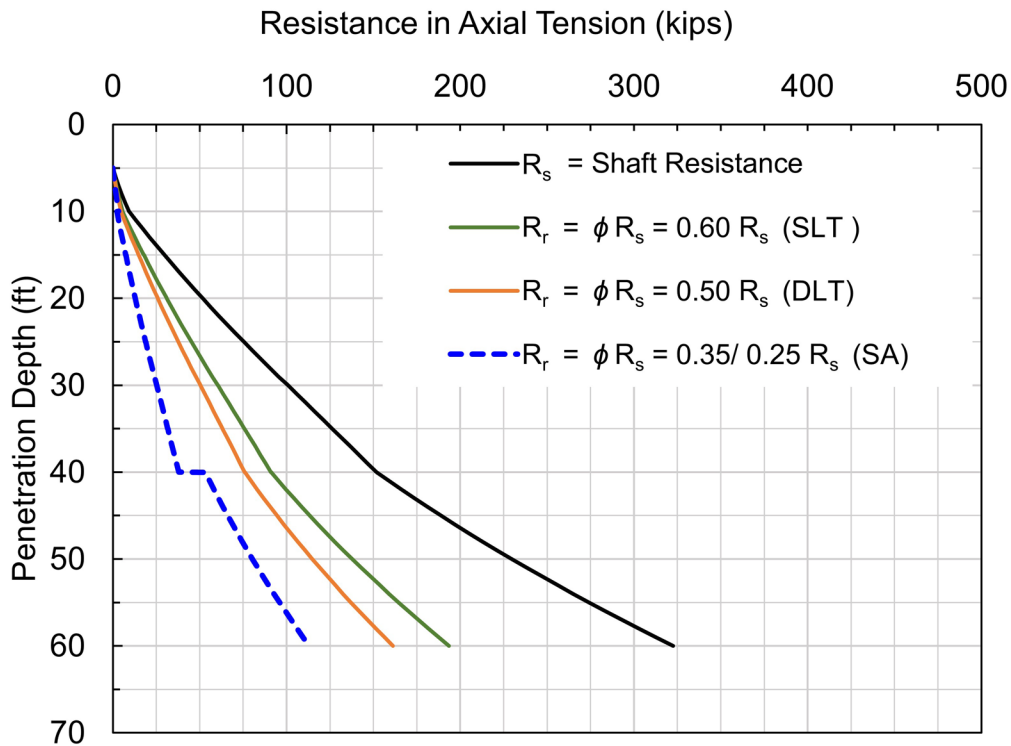


Figure D-56 Design chart of nominal and factored geotechnical resistance in axial tension versus pile penetration depth for HP 12x74 at the Pier 2 (design flood).

Figures D-57 to D-59 present the factored geotechnical resistance in axial tension versus penetration depth for all candidate pile sections at the design flood based on the resistance determination method. For all the candidate pile sections, these figures illustrate the effects the various resistance determination methods have on the pile length required for a given factored resistance, the factored resistance available from a given pile section, and the potential impact of these factors on the number of piles needed to resist axial tension loads.

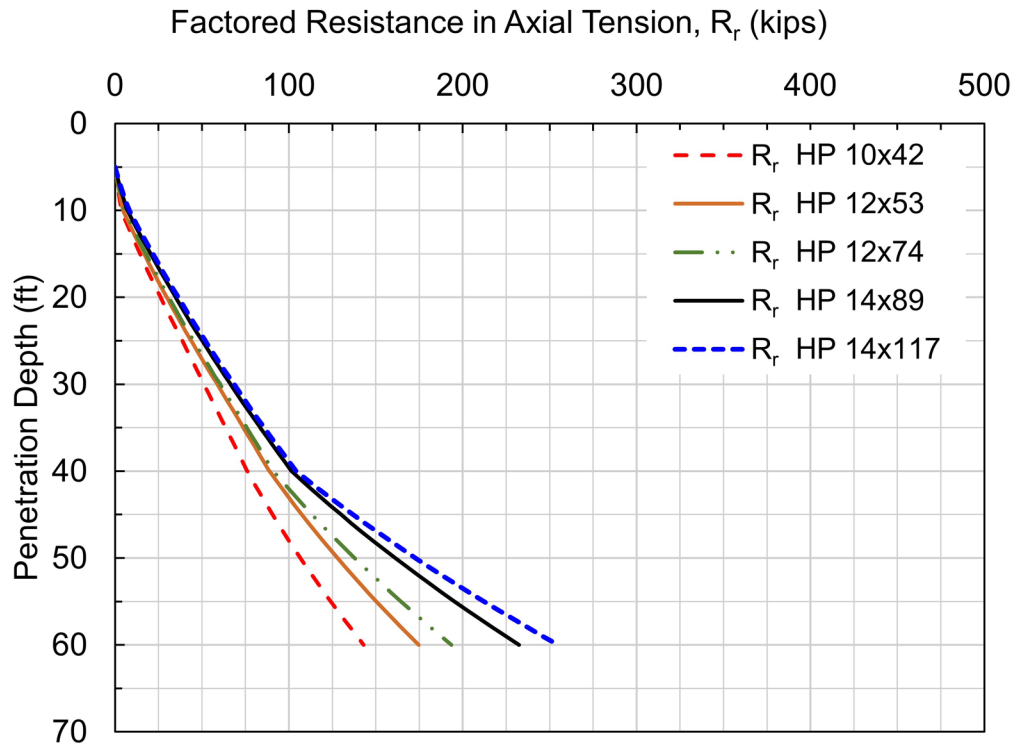


Figure D-57 Factored geotechnical resistance, R_r , in axial tension based on field determination by static load test, $\phi_{dyn}=0.60$ (design flood).

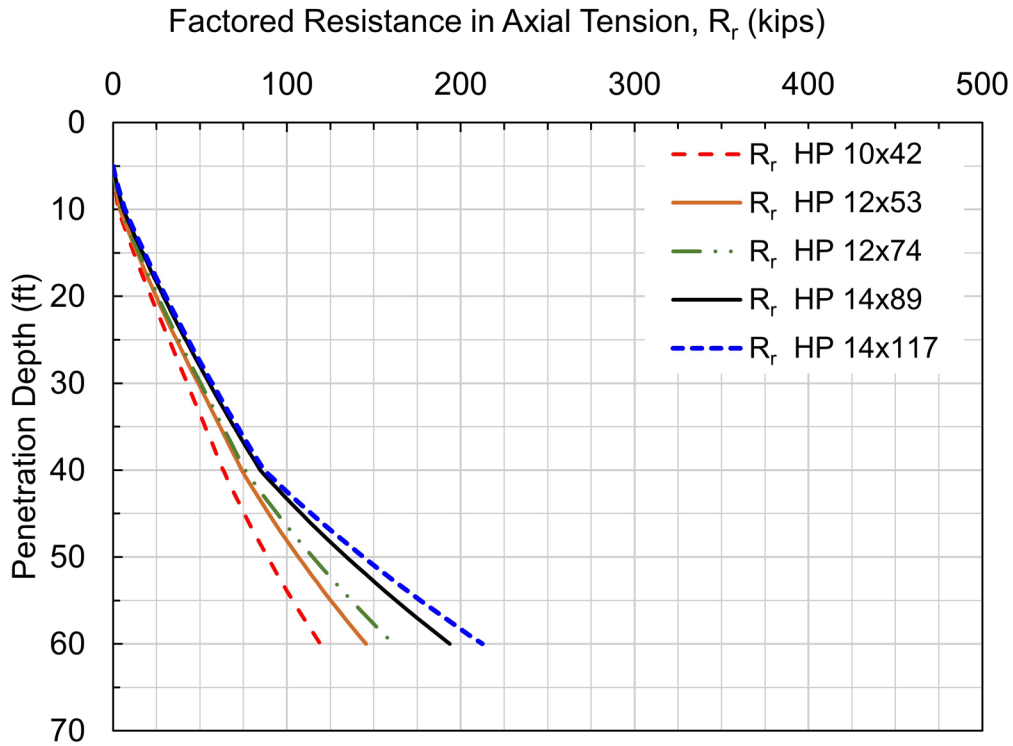


Figure D-58 Factored geotechnical resistance, R_r , in axial tension based on field determination by dynamic testing with signal matching, $\phi_{dyn}=0.50$ (design flood).

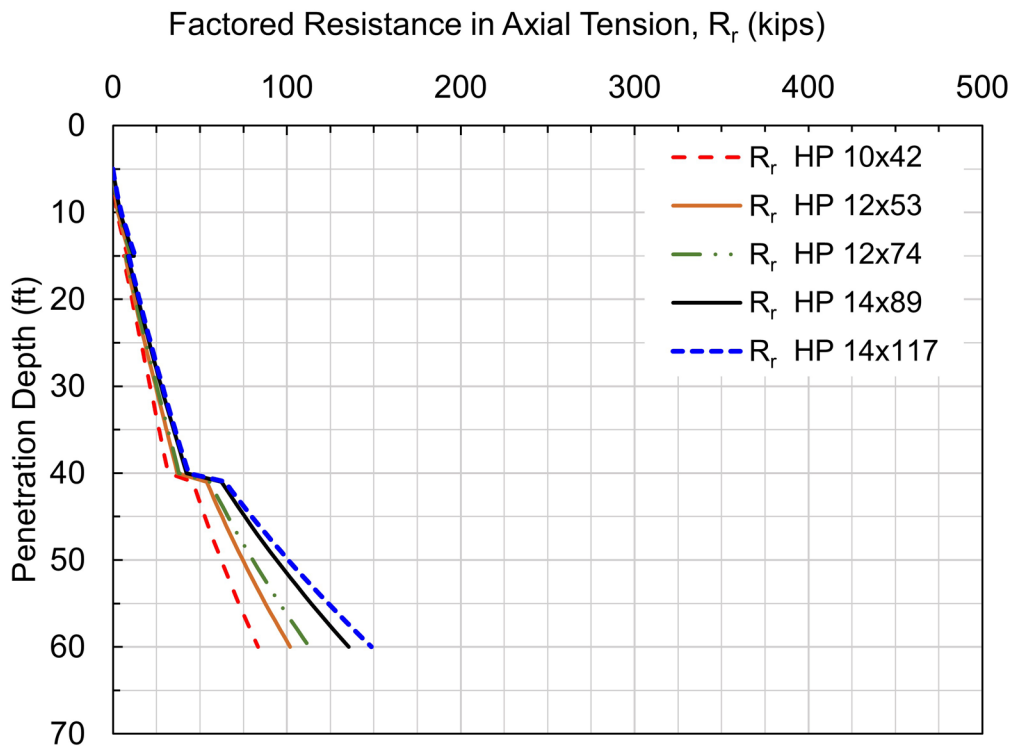


Figure D-59 Factored geotechnical resistance, R_r , in axial tension based on determination using static analysis, $\phi_{stat}=0.35$ or $\phi_{stat}=0.25$ (design flood).

D.22.2.2 Geotechnical Resistance in Axial Tension at the Check Flood

The nominal geotechnical resistance in axial tension at the Extreme Event check flood must also be evaluated. In the check flood, 5 feet of channel degradation scour is anticipated. The channel degradation scour results in a change in vertical effective stress from the removal of 5 feet of channel materials. The channel degradation scour occurs above the bottom of the pile cap. Hence the channel degradation scour results in no loss of nominal resistance other than that caused by the reduction in overburden stress.

In addition to the channel degradation scour, 12 feet of local scour occurs during the check flood. This 12 feet of local scour occurring below the channel degradation scour results in the loss of frictional resistance on the upper 7 feet of the pile in the check flood Extreme Event. Hence, these two scour mechanisms require a more rigorous analysis of the nominal resistance beyond simple subtraction of the shaft resistance within the scour prism.

The nominal geotechnical resistance in axial tension at the check flood (and associated factored geotechnical resistance based upon resistance determination method) will therefore be used to evaluate axial tension at the Extreme Event limit state.

Figure D-60 presents the nominal shaft resistance versus penetration depth for all the candidate pile sections. As outlined in Section 7.2.3.2.1 of Chapter 7, the factored uplift resistance for a single pile is the shaft resistance multiplied by the appropriate field determination resistance factor, ϕ_{up} . Figure D-61 presents a design chart of the nominal shaft resistance, R_s , and the factored geotechnical resistance, R_r , in axial tension versus pile penetration depth for the HP 12x74 H-pile section. A resistance factor of 0.60 is used when the uplift resistance is determined by a static load test, 0.50 when determined by a dynamic test with signal matching, 0.35 when determined by the Nordlund static analysis method and 0.25 when determined by the alpha static analysis method.

For a single pile section (HP 12x74), this figure illustrates the effects the various resistance determination methods have on the pile length required for a given factored resistance, the factored resistance available from a given pile section, and the potential impact of these factors on the number of piles needed to resist axial tension loads.

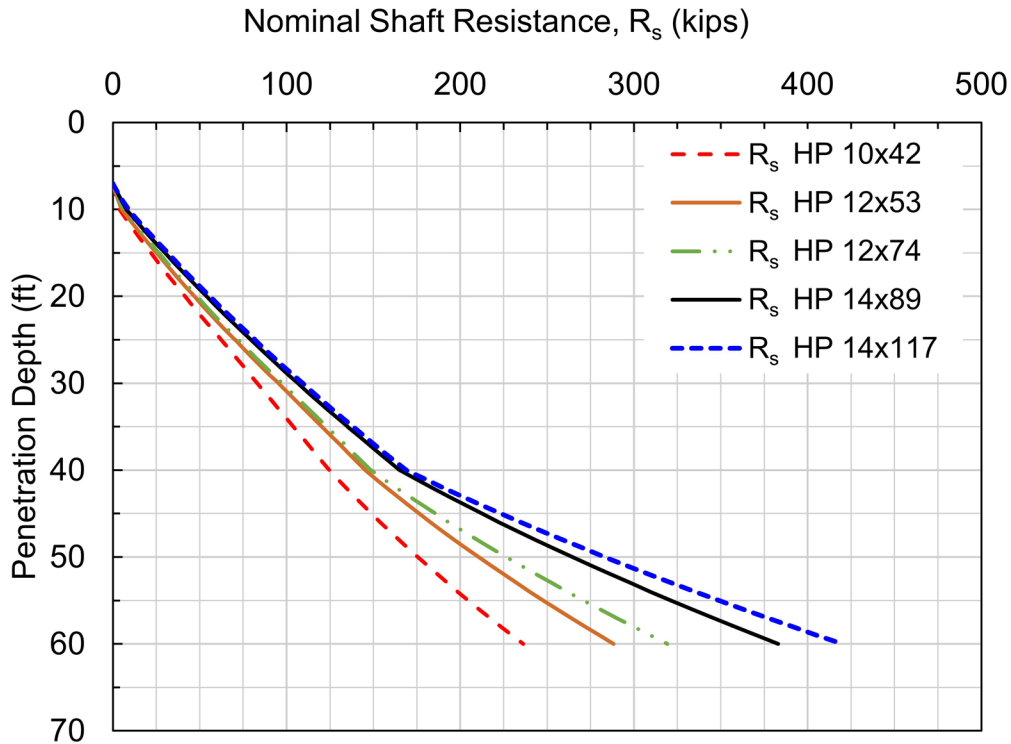


Figure D-60 Nominal shaft resistance versus pile penetration depth for all candidate pile sections at Pier 2 during Extreme Event (check flood).

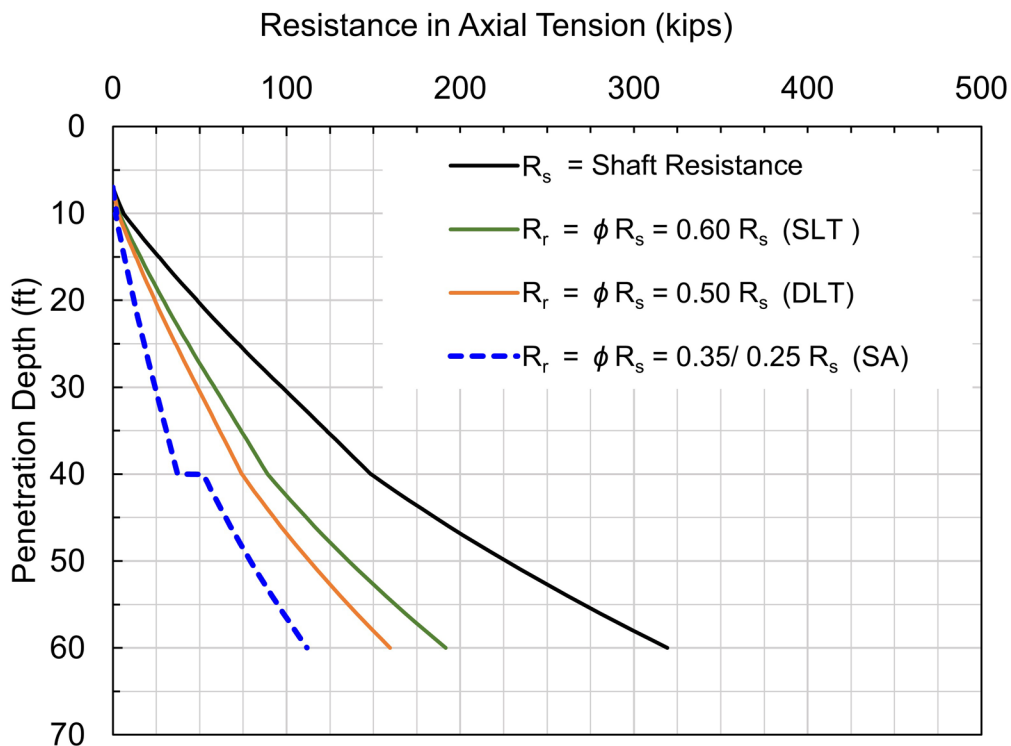


Figure D-61 Design chart of nominal and factored geotechnical resistance in axial tension versus depth for HP 12x74 at the Pier 2 during Extreme Event (check flood).

Figures D-62 to D-64 present the factored geotechnical resistance in axial tension versus penetration depth based on the field determination method for each of the candidate pile sections. For all the candidate pile sections, these figures illustrate the effects the various resistance determination methods have on the pile length required for a given factored resistance, the factored resistance available from a given pile section, and the potential impact of these factors on the number of piles needed to resist axial tension loads.

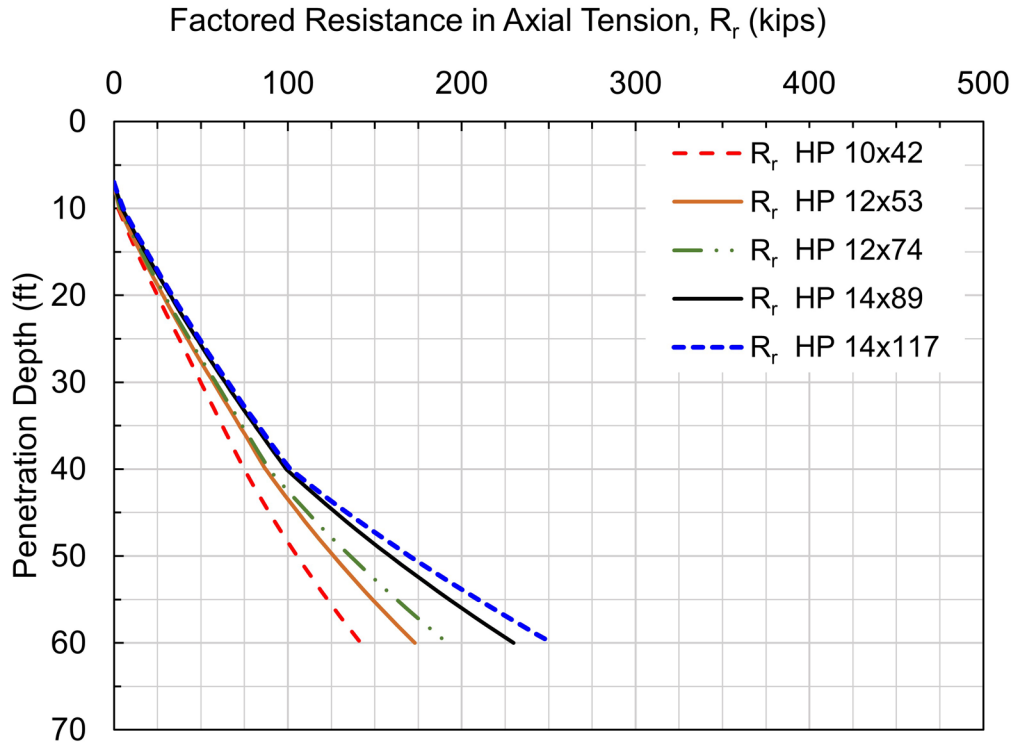


Figure D-62 Factored geotechnical resistance, R_r , in axial tension based on field determination by static load test, $\phi_{dyn}=0.60$.

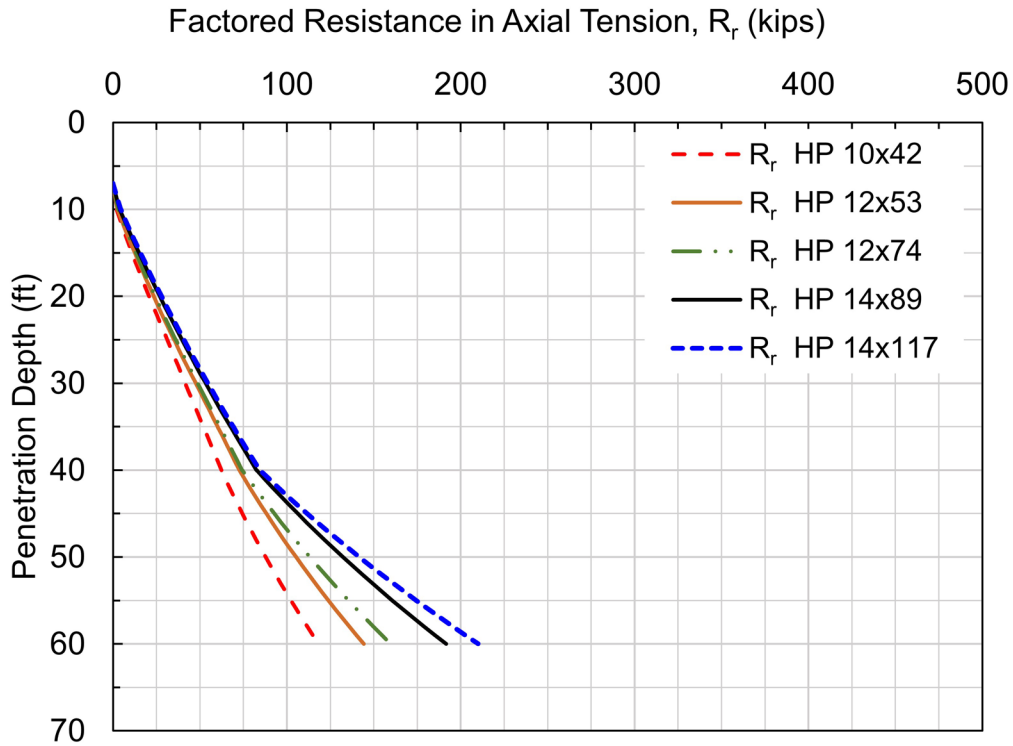


Figure D-63 Factored geotechnical resistance, R_r , in axial tension based on field determination by dynamic testing with signal matching, $\phi_{dyn}=0.50$.

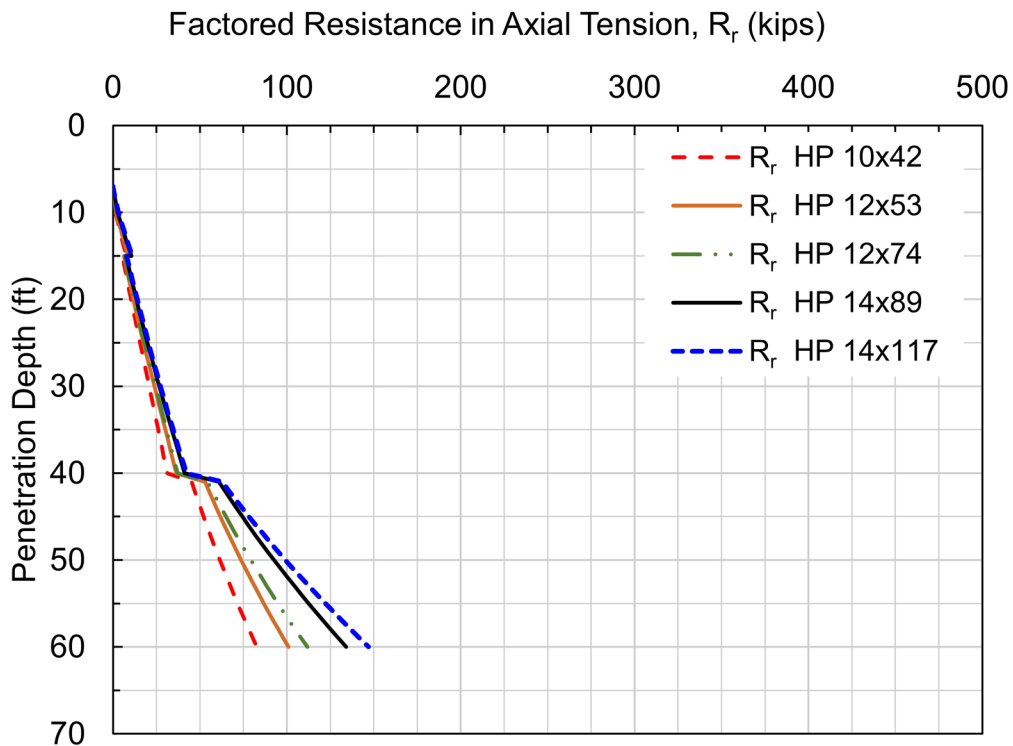


Figure D-64 Factored geotechnical resistance, R_r , in axial tension based on determination using static analysis, $\phi_{stat}=0.35$ or $\phi_{stat}=0.25$.

D.22.3 Preliminary Pile Drivability Assessment

Preliminary assessments of pile drivability are now performed at this stage of the design. A drivability check at this time is essential to assess the constructability of candidate pile types and/or sections and to eliminate sections with insufficient drivability. Section 12.4 provides a detailed discussion of wave equation drivability analyses and their applications.

A candidate pile section must be capable of being driven to the penetration depth necessary to achieve the nominal geotechnical resistance in axial compression and tension, to a penetration depth necessary to satisfy lateral load demands, as well as a penetration depth necessary to satisfy deformation requirements. A suitably sized pile hammer must be capable of driving the pile to its established minimum penetration depth and to the nominal resistance at a reasonable blow count without exceeding material stress limits. As detailed in Chapter 12, the blow count should be between 30 and 120 blows per foot at the nominal resistance. If the pile cannot be driven within these requirements, a larger pile hammer, a pile section with greater impedance, or pile installation aids such as predrilling or jetting may be required to satisfy or improve drivability.

Driving stresses during pile installation should remain below the driving stress limits tied to pile type and material strength. For the candidate steel H-piles, compression driving stress limits are given by Equation 8-33. As per ASTM A-572 requirements, new steel H-piles are rolled with a minimum yield stress of 50 ksi.

The driving stress limit, σ_{dr} , for candidate pile sections is then calculated as follows:

$$\sigma_{dr} = \phi_{da} (0.9 F_y) \quad [\text{Eq. 8-33}]$$

Where: ϕ_{da} = resistance factor, 1.0 for steel piles.
 F_y = yield stress, 50 ksi.

Therefore, the driving stress limit, σ_{dr} , is 45 ksi.

$$\sigma_{dr} = (1.0)(0.9 (50 \text{ ksi})) = 45 \text{ ksi}$$

Drivability analyses were performed for all five candidate pile sections. Since the specific pile hammer is often unknown at this point in the design, a reasonably sized, commonly available single acting diesel hammer was chosen for each of the candidate pile sections. As noted in Section 15.19, a hammer having a ram weight

of 1 to 2% of the larger of the required nominal resistance or required nominal driving resistance often provides a reasonable initial estimate of hammer size for wave equation analysis. Table D-54 summarizes the factored structural resistance in axial compression, P_r , and corresponding minimum and maximum nominal driving resistance associated with full section utilization and the full range of field determination methods resistance factors (static load test and dynamic testing ($\phi_{dyn}=0.80$), and FHWA modified Gates dynamic formula ($\phi_{dyn}=0.40$)). Given that full utilization of the structural section is uncommon, a reasonable initial estimate of the hammer size for a wave equation drivability analysis is 1% of the minimum R_{ndr} . Driving stress limits would likely be exceeded by choosing a significantly larger pile hammer. For each pile hammer, the wave equation default values were used for the helmet weight, hammer cushion materials, and the cushion material properties.

Table D-54 Summary of Pile Hammers Used in Drivability Analyses

Pile Section	Pile Cross Sectional Area (in ²)	Factored Structural Resistance, P_r (kips)	Minimum R_{ndr} $\phi_{dyn}=0.80$ (kips)	Maximum R_{ndr} $\phi_{dyn}=0.40$ (kips)	Ram Weight 1% of Min R_{ndr} (%)	Diesel Model	Ram Weight (kips)	Rated Energy (ft-kips)
HP 10x42	12.4	309	386	773	3.86	D25-52	5.51	62.0
HP 12x53	15.5	383	478	958	4.78	D30-52	6.62	74.4
HP 12x74	21.8	544	680	1360	6.80	D36-52	7.94	89.3
HP 14x89	26.1	652	815	1630	8.15	D46-32	10.14	114.1
HP 14x117	34.4	860	1075	2150	10.07	D50-52	11.03	124.0

For the soil resistance model, the output from DrivenPiles was converted to unit shaft resistance and unit toe resistance values and then input into the wave equation program. Similar soil resistances are thereby calculated versus depth by both the static analysis and wave equation analysis programs.

The dynamic soil properties for each soil layer were chosen in accordance with wave equation program recommendations. Selection of soil quake and damping parameters is discussed in Section 12.6.7. Pile driving at Pier 2 will commence at the bottom of pile cap excavation of Elevation 270 feet. Therefore dynamic soil properties for Layer 1 are not required for the preliminary drivability analyses. For the Pier 2 soil profile, a setup up factor of 2.0 was selected for the very stiff silty clay comprising Layer 3, while no setup is expected in the extremely dense gravel of Layer 2 or the dense gravel with sand of Layer 4. Soil setup is discussed in Section 7.2.4.2 and a summary of typical soil setup factors is provided in Table 7-16. The dynamic properties chosen are summarized in Table D-55.

Table D-55 Dynamic Soil Properties for Pier 2 Soil Profile

Soil Layer	Pile Section	Shaft Quake (in)	Toe Quake (in)	Shaft Damping (s/ft)	Toe Damping (s/ft)	Soil Set-Up Factor
2	HP 10 x 42	0.10	0.08	0.05	0.15	1.0
2	HP 12 x 53	0.10	0.10	0.05	0.15	1.0
2	HP 12 x 74	0.10	0.10	0.05	0.15	1.0
2	HP 14 x 89	0.10	0.12	0.05	0.15	1.0
2	HP 14 x 117	0.10	0.12	0.05	0.15	1.0
3	HP 10 x 42	0.10	0.17	0.20	0.15	2.0
3	HP 12 x 53	0.10	0.20	0.20	0.15	2.0
3	HP 12 x 74	0.10	0.20	0.20	0.15	2.0
3	HP 14 x 89	0.10	0.23	0.20	0.15	2.0
3	HP 14 x 117	0.10	0.23	0.20	0.15	2.0
4	HP 10 x 42	0.10	0.08	0.05	0.15	1.0
4	HP 12 x 53	0.10	0.10	0.05	0.15	1.0
4	HP 12 x 74	0.10	0.10	0.05	0.15	1.0
4	HP 14 x 89	0.10	0.12	0.05	0.15	1.0
4	HP 14 x 117	0.10	0.12	0.05	0.15	1.0

In soils that exhibit setup, the nominal resistance will be higher than the nominal driving resistance. Therefore, a gain/loss factor of 0.5 was used to estimate the nominal driving resistance versus depth in the drivability analyses. This gain/loss factor was determined from the inverse of the highest soil setup factor within the soil model (e.g., 1 divided by 2.0 equals 0.5). A gain/loss factor of 1 would be used if it was desired to model the nominal resistance instead of the nominal driving resistance and not consider the soil strength loss during driving and any subsequent soil setup. Refer to Chapter 12 for more detailed discussion on the selection of dynamic soil properties and soil setup factors.

The DrivenPiles program calculates the nominal driving resistance which models the soil strength lost during driving as well as the geotechnical nominal resistance once setup occurs. Figure D-65 presents the shaft resistance, toe resistance, and nominal driving resistance versus pile penetration depth for the HP 12x74 H-pile section at Pier 2. These nominal driving resistances are presented numerically in Table D-56. To quantify the expected soil setup at a given pile penetration depth, the values from Table D-56 can be compared against the nominal resistance previously presented in Table D-51. Figure D-66 illustrates the significant difference between the expected nominal driving resistance and the geotechnical nominal resistance after setup for the HP 12x74 candidate pile section.

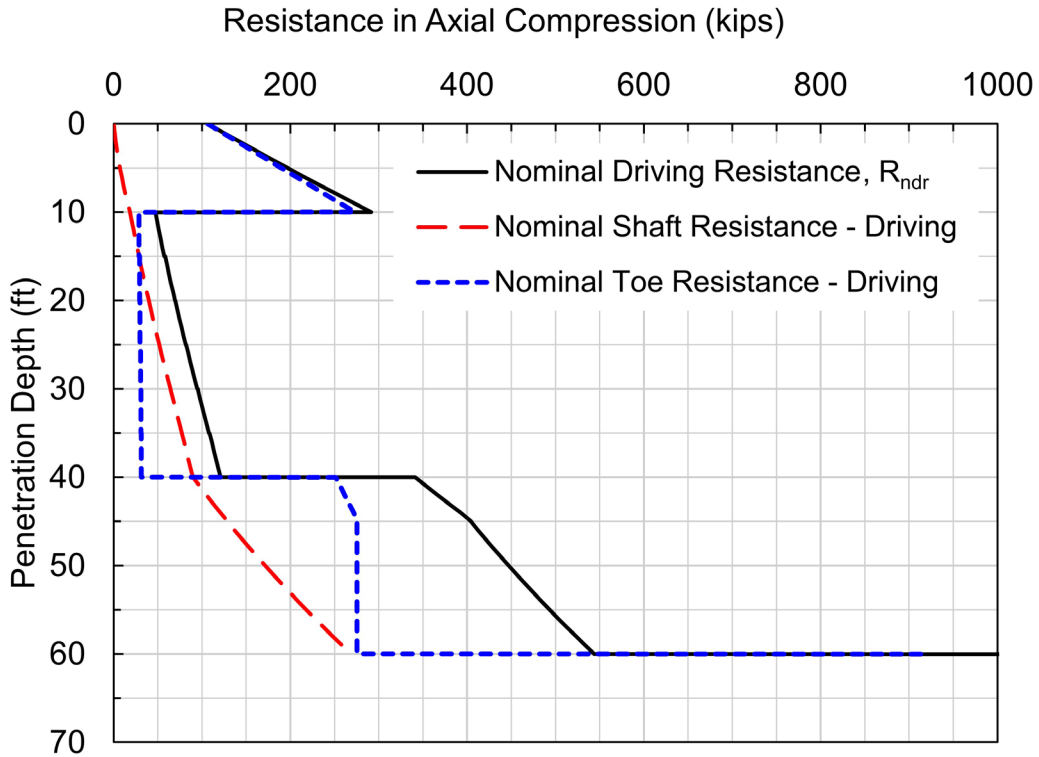


Figure D-65 Nominal driving resistance for HP 12x74 at Pier 2.

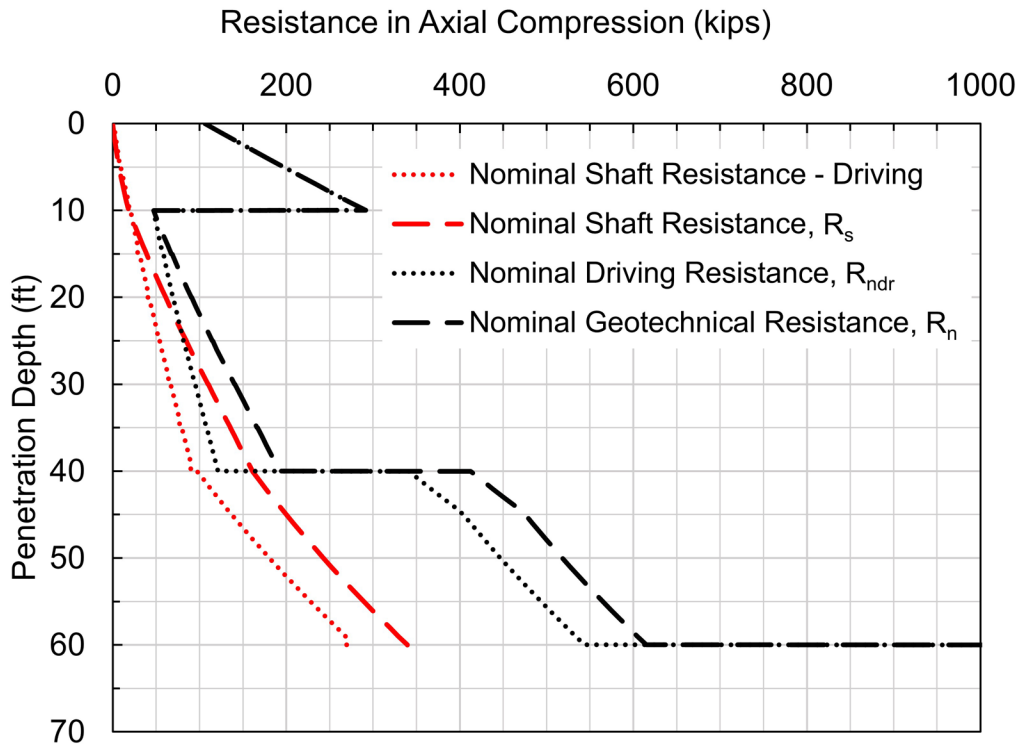


Figure D-66 Comparison of nominal driving resistance and nominal geotechnical resistance in axial compression for HP 12x74 at Pier 2.

Table D-56 Summary of Nominal Driving Resistance Versus Pile Penetration Depth for HP 12x74 at Pier 2

Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Driving Resistance (kips)	Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Driving Resistance (kips)
0.01	0.0	106.5	106.5	29.99	64.3	30.5	94.7
1	1.1	123.0	124.2	30.01	64.3	30.8	95.1
2	2.4	139.7	142.1	31	66.8	30.8	97.7
3	3.9	156.4	160.2	32	69.4	30.8	100.2
4	5.5	173.1	178.5	33	71.9	30.8	102.8
4.99	7.2	189.6	196.8	34	74.5	30.8	105.3
5.01	7.3	189.9	197.2	34.99	77.0	30.8	107.8
6	9.2	206.4	215.6	35.01	77.1	31.4	108.4
7	11.3	223.1	234.4	36	79.6	31.4	111.0
8	13.5	239.8	253.3	37	82.1	31.4	113.5
9	16.0	256.5	272.4	38	84.7	31.4	116.1
9.99	18.5	273.0	291.5	39	87.2	31.4	118.6
10.01	18.6	28.6	47.2	39.99	89.8	31.4	121.1
11	20.5	28.6	49.1	40.01	89.9	251.3	341.2
12	22.5	28.6	51.1	41	97.3	256.4	353.7
13	24.5	28.6	53.1	42	104.9	261.5	366.4
14	26.6	28.6	55.2	43	112.7	266.6	379.4
14.99	28.8	28.6	57.4	44	120.7	271.7	392.4
15.01	28.8	29.6	58.4	45	128.8	275.4	404.2
16	30.9	29.6	60.5	46	137.1	275.4	412.4
17	33.0	29.6	62.6	47	145.5	275.4	420.8
18	35.2	29.6	64.8	48	154.0	275.4	429.4
19	37.5	29.6	67.1	49	162.7	275.4	438.1
19.99	39.8	29.6	69.4	50	171.6	275.4	446.9
20.01	39.8	30.0	69.8	51	180.6	275.4	455.9
21	42.0	30.0	72.0	52	189.7	275.4	465.1
22	44.3	30.0	74.3	53	199.1	275.4	474.4
23	46.7	30.0	76.7	54	208.5	275.4	483.9
24	49.1	30.0	79.1	55	218.1	275.4	493.5
24.99	51.6	30.0	81.6	56	227.9	275.4	503.2
25.01	51.7	30.5	82.1	57	237.8	275.4	513.2
26	54.0	30.5	84.5	58	247.9	275.4	523.2
27	56.5	30.5	86.9	59	258.1	275.4	533.4
28	59.0	30.5	89.5	59.99	268.4	275.4	543.7
29	61.6	30.5	92.1	60.01	268.5	1042.6	1311.1

Graphical outputs of the preliminary drivability analyses are shown in Figure D-67. The nominal driving resistance, the blow count or pile penetration resistance, and the compression driving stress are presented versus pile penetration depth for each

of the five candidate pile sections. As previously noted, the recommended blow count limit is 120 blows per foot (10 blows per inch), and the recommended driving stress limit is 45 ksi. A circular reference marker is indicated on the blow count versus depth plot highlighting the depth where the blow count first exceeds 120 blows per foot. This marker is also shown at the same depth on the nominal driving resistance versus depth plot indicating the nominal driving resistance achieved when practical refusal driving conditions are encountered with the selected hammer in the modeled driving conditions. Similarly, the marker is shown at the same depth on the compression driving stress versus depth plot indicating the compression driving stress when practical refusal driving conditions are encountered.

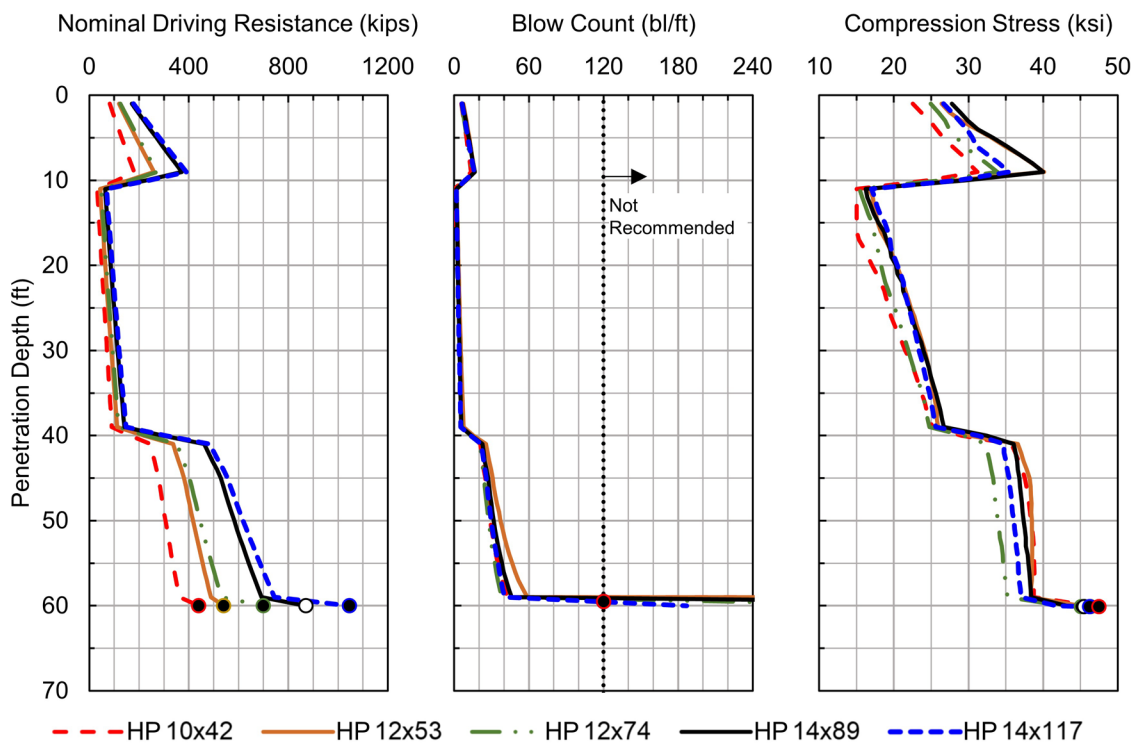


Figure D-67 Preliminary drivability results at Pier 2.

For all of the candidate pile sections and selected pile hammers, the blow count was 60 blows per foot or less before encountering hard rock at a penetration depth of 60 feet. Once bedrock was encountered, the blow count quickly transitioned to 120 blows per foot. Compression driving stresses for all candidate pile sections were also less than the 45 ksi driving stress limit prior to reaching bedrock.

If the final design requires piles be driven to hard bedrock, the driving criteria should be established to control compression stresses and prevent pile toe damage once hard rock is encountered (e.g., limit the number of blows at refusal driving

conditions). A summary of the preliminary drivability results is presented in Table D-57. The anticipated nominal resistance in this table is the expected resistance after soil setup that can be mobilized by the driving system at 10 blows per inch. For piles terminated on hard rock, a higher geotechnical nominal resistance, up to the structural resistance of the pile is actually available.

Once the estimated and/or minimum pile toe elevations are determined in Block 12 through Block 15 of the design process, the drivability results in Figure D-67 and Table D-57 should be reviewed to confirm that the candidate pile section can be driven to the estimated or required pile penetration depth, at reasonable blow counts, and with driving stress limits.

Table D-57 Summary of Preliminary Drivability Results at Pier 2

Pile Section	Pile Hammer	Fuel Setting	Pile Penetration Depth at Practical Refusal Limit (feet)	Nominal Driving Resistance at Practical Refusal Limit (kips)	Anticipated Nominal Resistance at Depth of Practical Refusal (kips)	Penetration Depth Exceeding Compression Driving Stress Limit (feet)	Maximum Compression Driving Stress (ksi)
HP 10x42	D25-52	4	60	440	500	60	46.4
HP 12x53	D30-52	4	60	540	610	60	44.9
HP 12x74	D36-52	4	60	700	770	60	42.6
HP 14x89	D46-52	4	60	870	950	60	43.2
HP 14x117	D50-52	4	60	1045	1125	60	41.8

D.23 Block 11: Pier 2 – Estimate Preliminary Number of Piles, Preliminary Pile Group Size, and Resolve Individual Pile Loads for All Limit States

The structural engineer has provided the anticipated loads for the controlling limit states at Pier 2. These limit state loads are restated in Table D-58. The Strength V limit state loads are used to evaluate geotechnical resistance in axial compression and tension, as well as for lateral loading. Although loads at Service I limit state were provided by the structural engineer, live loads should be removed when evaluating vertical deformation. Moreover, the Service I without live load includes only unfactored permanent loads such as the superstructure and wearing surface, pile cap and stem, utilities and vertical earth pressure among others. The Extreme Event II limit state must also be evaluated at this substructure location to consider effects of the check flood and associated loads.

Table D-58 Limit State Loads on Pier 2

Limit State	Q (kips)	V _{ux} (kips)	V _{uy} (kips)	M _{ux} (k-ft)	M _{uy} (k-ft)
Strength V	3456	18	109	-2981	3982
Service I	2682	15	100	-2749	2997
Service I, without LL	2172	0	0	0	0
Extreme Event II	3023	30	23	-797	1273

Pier construction will include a cofferdam and excavation of the existing geomaterials. The Agency practice for pier construction in a river is to use a substructure design with the smallest footprint. A minimum center to center pile spacing of 3 pile diameters is typical. Three potential pile group configurations for Pier 2 are under consideration. These are identified as Group Configurations 4, 5, and 6, respectively in Table D-59. Each group configuration has 5 rows of piles in the transverse direction. In the longitudinal direction, Group Configuration 4 has 3 rows of piles while Group Configurations 5 and 6 have 4 rows of piles. Furthermore, the distance from the center of any exterior pile to the pile cap edge must be at least 1.25 feet in both the transverse (x) and longitudinal (y) direction.

Table D-59 Potential Pile Group Configurations

Group Configuration	Piles per Row Y dir	Piles per Row X dir	Total Number of Piles	S _{bx} * (feet)	Total Footing Length (feet)	S _{by} * (feet)	Total Footing Width (feet)
4	3	5	15	3.0	14.5	3.0	8.5
5	4	5	20	3.0	14.5	3.0	11.5
6	4	5	20	5.0	22.5	4.0	14.5

* S_{bx} and S_{by} are illustrated in Figure D-68.

The following example calculation considers Group Configuration 6 and all applicable loads. For this alternative, 5 piles per row in the X direction and 4 piles per row in the Y direction are proposed. Thus the transverse pile spacing, S_{bx}, is 3'-0" and the total footing length is 14'-6". The longitudinal pile spacing, S_{by}, is 3'-0" and the total footing width is 11'-6". Figure D-68 shows the layout for the Group Configuration 6 pile cap.

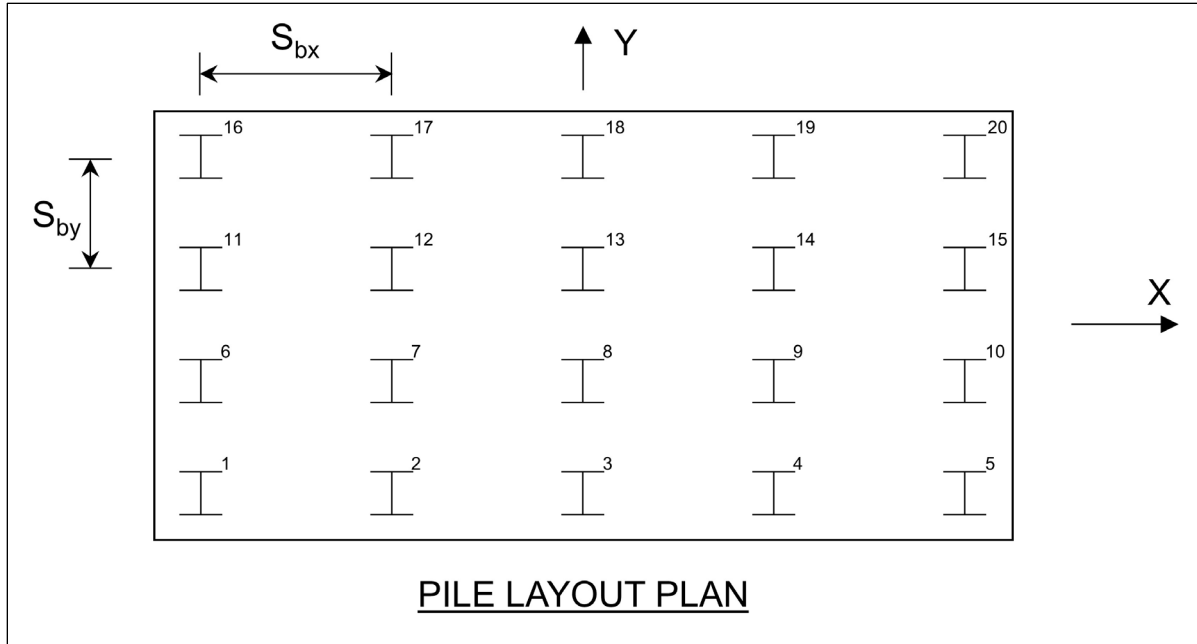


Figure D-68 Group Configuration 6 pile cap plan view.

For the trial pile group configuration in Figure D-68, loads for each pile in the group were determined using the limit state loads presented in Table D-58. The lateral load per pile in each direction was calculated by dividing the factored horizontal load V_{ux} or V_{uy} in Table D-58 by the number of piles, while the axial load per pile was calculated using Equation 8-79. For example, in Group Configuration 6, the lateral load per pile in the longitudinal direction is calculated by dividing 109 kips by 20 piles (i.e., 5.5 kips per pile). For the axial load per pile, an example calculation for the load applied to pile 5 at the Strength V limit state is presented below. Table D-60 shows the full calculation for all piles Group Configuration 6. The maximum individual pile load (on pile number 5) is 257 kips.

$$P_{ui} = \frac{P_{uz} + W_c + W_s}{n} + \frac{M_{ux}y}{\sum y^2} + \frac{M_{uy}x}{\sum x^2} \quad [\text{Eq. 8-79}]$$

In this case, the factored limit state load, Q , replaces the factored axial load from superstructure/substructure acting upon pile cap, P_{uz} , estimated weight of pile cap, W_c and estimated weight of soil above pile cap, W_s as follows:

$$Q = P_{uz} + W_c + W_s$$

Calculate the individual pile axial load on pile number 5.

- x = distance along x-axis from the center of the column to each pile center, 10 feet.
- y = distance along y-axis from the center of the column to each pile center -6 feet.
- $\sum x^2$ = sum of square distance along x-axis from the center of the column to each pile center, 1000 ft².
- $\sum y^2$ = sum of square distance along y-axis from the center of the column to each pile center, 400 ft².
- n = total number of piles, 20 piles (Group Configuration 6, Table D-59).
- Q = factored axial load at Strength V limit state, 3456 kips (Table D-58).
- M_{ux} = factored moment about the x axis acting on the pile cap, -2,981 kip-ft.
- M_{uy} = factored moment about the y axis acting on the pile cap, 3,982 kip-ft.

$$P_{ui} = \frac{Q}{n} + \frac{M_{ux}y}{\sum y^2} + \frac{M_{uy}x}{\sum x^2} \quad [\text{Eq. 8-79}]$$

$$P_{ui} = \frac{(3,456 \text{ kips})}{(20 \text{ piles})} + \frac{(-2,981 \text{ kip-ft}) * (-6 \text{ feet})}{(400 \text{ ft}^2)} + \frac{(3,982 \text{ kip-ft}) * (10 \text{ feet})}{(1000 \text{ ft}^2)}$$

$$P_{ui} = 257 \text{ kips}$$

Table D-60 Calculation of Individual Pile Load

Pile Number	x (feet)	y (feet)	x ² (ft ²)	y ² (ft ²)	$\frac{Q}{n}$ (kips)	$\frac{M_{ux}y}{\sum y^2}$ (kips)	$\frac{M_{uy}x}{\sum x^2}$ (kips)	P _{ui} (kips)
1	-10.0	-6.0	100.0	36.0	172.8	44.7	-39.8	177.7
2	-5.0	-6.0	25.0	36.0	172.8	44.7	-19.9	197.6
3	0.0	-6.0	0.0	36.0	172.8	44.7	0.0	217.5
4	5.0	-6.0	25.0	36.0	172.8	44.7	19.9	237.4
5	10.0	-6.0	100.0	36.0	172.8	44.7	39.8	257.3
6	-10.0	-2.0	100.0	4.0	172.8	14.9	-39.8	147.9
7	-5.0	-2.0	25.0	4.0	172.8	14.9	-19.9	167.8
8	0.0	-2.0	0.0	4.0	172.8	14.9	0.0	187.7
9	5.0	-2.0	25.0	4.0	172.8	14.9	19.9	207.6
10	10.0	-2.0	100.0	4.0	172.8	14.9	39.8	227.5
11	-10.0	2.0	100.0	4.0	172.8	-14.9	-39.8	118.1
12	-5.0	2.0	25.0	4.0	172.8	-14.9	-19.9	138.0
13	0.0	2.0	0.0	4.0	172.8	-14.9	0.0	157.9
14	5.0	2.0	25.0	4.0	172.8	-14.9	19.9	177.8
15	10.0	2.0	100.0	4.0	172.8	-14.9	39.8	197.7
16	-10.0	6.0	100.0	36.0	172.8	-44.7	-39.8	88.3
17	-5.0	6.0	25.0	36.0	172.8	-44.7	-19.9	108.2
18	0.0	6.0	0.0	36.0	172.8	-44.7	0.0	128.1
19	5.0	6.0	25.0	36.0	172.8	-44.7	19.9	148.0
20	10.0	6.0	100.0	36.0	172.8	-44.7	39.8	167.9
Sum	-	-	1000.0	400.0	-	-	-	3456.0

The process outlined above was used to determine the individual pile lateral and axial load for the remaining limit state loads presented in Table D-58. The maximum individual pile axial load is carried forward in the analysis for each group configuration. Based upon the applied loads and group configurations selected, no pile is put into axial tension. Therefore, only axial compression and lateral loads are required for the foundation design at Pier 2. Table D-61 presents the maximum factored axial compression load per pile based upon the number of piles in each group configuration. Table D-62 presents the maximum factored lateral load per pile based upon the number of piles in each group configuration.

Table D-61 Factored Axial Compression Load Per Pile for Alternative Pile Group Configurations

Group Configuration	Strength V, Q (kips)	Service I, without LL Q (kips)	Extreme Event II Q (kips)
4	418	145	256
5	299	109	188
6	257	109	176

Table D-62 Factored Lateral Load Per Pile for Alternative Pile Group Configurations

Group Configuration	Strength V, V _{uy} (kips)	Strength V, V _{ux} (kips)	Extreme Event II V _{uy} (kips)	Extreme Event II V _{ux} (kips)
4	8	2	2	2
5	6	1	2	2
6	6	1	2	2

D.24 Block 12: Pier 2 – Estimate Pile Penetration Depth for Maximum Axial Compression Loads. Check Group Efficiency in Axial Compression

The estimated minimum pile penetration depth necessary to obtain the factored geotechnical resistance equal to the maximum factored load per pile should now be determined. Note that the factored geotechnical resistance in axial compression and the resulting pile penetration depth is dependent upon the resistance determination method. Therefore, the influence of the field resistance determination method on the design is evaluated at this point in the design process. Thus, some resistance determination methods may be eliminated from further design consideration.

At Pier 2, drivability and potential damage when driving through the extremely dense gravel deposit is a concern. In addition, piles will also likely be driven into the gravel layer with cobbles encountered 40 feet below footing grade. This layer also presents a concern for pile damage. Therefore, it is determined that the factored geotechnical resistance will be substantiated by dynamic testing 2% of the piles. Figure D-45 illustrates the factored geotechnical resistance versus penetration depth for the 5 candidate pile sections based on this resistance determination method. From this plot, the estimated penetration depth for a factored geotechnical resistance of 257 kips in axial compression for the Group Configuration 6 ranges from 40 feet for the HP 14x117 to 57 feet for the HP 10x42. Results of this comparison for each pile section and group configuration are listed in Table D-63.

Table D-63 Estimated Pile Penetration Depths for the Factored Geotechnical Resistance in Axial Compression at the Strength V Limit State

Group Configuration	Factored Load per Pile (kips)	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
4	418	60	60	60	50	47
5	299	60	46	44	40	40
6	257	57	42	40	40	40

The Extreme Event II factored axial compression loads shown in Table D-61 were also used to estimate the pile penetration depth required to achieve the factored geotechnical resistance in axial compression. In this case, the nominal geotechnical resistance present in the check flood was used. Figure D-52 illustrates the factored geotechnical resistance versus penetration depth at the check flood for the 5 candidate pile sections based on determination testing by dynamic testing 2% of the piles. From this plot, the estimated penetration depth for the maximum factored geotechnical resistance of 176 kips in axial compression associated with Group Configuration 6 is 40 feet for all candidate pile sections except for the HP 10x42 section which requires approximately 57 feet. The 40 foot penetration depth is upper contact of the dense gravel layer. Results of this comparison for each pile section and group configuration are summarized in Table D-64.

Table D-64 Estimated Pile Penetration Depths for the Factored Geotechnical Resistance in Axial Compression at the Extreme Event II Limit State

Group Configuration	Factored Load per Pile (kips)	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
4	256	57	42	41	40	40
5	188	41	40	40	40	40
6	176	40	40	40	40	40

Based on the factored loads and estimated factored geotechnical resistances at Pier 2, the estimated pile penetration depth considering Strength V loads require the deepest pile penetration depth. Therefore, the estimated pile penetration depths presented in Table D-63 will be carried forward in the design.

Although a mixed soil profile appears at Pier 2, the stratum providing the majority of nominal geotechnical resistance is cohesionless soil, and will therefore be used as the controlling stratum for group efficiency. Per Section 7.2.2.1, the nominal geotechnical resistance of a pile group in cohesionless soil can be taken as the sum of the individual pile nominal geotechnical resistances. In a similar manner, the factored geotechnical resistance of the pile group in cohesionless soil is taken as the sum of the individual pile factored geotechnical resistances. This is recommended so long as 1) the bearing layer is not underlain by weak soil layers, 2) the piles are not installed at a pile spacing of less than 3 times the pile diameter, and 3) no special installation procedures are anticipated such as jetting or predrilling. Since all these conditions are met at Pier 2, the factored group resistances are satisfactory.

D.25 Block 13: Pier 2 – Establish Minimum Pile Penetration Depth for Axial Tension Loads. If Conditions Warrant, Modify Design and Return to Block 10

For the limit state loads identified in Table D-58 in combination with the group configurations at Pier 2 listed in Table D-59, no pile within any group configuration is loaded in tension. Therefore, the tension or uplift resistance is not evaluated, and there is no minimum pile penetration depth required for axial tension loads at Pier 2.

D.26 Block 14: Pier 2 – Establish Minimum Pile Penetration Depth for Lateral Loads. Determine p-y Models, Required Geomaterial Parameters, and Perform Lateral Load Analysis. If Conditions Warrant, Modify Design and Return to Block 10

Next, lateral analyses are performed to establish the required minimum pile penetration depth for lateral loading and to evaluate pile deflection and structural resistance for the applied limit state loads. No minimum penetration depth was required to satisfy the nominal geotechnical resistance requirements in axial tension in Block 13. A minimum pile penetration depth may be required for lateral loading based on the combination of factored lateral loads and structural resistances, or deflection limits. Excessive deflections and moments develop at relatively short pile lengths, where a depth to fixity is not achieved. Furthermore, the structural resistance of pile sections must be evaluated based upon the axial, lateral and moment loads. Factored structural resistances were presented in Table D-47 while a lateral deformation limit of 1 inch was established as a global performance requirement in Block 1 and confirmed in Block 4 as the design progressed.

As discussed in Section 7.3.7.6, p-multipliers are applied to the p-y curves to model pile group behavior. The p-multipliers depend on the center to center pile spacing within the pile group. For Group Configurations 6 at the Pier 2, the pile spacing in the longitudinal direction is 4 feet. Therefore, per Section 7.3.7.6 and AASHTO (2014) design specifications, interpolation was used to determine p-multipliers for a pile spacing of 4b. In this case, the front row p-multiplier is 0.90, the second row is 0.625, and the third and fourth rows are 0.5. For the same group configuration, piles in the transverse direction are spaced at 5 feet. Therefore p-multipliers for a pile spacing of 5b were used. In this case, the front row p-multiplier is 1.0, the second row is 0.85 and the third and fourth rows are 0.7.

Soil properties utilized for the lateral analysis are given in Figure D-39. Cyclic loading was performed for all rows using LPILE's Load Type 2 option, which uses shear and slope to model a fixed head condition. Considering loading conditions at this pier, lateral analyses in the longitudinal (y-direction) were performed about the pile section's strong axis. Figure D-68 shows the pile orientation within the trial pile cap design.

The following example is presented for the HP 12x74 pile section using a range of factored axial and lateral loads for Group Configuration 6. The Strength V limit state loads are applied in combination with the geotechnical resistance at the design flood, (i.e., 5 feet of scour below the pile head) Tables D-65 to D-67 provide LPILE output summaries for the longitudinal direction considering a pile penetration (from the bottom of pile cap) of 20 feet. Due to scour conditions, this is modeled as 15 feet of embedded pile length with 5 feet of pile stickup.

Table D-65 LPILE Summary Output at Pile Head for Front Row, $p_m=0.90$

Load Case	Load Type No.	Pile-Head Condition 1 V (kips)	Pile-Head Condition 2 S (rad)	Axial Load (kips)	Pile-Head Deflection (inches)	Maximum Moment in Pile (kip-ft)	Maximum Shear in Pile (kips)	Pile-Head Rotation (radians)
1	2	0	0	257	0.000	0.0	0.0	0
2	2	1	0	257	0.011	-5.4	-1.3	0
3	2	2	0	257	0.023	-10.8	-2.4	0
4	2	3	0	257	0.035	-16.3	-3.3	0
5	2	4	0	257	0.048	-21.8	-4.1	0
6	2	5	0	257	0.060	-27.3	5.0	0
7	2	6	0	257	0.073	-32.8	6.0	0
8	2	7	0	257	0.086	-38.3	7.0	0
9	2	8	0	257	0.099	-43.8	8.0	0
10	2	9	0	257	0.112	-49.3	9.0	0

Table D-66 LPILE Summary Output at Pile Head for Second Row, $p_m=0.625$

Load Case	Load Type No.	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load (kips)	Pile-Head Deflection (inches)	Maximum Moment in Pile (kip-ft)	Maximum Shear in Pile (kips)	Pile-Head Rotation (radians)
		V (kips)	S (rad)					
1	2	0	0	257	0.000	0.0	0.0	0
2	2	1	0	257	0.012	-5.6	-1.2	0
3	2	2	0	257	0.026	-11.2	-2.2	0
4	2	3	0	257	0.040	-16.9	3.0	0
5	2	4	0	257	0.054	-22.7	4.0	0
6	2	5	0	257	0.069	-28.4	5.0	0
7	2	6	0	257	0.083	-34.2	6.0	0
8	2	7	0	257	0.098	-40.0	7.0	0
9	2	8	0	257	0.113	-45.7	8.0	0
10	2	9	0	257	0.128	-51.5	9.0	0

Table D-67 LPILE Summary Output at Pile Head for Third and Fourth Row, $p_m=0.50$

Load Case	Load Type No.	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load (kips)	Pile-Head Deflection (inches)	Maximum Moment in Pile (kip-ft)	Maximum Shear in Pile (kips)	Pile-Head Rotation (radians)
		V (kips)	S (rad)					
1	2	0	0	257	0.000	0.0	0.0	0
2	2	1	0	257	0.013	-5.7	-1.2	0
3	2	2	0	257	0.028	-11.5	-2.1	0
4	2	3	0	257	0.043	-17.4	3.0	0
5	2	4	0	257	0.059	-23.3	4.0	0
6	2	5	0	257	0.075	-29.2	5.0	0
7	2	6	0	257	0.091	-35.1	6.0	0
8	2	7	0	257	0.107	-41.1	7.0	0
9	2	8	0	257	0.124	-47.0	8.0	0
10	2	9	0	257	0.140	-52.9	9.0	0

From LPILE's deflection results for individual rows, pile group deflection can be estimated. Factored load versus deflection for each row is plotted along with the average load for a given deflection as shown in Figure D-69. A discussion of this procedure is outlined in Section 7.3.7.6.

The rigid cap method assumes piles move together, and therefore experience the same shear and lateral load. Accordingly, at the resulting factored lateral load per pile, V_{ly} , of 6 kips (Table D-62), the estimated lateral group deflection at the pile head is determined as 0.085 inches. This lateral deflection is less than the 1 inch tolerance based upon project specific requirements.

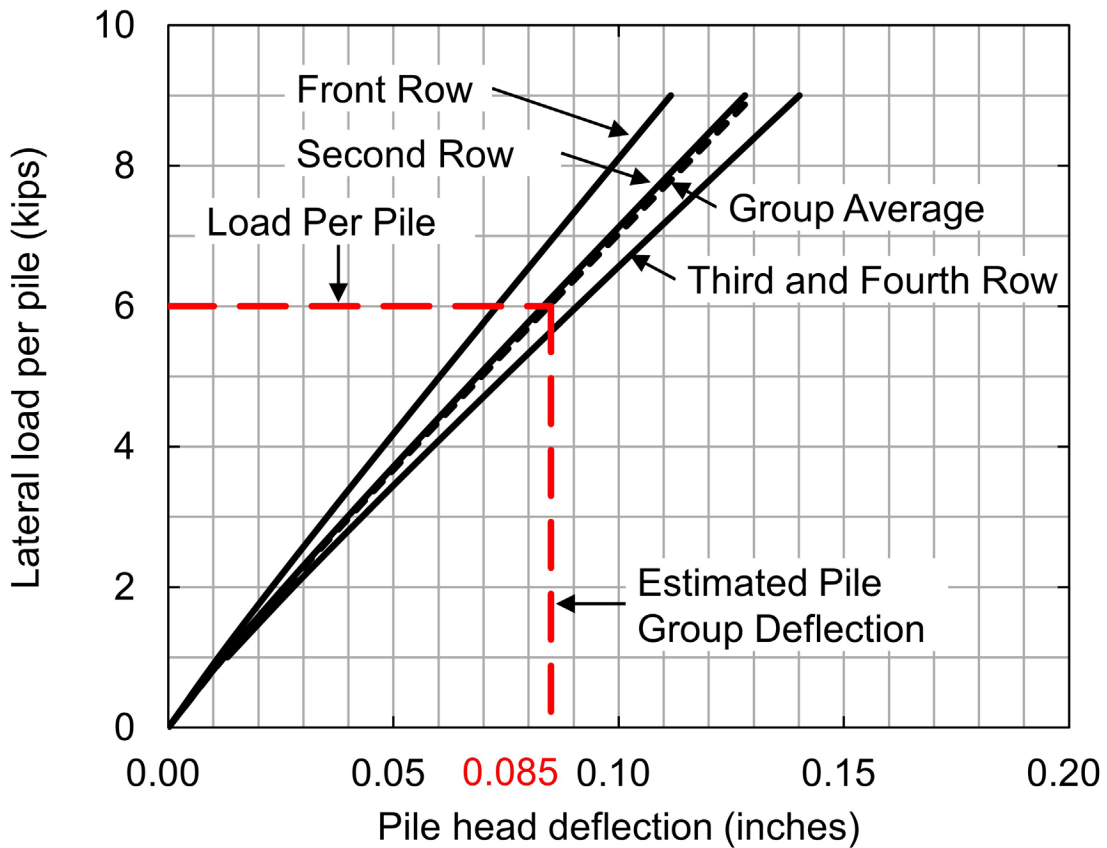


Figure D-69 Factored load versus deflection in longitudinal direction for HP 12x74 at Pier 2.

In addition to the lateral deflection limit, stresses from the resulting bending moment and shear must be evaluated to check that the pile section does not fail structurally. Using the LPILE tabular results, Figure D-70 plots the front row bending moment versus depth for a deflection of 0.085 inches.

Figure D-71 plots the maximum bending moment versus pile head deflection for all rows, however only the maximum bending moment for the front row is used as a “worst case” inspection of the structural resistance in combined axial compression and flexure. As illustrated in Figure D-71, at the estimated pile head deflection of 0.085 inches, the maximum bending moment, M_{uy} , is 38 kip-ft.

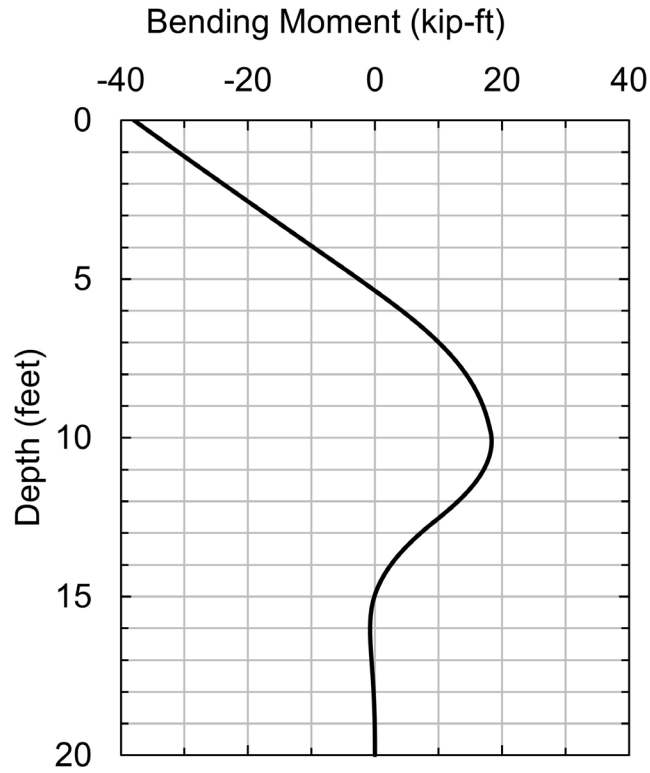


Figure D-70 Front row bending moment versus depth in longitudinal direction.

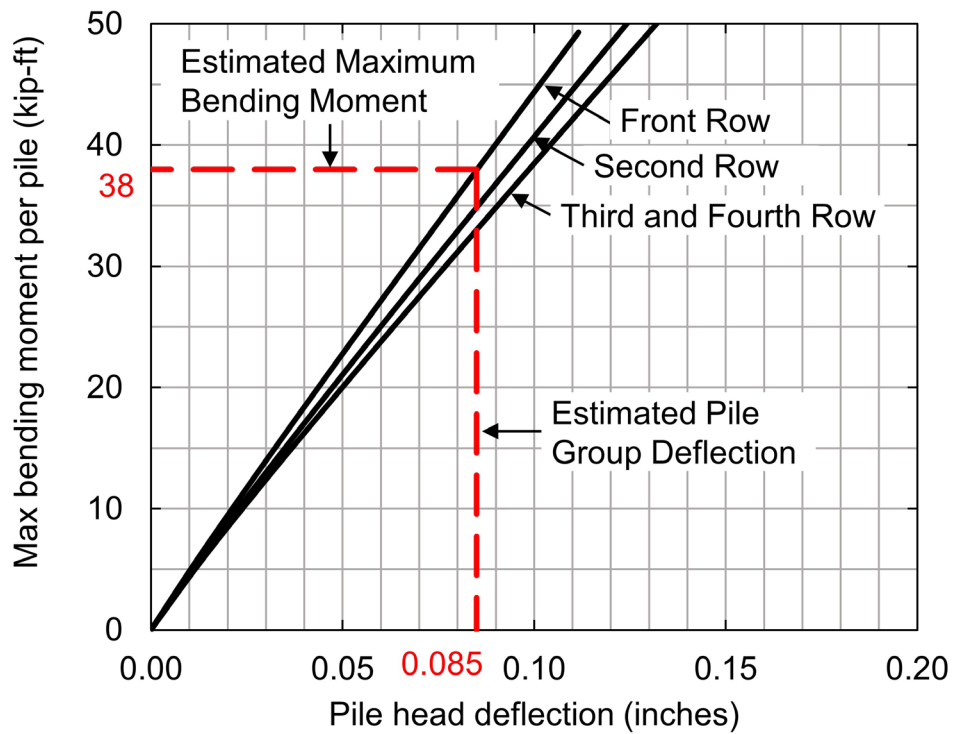


Figure D-71 Bending moment versus deflection in longitudinal direction for HP 12x74 at Pier 2.

The lateral analysis was also performed for the transverse direction with the Strength V limit state loads and the geotechnical resistance at the design flood, (i.e., 5 feet of scour below the pile head). Tables D-68 to D-70 provide LPILE output summaries for the transverse direction considering a pile penetration (from the bottom of footing) of 20 feet. Due to scour in the design flood, this is modeled as 15 feet of embedded pile length with 5 feet of pile stickup.

Table D-68 LPILE Summary Output at Pile Head for Front Row, $p_m=1.0$

Load Case	Load Type No.	Pile-Head Condition 1 V (kips)	Pile-Head Condition 2 S (rad)	Axial Load (kips)	Pile-Head Deflection (inches)	Maximum Moment in Pile (kip-ft)	Maximum Shear in Pile (kips)	Pile-Head Rotation (radians)
1	2	0.00	0	257	0.000	0.0	0.0	0
2	2	0.25	0	257	0.003	-1.3	-0.3	0
3	2	0.50	0	257	0.005	-2.6	-0.7	0
4	2	0.75	0	257	0.008	-4.0	-1.0	0
5	2	1.00	0	257	0.011	-5.3	-1.3	0
6	2	1.25	0	257	0.014	-6.7	-1.6	0
7	2	1.50	0	257	0.016	-8.0	-1.9	0
8	2	1.75	0	257	0.019	-9.3	-2.2	0
9	2	1.00	0	257	0.011	-5.3	-1.3	0

Table D-69 LPILE Summary Output at Pile Head for Second Row, $p_m=0.85$

Load Case	Load Type No.	Pile-Head Condition 1 V (kips)	Pile-Head Condition 2 S (rad)	Axial Load (kips)	Pile-Head Deflection (inches)	Maximum Moment in Pile (kip-ft)	Maximum Shear in Pile (kips)	Pile-Head Rotation (radians)
1	2	0.00	0	0	0.000	0.0	0.0	0
2	2	0.25	0	0	0.003	-1.3	-0.3	0
3	2	0.50	0	0	0.006	-2.7	-0.7	0
4	2	0.75	0	0	0.008	-4.0	-1.0	0
5	2	1.00	0	0	0.011	-5.4	-1.3	0
6	2	1.25	0	0	0.014	-6.8	-1.6	0
7	2	1.50	0	0	0.017	-8.1	-1.8	0
8	2	1.75	0	0	0.020	-9.5	-2.1	0
9	2	1.00	0	0	0.011	-5.4	-1.3	0

Table D-70 LPILE Summary Output at Pile Head for Third and Fourth and Fifth Rows, $p_m=0.70$

Load Case	Load Type No.	Pile-Head Condition 1 V (kips)	Pile-Head Condition 2 S (rad)	Axial Load (kips)	Pile-Head Deflection (inches)	Maximum Moment in Pile (kip-ft)	Maximum Shear in Pile (kips)	Pile-Head Rotation (radians)
1	2	0.00	0	0	0.000	0.0	0.0	0
2	2	0.25	0	0	0.003	-1.4	-0.3	0
3	2	0.50	0	0	0.006	-2.7	-0.7	0
4	2	0.75	0	0	0.009	-4.1	-1.0	0
5	2	1.00	0	0	0.012	-5.5	-1.3	0
6	2	1.25	0	0	0.015	-6.9	-1.5	0
7	2	1.50	0	0	0.018	-8.3	-1.8	0
8	2	1.75	0	0	0.022	-9.7	-2.0	0
9	2	1.00	0	0	0.012	-5.5	-1.3	0

From LPILE's deflection results for individual rows, pile group deflection can be estimated. Factored load versus deflection for each row is plotted along with the average load for a given deflection as shown in Figure D-72. A discussion of this procedure is outlined in Section 7.3.7.6.

The rigid cap method once again assumes piles move together, and therefore experience the same shear and lateral load. Accordingly, at the resulting factored lateral load per pile, V_{uy} , of 1 kip (Table D-62), the estimated lateral group deflection at the pile head is determined as 0.011 inches. This lateral deflection is less than the 1 inch tolerance based upon project specific requirements.

In addition to the lateral deflection limit, stresses from the resulting bending moment and shear must be evaluated to check that the pile section does not fail structurally. Using the LPILE tabular results, Figure D-73 plots the front row bending moment versus depth for deflection of 0.011 inches.

Figure D-74 plots the maximum bending moment versus pile head deflection for all rows, however only the maximum bending moment for the front row is used as a "worst case" inspection of the structural resistance in combined axial compression and flexure. As illustrated is Figure D-74, at the estimated pile head deflection of 0.011 inches, the maximum bending moment, M_{uy} , is 5.6 kip-ft.

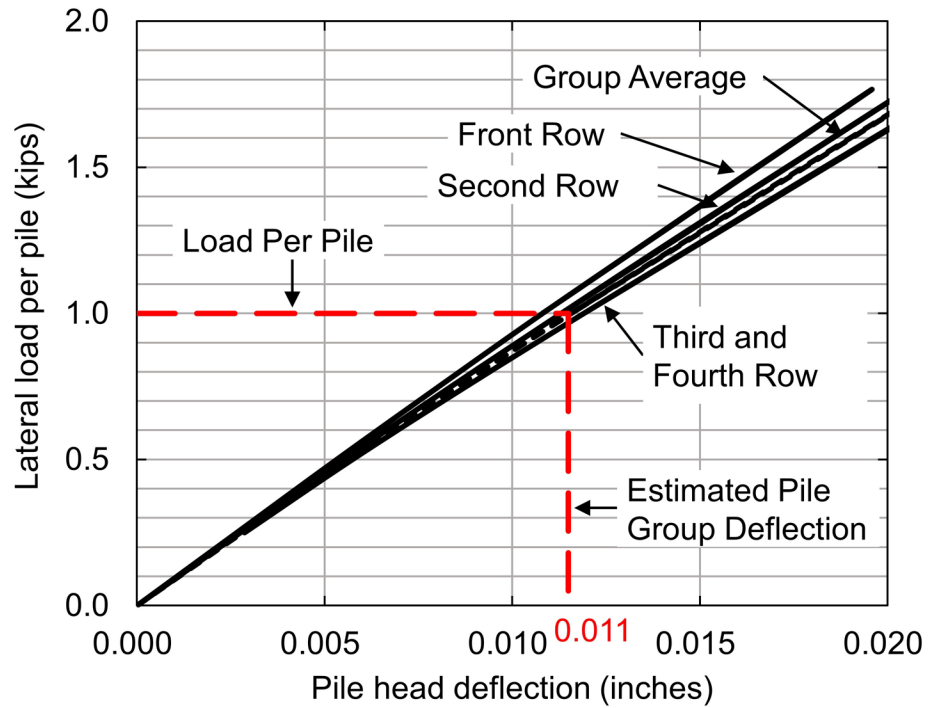


Figure D-72 Factored load versus deflection in transverse direction for HP 12x74 at Pier 2.

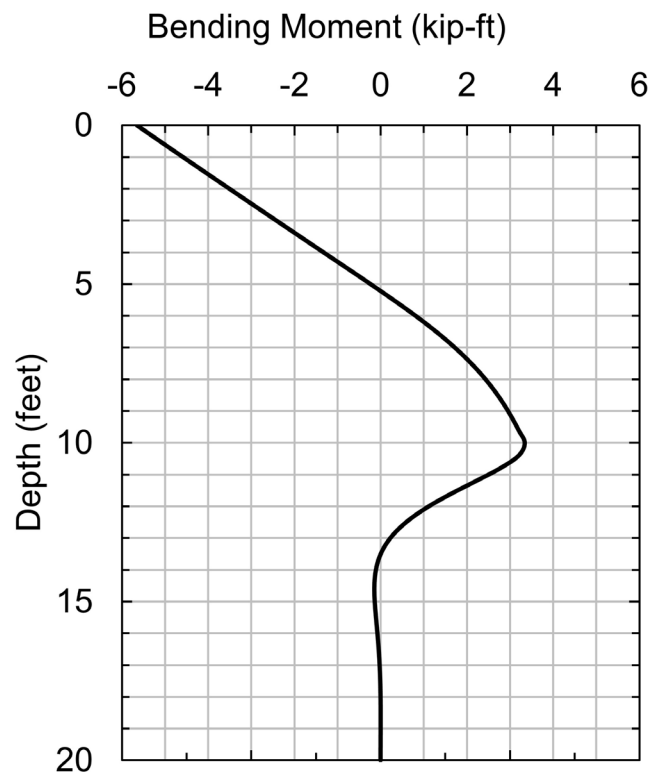


Figure D-73 Front row bending moment versus depth in transverse direction.

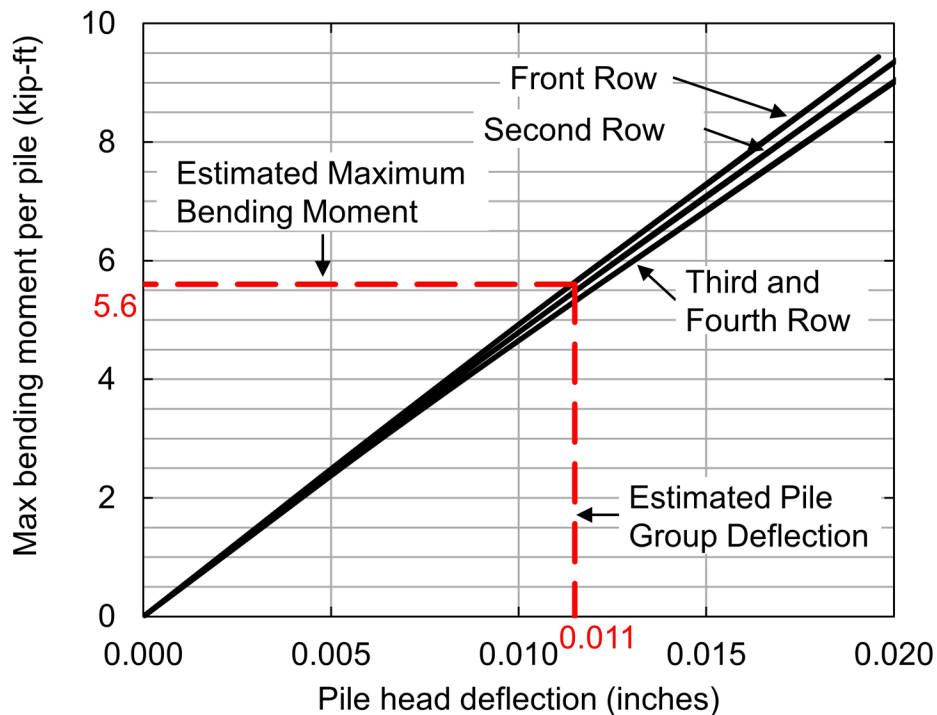


Figure D-74 Bending moment versus deflection in transverse direction for HP 12x74 at Pier 2.

The interaction shown in Equation 8-58 must be satisfied for the factored axial compression load and moment in the pile. Using results of the lateral analysis, the factored structural resistance was evaluated at the pile head using the factored axial compression load and maximum bending moment (determined using factored loads). The factored structural resistances were determined as shown in Table D-47.

Equation 8-58 must be satisfied for the pile section to be acceptable.

- P_u = factored axial load, 257 kips (Table D-61).
- P_r = factored axial resistance, 730 kips (Table D-47).
- M_{ux} = factored moment about x-axis, 5.6 kip-ft (Figure D-74).
- M_{rx} = factored flexural resistance about x-axis, 118 kip-ft (Table D-47).
- M_{uy} = factored moment about y-axis, 38 kip-ft (Figure D-71).
- M_{ry} = factored flexural resistance about y-axis, 433 kip-ft (Table D-47).

$$\frac{P_u}{P_r} + \frac{8.0}{9.0} \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \right) \leq 1.0 \quad [\text{Eq. 8-58}]$$

$$\frac{(257 \text{ kips})}{(730 \text{ kips})} + \frac{8.0}{9.0} \left(\frac{(5.6 \text{ kip-ft})}{(118 \text{ kip-ft})} + \frac{(38 \text{ kip-ft})}{(433 \text{ kip-ft})} \right) \leq 1.0$$

$$0.47 \leq 1.0$$

The maximum shear from factored lateral loading was then compared to the factored shear resistance from Table D-47. The maximum shear within the pile occurred at the pile head. Based on the factored loads, the factored shear resistance is acceptable.

V_r = factored shear resistance, 214 kips (HP 12x74, Table D-47).

V_{uy} = factored shear load, 6 kips (Group Configuration 6, Table D-62).

$$V_{uy} < V_r$$

$$6 \text{ kips} < 214 \text{ kips}$$

The lateral analysis was also performed for each alternative pile section, and subsequently the deflection and factored structural resistance was evaluated considering the factored loads for group configurations shown in Table D-62. Pile head deflection must be limited to 1 inch, and based upon the applied loads and pile section, the factored structural resistance of the pile must also satisfy the structural resistance interaction equation presented as Equation 8-58. Furthermore, the pile should be embedded such that fixity is established, and for this case, the second crossing of the moment versus depth curve with the y-axis (i.e., moment is 0 kip-ft) is assumed as fixity for this design example. A review of Figure D-70 shows this depth to be approximately 20 feet. Pile sections satisfying these criteria were deemed acceptable, and furthermore as presented in Table D-71, a minimum required pile penetration depth was established based on lateral loads at the Strength V limit state. The minimum penetration depth is identified as “- - -” for candidate pile sections not meeting the lateral deformation or structural resistance requirements.

Table D-71 Minimum Pile Penetration Depth Required for Strength V Lateral Loads at Pier 2

Group Configuration	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
4	- - -	- - -	30	25	25
5	25	25	25	25	25
6	20	20	20	20	20

In a similar manner, the lateral analysis was repeated using the Extreme Event II limit state loads and the geotechnical resistance at the check flood. Although axial loads and lateral loads in the longitudinal direction decreased, further loss of geotechnical resistance from scour required an additional analysis. Table D-72 presents the minimum pile penetration depth required to satisfy lateral loading at the Extreme Event II limit state. As a result of reduced axial and lateral loads, the HP 10x42 and HP 12x52 pile sections in Group Configuration 4 would be acceptable. For the remaining candidate pile sections and group configurations, no additional pile penetration was required beyond the previously identified pile penetration depths in Table D-71.

Table D-72 Minimum Pile Penetration Depth Required for Extreme Event II Lateral Loads at Pier 2

Group Configuration	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
4	30	30	30	25	25
5	25	25	25	25	25
6	20	20	20	20	20

After evaluating lateral loads with the respective geotechnical resistance at the Strength and Extreme Event limit states, the established minimum pile penetration depth is taken as the worst case for each candidate section and group configuration. In this case, the more conservative depth for all candidate sections and group configurations is taken from the lateral loading evaluation at the Strength limit state. The established minimum pile penetration depth required for lateral loading is presented in Table D-73.

Table D-73 Established Minimum Pile Penetration Depth Required for Lateral Loading at Pier 2

Group Configuration	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
4	---	---	30	25	25
5	25	25	25	25	25
6	20	20	20	20	20

D.27 Block 15: Pier 2 – Establish Pile Penetration Depths that Satisfy Tolerable Deformations. Estimate Group Settlement over the Minimum and Maximum Range of Pile Penetration Depths From Blocks 12 to 14 and Identify All Pile Toe Elevations Which Result in Intolerable Deformations. If Conditions Warrant, Modify Design and Return to Block 10

Pile group settlement at Pier 2 was estimated using two methods, the Hough (1959) method and the Janbu Tangent modulus method. Ideally, the settlement method chosen by the designer is one that has shown good correlation with observed results. In order to obtain the factored geotechnical resistance in axial compression, the piles must be driven into the dense gravel with sand layer (Soil Layer 4, Figure D-39). As this is a cohesionless soil, pile group settlement was first estimated using the Hough (1959) Method. The Hough approach is a traditional settlement estimation method for pile groups in cohesionless soils and one that is referenced in the AASHTO (2014) design specifications. However, the soil conditions across the bridge substructure locations are quite variable and a settlement method that could be used at all substructure locations was also desired. Therefore, group settlement was also computed with the Janbu Tangent modulus approach using an equivalent footing placed at the neutral plane.

Compressibility properties for the silty clay layer were determined from one dimensional consolidation tests which were performed on undisturbed samples collected near the middle of the clay layer. Table D-74 presents the void ratio, e_o , overconsolidation ratio, OCR, compression index, C_c , and recompression index, C_r for the silty clay layer. The OCR of the soil was used to calculate the preconsolidation stress, σ'_p , at discrete depths.

Table D-74 Soil Properties Determined from One Dimensional Consolidation Test

Soil Layer	e_o	OCR	C_c	C_r
3	0.80	2.01	0.30	0.03

The Hough (1959) method as presented in Section 7.3.5.2 was used to estimate group settlement. As mentioned, piles must be driven to the dense gravel layer (Soil Layer 4) to achieve the required factored geotechnical resistance in axial compression. The settlement calculations were performed using only unfactored permanent loads. Therefore, the loads from the Service I limit state without live load were used to estimate settlement (i.e., load factor of 1.0 on permanent loads). The average contact stress for the trial pile group was calculated using the vertical load, Q , and pile group area $B \times Z$. The following calculation is performed for Group

Configuration 6. Both the group width, B , and length, Z , were calculated from exterior pile edge to exterior pile edge.

An equivalent footing, with plan dimensions equal to those of the pile group, was evaluated at increasing pile penetration depths, and the resulting pile group settlement computed using the Hough (1959) method. The equivalent footing concept for piles driven to hard clay or sand through soft clay as presented in Figure 7-46a of Chapter 7 was utilized this analysis.

The following calculation is performed for the equivalent footing located at the silty clay / dense gravel interface at Elevation 235.0 feet. This location is 40 feet below the bottom of pile cap at Elevation 275.0 feet. Figure D-75 illustrates the pile group and resulting stress distribution at the equivalent footing. It was also assumed that vertical deformation below the gravel and bedrock interface at Elevation 215.0 feet is negligible. For Group Configuration 6, the length of the pile group is 21 feet, while the width is 13 feet. These dimensions are taken from exterior pile edge to exterior pile edge. This example calculation is therefore only suitable for this configuration. The soil profile was divided into 5 foot thick layers to evaluate settlement. Tabulated values for this group settlement analysis are presented in Table D-75. The elevation shown in Table D-75 references the midpoint of each respective soil layer, while z is the depth below the equivalent footing to the midpoint of each respective soil layer.

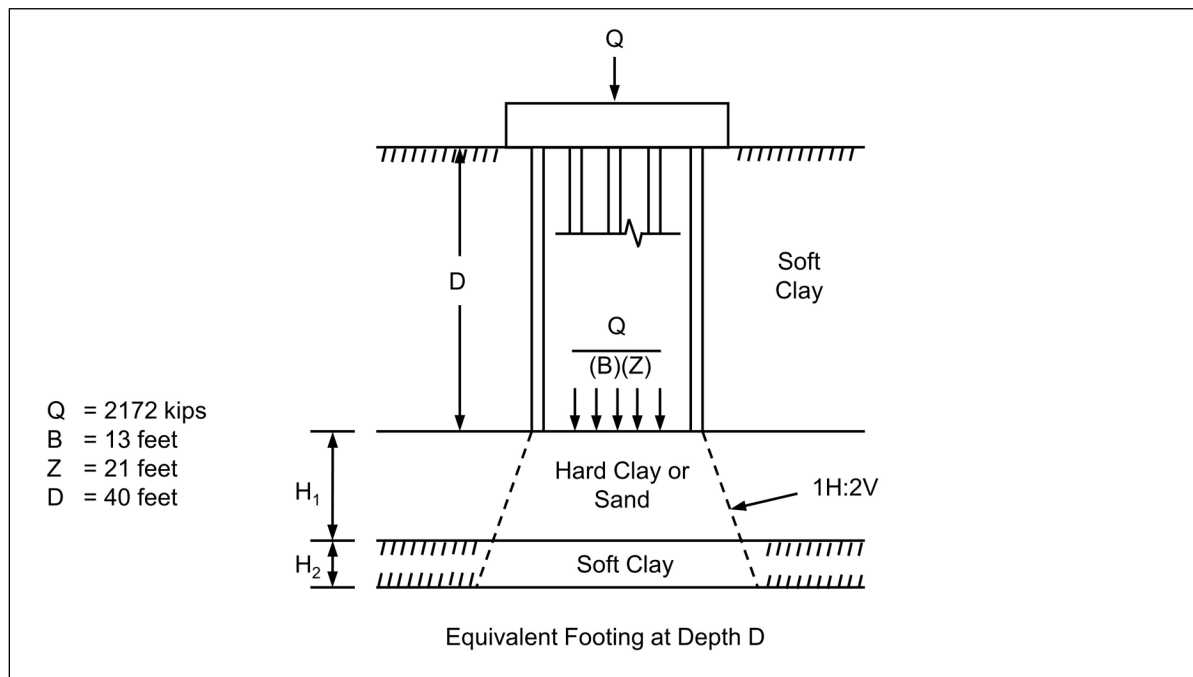


Figure D-75 Equivalent footing and stress distribution toe bearing piles in hard clay or sand considering Group Configuration 6 dimensions at Pier 2.

For the Hough (1959) method, the bearing capacity index must first be determined using the corrected SPT (N_1) value, however a review of the design soil profile in Figure D-39 shows SPT (N_1)₆₀ value as 46 for the dense gravel layer. To note, this value is corrected for both depth and energy. Figure D-76 presents the modified Hough (1959) method chart to determine the bearing capacity index from the respective soil layer's (N_1) value. Because the graph will be entered with a corrected SPT (N_1)₆₀ value, the x-axis scale consistent with the Safety hammer was used to enter the graph, while the soil type selected to determine the bearing capacity index was conservatively taken as the “clean, well graded fine to coarse sand.” Based on these two assumptions, the bearing capacity index, C' , was determined as 180.

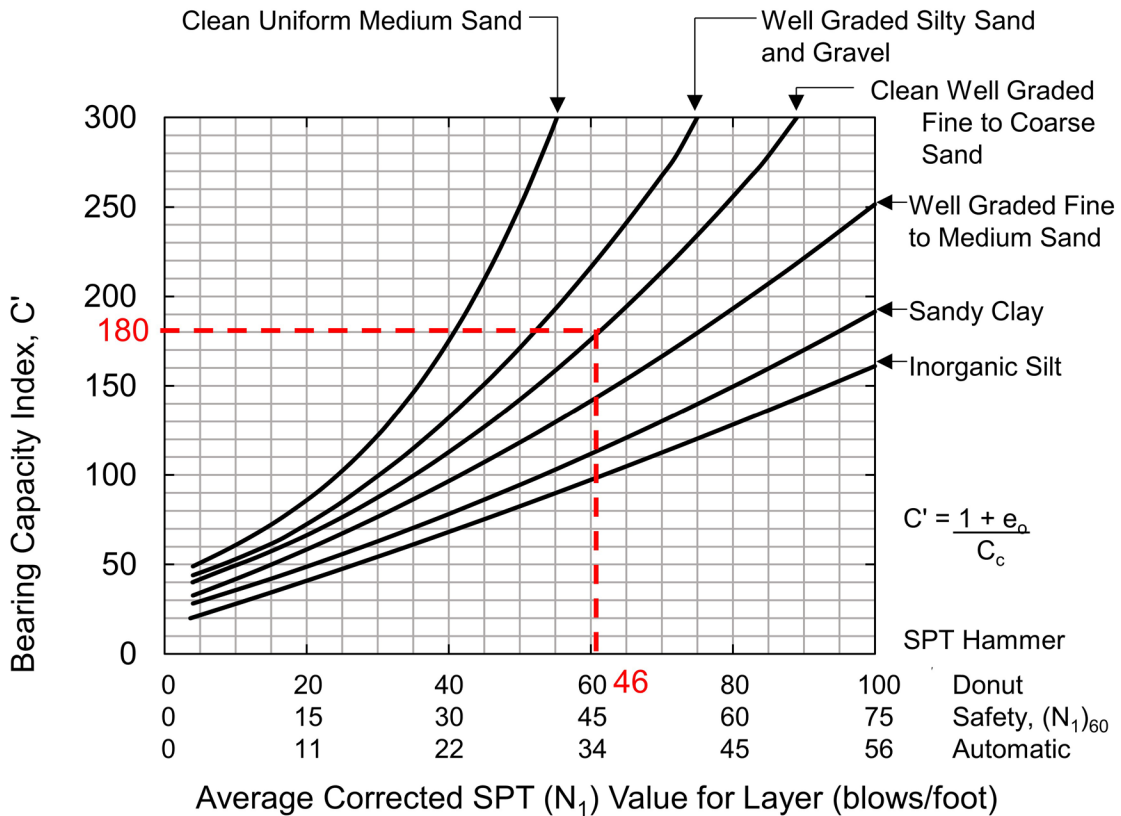


Figure D-76 Hough (1959) method chart to determine bearing capacity index from SPT (N_1) at Pier 2.

Considering only the unfactored permanent load, Q, calculate the vertical effective stress increase below the equivalent footing, $\Delta\sigma'_v$, at EL 235.0 feet.

- Q = unfactored permanent load, 2182 kips (Service I, without LL, Table D-58).
- B = pile group width, 13 feet.
- Z = pile group length, 21 feet (Group Configuration 6 only)
- z = depth below equivalent footing, 2.5 feet (Elev. 232.5 feet)

$$\Delta\sigma'_v = \frac{Q}{(B+z)+(Z+z)} \quad [\text{Eq. 7-55}]$$

$$\Delta\sigma'_v = \frac{(2182 \text{ kips})}{((13 \text{ feet})+(2.5 \text{ feet}))*((21 \text{ feet})+(2.5 \text{ feet}))}$$

$$\Delta\sigma'_v = 5.96 \text{ ksf}$$

Determine the percent stress increase by comparing σ'_{v0} and σ'_1

σ'_{v0} = initial vertical effective stress at depth z below the equivalent footing, 2.92 ksf.

σ'_1 = new vertical effective stress considering the unfactored permanent load, 2.92 ksf + 5.96ksf = 8.88 ksf.

$$\text{Percent Stress Increase} = \frac{\sigma'_1 - \sigma'_{v0}}{\sigma'_{v0}} * 100\% \quad [\text{Eq. D-5}]$$

$$\text{Percent Stress Increase} = \frac{(8.88 \text{ ksf}) - (2.92 \text{ ksf})}{(2.92 \text{ ksf})} * 100$$

$$\text{Percent Stress Increase} = 204\%$$

The stress increase is greater than or equal to 10%. Deformation for this depth increment should be estimated and included in the sum of all depth increments in which the stress increase is not less than 10%. A rock layer, which is considered incompressible, is located at Elev. 215.0 feet at this substructure location, and thus it is assumed that no settlement or compression occurs below this depth.

For the dense gravel at Elev. 232.5 feet, $z = 2.5$ determine deformation in the layer from the increase in vertical effective stress, S , with Equation 7-49.

H_o = initial soil layer thickness, 5 feet.

C' = dimensionless bearing capacity index, 180 (Figure D-76).

σ'_{vo} = vertical effective stress at midpoint of layer prior to stress increase, 2.92 ksf.

$\Delta\sigma'_v$ = vertical effective stress increase in the layer, 5.96 ksf.

$$S = H_o \left[\frac{1}{C'} \log \frac{\sigma'_{vo} + \Delta\sigma'_{vo}}{\sigma'_{vo}} \right] \quad [\text{Eq. 7-49}]$$

$$S = (5 \text{ feet}) \left[\frac{1}{(180)} \log \frac{(2.92 \text{ ksf}) + (5.96 \text{ ksf})}{(2.92 \text{ ksf})} \right] * \left(\frac{12 \text{ inches}}{1 \text{ foot}} \right)$$

$$S = 0.16 \text{ inches}$$

Table D-75 Settlement Estimate for Hough Method With Equivalent Footing Located at Elev. 235.0 feet

EL (feet)	z (feet)	H_o (ksf)	σ'_{vo} (ksf)	B (feet)	Z (feet)	$\Delta\sigma'_{v(ss)}$ (ksf)	σ'_1 (ksf)	Stress Incr. (%)	S (in)
232.5	2.5	5	2.92	15.5	23.5	5.96	8.88	204	0.16
227.5	7.5	5	3.23	20.5	28.5	3.72	6.95	115	0.11
222.5	12.5	5	3.54	25.5	33.5	2.54	6.08	72	0.08
217.5	17.5	5	3.85	30.5	38.5	1.85	5.70	48	0.06
								Total	0.41

The above analysis was performed for additional pile penetration depths considering pile group dimensions of all three group configurations. Table D-76 summarizes these analysis results for the Hough (1959) estimated settlement. For an equivalent footing at Elev. 240.0 feet, pile group settlement is magnified as 5 feet of silty clay remains below the equivalent footing, and thus consolidation settlement occurs. Conventional settlement equations as discussed in Section 7.3.5.6 were used to estimate settlement in this 5 feet thick layer in combination with the Hough (1959) method equations which were applied for the dense gravel. Elevation 235.0 feet is the upper contact of the dense gravel layer, and thus the comparatively smaller deformation below this depth is from elastic compression. A review of Table D-76 shows that for all group configurations, the vertical deformations reduce as the pile toe penetrates into the dense gravel layer at Elev. 235.0 feet. Therefore, the minimum pile penetration to design against intolerable deformations is 40 feet.

Table D-76 Summary of Pile Group Settlement Estimation Using Hough (1959)
Method For All Pile Group Configurations at Pier 2

Equivalent Footing Elevation (feet)	Pile Toe Elevation (feet)	Pile Penetration Depth (feet)	Estimated Settlement For Group Configuration 4 (inches)	Estimated Settlement For Group Configuration 5 (inches)	Estimated Settlement For Group Configuration 6 (inches)
240.0	240.0	35.0	4.77	3.96	2.33
235.0	235.0	40.0	0.63	0.56	0.41
230.0	230.0	45.0	0.52	0.46	0.33

Pile group settlement was also calculated using the neutral plane method and Janbu tangent modulus as discussed in Section 7.3.5.6. An equivalent footing, with plan dimensions equal to those of the pile group, was evaluated at increasing pile penetration depths, and the resulting pile group settlement computed using the Janbu tangent modulus. This procedure allowed the shallowest depth of an equivalent footing to be determined that met vertical deformation requirements. The required minimum pile penetration depth was then determined based on the pile toe elevation that would place the neutral plane at this same equivalent footing depth where vertical deformation requirements were satisfied.

The following example calculation is performed for the neutral plane located 40 feet below the bottom of footing. Figure D-77 presents the pile group and resulting stress distribution at the equivalent footing. From Elev. 270.0 feet to 235.0 feet, strain calculations for cohesive soil (stress exponent of $j=0$) were applied for Soil Layer 3. Below Elev. 235.0 feet however, strain calculations for dense coarse grained soil (stress exponent of $j=1$) were applied for the Soil Layer 4. It was assumed that vertical deformation below the encountered bedrock at Elev. 215.0 feet is negligible. For Group Configuration 6, the length of the pile group is 21 feet, while the width is 13 feet. This example calculation is therefore only suitable for this configuration. Dimensions are taken from exterior pile edge to exterior pile edge. The soil profile was again divided into 5 foot thick layers. Tabulated values for this analysis are recorded in Table D-77. The elevation shown in Table D-77 references the midpoint of each respective soil layer, while z is the depth below the equivalent footing to the midpoint of each respective soil layer.

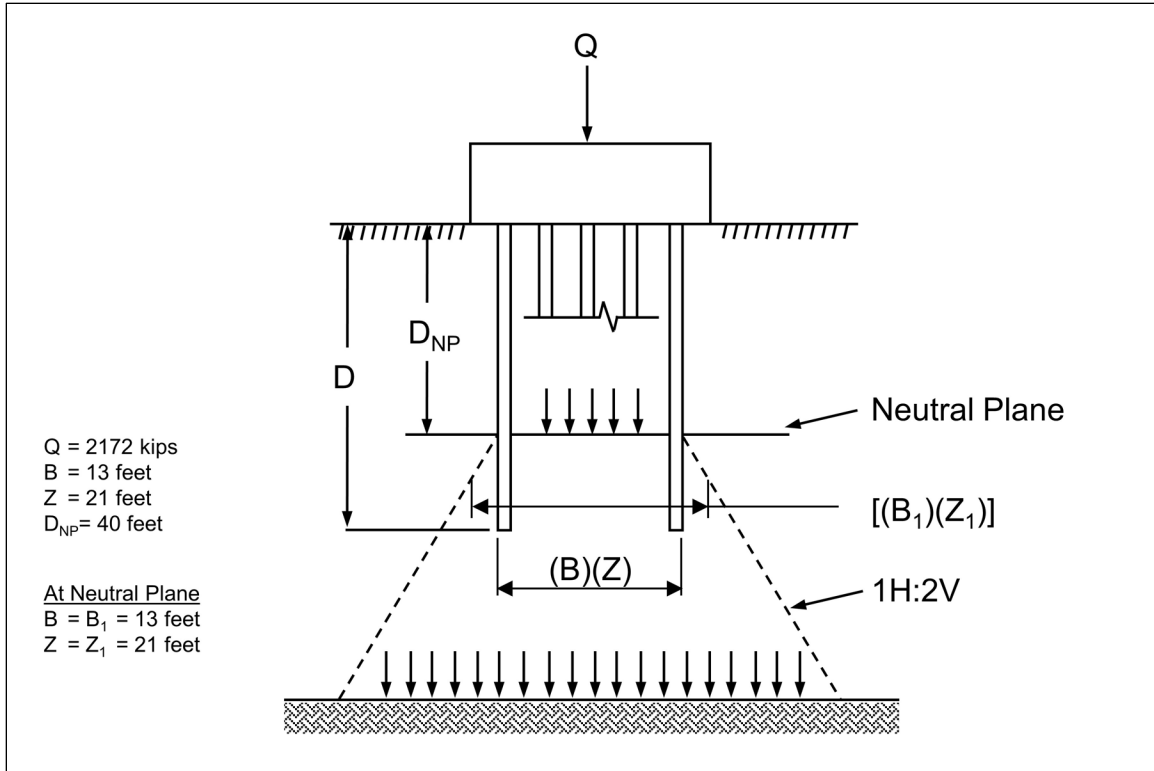


Figure D-77 Neutral plane at 40 feet below the pile cap and resulting stress distribution for Group Configuration 6.

Considering only the unfactored permanent load, Q , calculate the vertical effective stress increase below the equivalent footing, $\Delta\sigma'_{v}$, at EL 232.5 feet.

- Q = unfactored permanent load, 2182 kips (Service I, without LL, Table D-58).
- B = pile group width, 13 feet.
- Z = pile group length, 21 feet (Group Configuration 6 only).
- z = depth below equivalent footing, 2.5 feet (Elev. 232.5 feet).

$$\Delta\sigma'_{v(ss)} = \frac{Q}{(B+z)+(Z+z)} \quad [\text{Eq. 7-55}]$$

$$\Delta\sigma'_{v(ss)} = \frac{(2182 \text{ kips})}{((13 \text{ feet})+(2.5 \text{ feet}))*((21 \text{ feet})+(2.5 \text{ feet}))}$$

$$\Delta\sigma'_{v(ss)} = 5.96 \text{ ksf}$$

Determine the percent stress increase by comparing σ'_{vo} and σ'_1 .

σ'_{vo} = initial vertical effective stress at depth z below the equivalent footing, 2.92 ksf.

σ'_1 = new vertical effective stress considering the unfactored permanent load, 2.92 ksf + 5.96ksf = 8.88 ksf.

$$\text{Percent Stress Increase} = \frac{\sigma'_1 - \sigma'_{vo}}{\sigma'_{vo}} * 100\% \quad [\text{Eq. D-5}]$$

$$\text{Percent Stress Increase} = \frac{(8.88 \text{ ksf}) - (2.92 \text{ ksf})}{(2.92 \text{ ksf})} * 100$$

$$\text{Percent Stress Increase} = 205\%$$

The stress increase is greater than or equal to 10%. Deformation for this depth increment should be estimated and included in the sum of all depth increments in which the stress increase is not less than 10%. A rock layer, which is considered incompressible, is located at Elev. 215 feet at this pier, and thus it is assumed that no settlement or compression occurs below this depth.

For the dense coarse grained soil at Elev. 232.5 feet, $z = 2.5$ (stress exponent of $j = 1.0$), determine the strain in the layer from the increase in vertical effective stress, ε , with Equation 7-61.

E_s = elastic modulus of soil, 1104 ksf (Figure D-39).

σ'_{vo} = initial vertical effective stress at depth z below the equivalent footing, 2.92 ksf.

σ'_1 = new vertical effective stress below the equivalent footing, 8.88 ksf.

$$\varepsilon = \frac{1}{E_s} [\sigma'_1 - \sigma'_{vo}] \quad [\text{Eq. 7-61}]$$

$$\varepsilon = \frac{1}{(1104 \text{ ksf})} [(8.88 \text{ ksf}) - (2.92 \text{ ksf})]$$

$$\varepsilon = 0.0054$$

Calculate the layer compression denoted, S , with the initial height of the layer, H_o .

$$S = \varepsilon * H_o$$

$$S = 0.0054 * 5 \text{ feet} * \left(\frac{12 \text{ inches}}{1 \text{ foot}} \right)$$

$$S = 0.32 \text{ inches}$$

Table D-77 Settlement Estimation for Neutral Plane Method with the Neutral Plane at EL 235.0 feet

EL (feet)	z (feet)	H _o (ksf)	σ' _{vo} (ksf)	B (feet)	Z (feet)	Δσ' _{v(ss)} (ksf)	σ' ₁ (ksf)	Stress Incr. (%)	ε	S (in)
232.5	2.5	5	2.92	15.5	23.5	5.96	8.88	205	0.0054	0.32
227.5	7.5	5	3.23	20.5	28.5	3.72	6.95	115	0.0034	0.20
222.5	12.5	5	3.54	25.5	33.5	2.54	6.08	72	0.0023	0.14
217.5	17.5	5	3.85	30.5	38.5	1.85	5.70	48	0.0017	0.10
									Total	0.76

Table D-78 summarizes the settlement analysis results estimated using the neutral plane method and Janbu Tangent modulus approach. Although not yet determined, it is preliminarily estimated that elastic pile compression may be on the order of 0.3 inches, effectively limiting tolerable soil settlement to on the order of 1 inch.

Therefore, to limit the total vertical deformation from elastic pile compression and settlement to less than 1.5 inches, the results in Table D-78 indicate that the neutral plane should be located 40 feet below the bottom of footing for Group Configuration 6. In a similar manner, the neutral plane should be located 55 feet and 50 feet below the bottom of footing for Group Configuration 4 and 5, respectively.

Table D-78 Estimated Pile Group Settlement Using Janbu Tangent Modulus with Neutral Plane Method For All Pile Group Configurations

Neutral Plane Elevation (feet)	Neutral Plane Depth (feet)	Estimated Settlement GC 4 (inches)	Estimated Settlement GC 5 (inches)	Estimated Settlement GC 6 (inches)
240.0	35	5.22	4.32	2.44
235.0	40	1.59	1.28	0.76
230.0	45	1.44	1.14	0.66
225.0	50	1.20	0.94	0.53
220.0	55	0.80	0.61	0.32

It is assumed that the equivalent footing acts at the same location as the neutral plane. Accordingly, an analysis was performed to determine the pile toe elevation necessary to locate the neutral plane at Elev. 235.0 (for piles in Group Configuration 6), thereby establishing the minimum required pile penetration depth to satisfy tolerable deformations.

The location of the neutral plane and the magnitude of drag force are evaluated following the procedure outlined in Section 7.3.6, using unfactored permanent loads and nominal geotechnical resistance. Because load factors for the Service limit state are 1.0, applicable loads at this limit state may be considered unfactored. The Service I, without LL limit state loads are therefore used for evaluation for the neutral plane. This example again utilizes the load for Group Configuration 6 (Q= 109 kips, Table D-61). Figure D-78 presents a graphical interpretation of the neutral plane for the HP 12x74 pile driven to 56 feet (pile toe at Elev. 219.0).

The unfactored permanent load and the cumulative shaft resistance are first plotted, followed by the mobilized toe resistance minus the cumulative shaft resistance (all vs. pile penetration depth). The exact percentage of toe mobilization is unknown at this stage of design, and therefore at least two of the toe mobilization curves should be evaluated to determine the neutral plane location. The 0% toe mobilization curve is the most conservative location to evaluate settlement (since it locates the neutral plane at the highest elevation), and the 100% toe mobilization curve should be used to check the pile section's structural strength (since it results in the greatest axial force in the pile). The structural strength check is performed in Block 17.

At Pier 2, it is expected that piles will be supported partially by toe resistance when driven to the dense gravel layer. Therefore the 0% toe mobilization curve presents

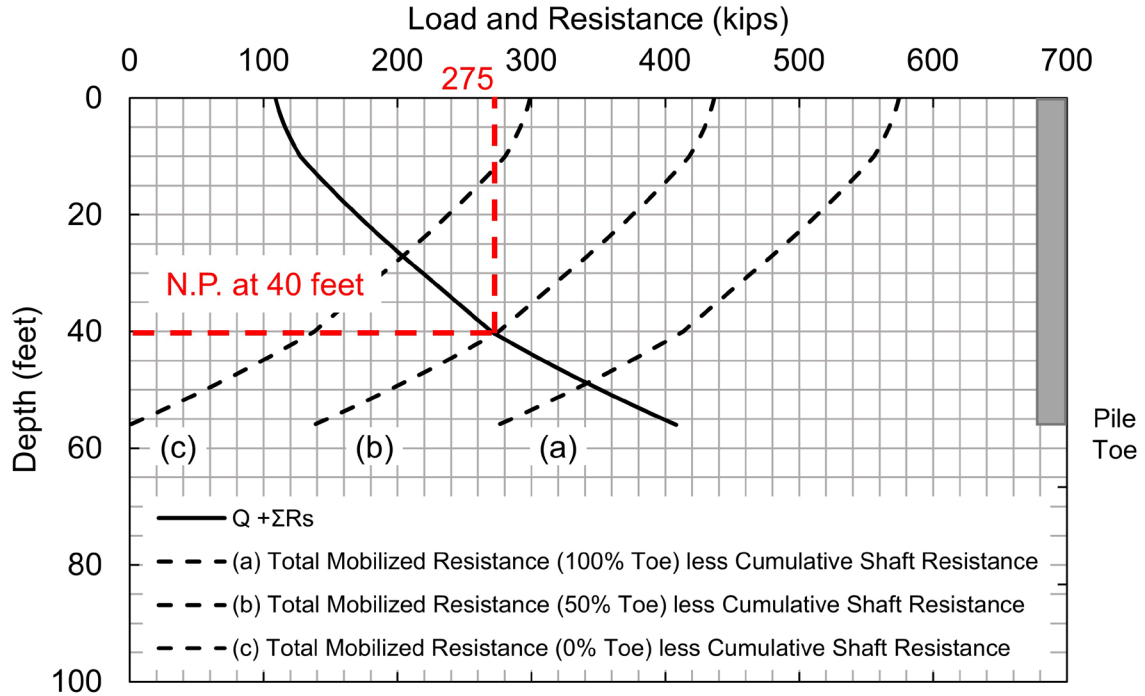


Figure D-78 Neutral plane location considering 50% toe mobilization for HP 12x74 at Pier 2.

an unreasonable baseline to evaluate settlement in this case. Some toe resistance is likely mobilized, and to remain consistent with the other substructure locations, the 50 % toe mobilization curve is again used to evaluate settlement.

Assuming 50 percent toe mobilization, the neutral plane is located 40 feet below the bottom footing with a resulting maximum load in the pile of 275 kips. For the remaining candidate pile sections and trial group configurations, the above analysis was also performed to determine the pile penetration depth required to locate the neutral plane at the depths indicated in Table D-78 for each respective pile group configuration. Table D-79 summarizes the results from these analyses.

Table D-79 Summary of Pile Penetration Depth Required to Locate Neutral Plane at Depth Determined From Settlement Estimation

Group Configuration	Load per Pile Q (kips)	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
4	145	60	60	60	60	60
5	109	60	60	60	58	60
6	109	60	58	56	52	49

To limit unacceptable settlement, it was decided that the method yielding the greatest settlement estimate would be used to establish the range of pile penetration depths to preclude unacceptable settlement. The neutral plane method illustrated above was therefore performed for additional equivalent footing depths until settlements within the project deformation criteria were estimated. Experience and judgement should be used to assess the required pile penetration depth with any settlement estimation approach.

A review of the soil profile at Pier 2 in Figure D-39 indicates no compressible soil layers below Soil Layer 4. Therefore, no effort is necessary to identify a maximum pile penetration depth to prevent punching through a dense layer into an unsuitable soil layer, or to apply a large stress increase on a lower compressible layer causing excessive settlement. Table D-80 presents the established minimum pile penetration depths to satisfy tolerable deformations at Pier 2.

Table D-80 Established Minimum Pile Penetration Depths to Satisfy Tolerable Deformations at Pier 2

Group Configuration	Load per Pile Q (kips)	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
4	145	60	60	60	60	60
5	109	60	60	60	58	60
6	109	60	58	56	52	49

Elastic shortening of the pile should be considered in the total pile deformation, noting that the drag force from negative shaft resistance adds to the axial compression force in the pile. Negative shaft resistance is determined from the previously discussed neutral plane calculation. For elastic compression, the load per pile from the Service I, without live load limit state (which is, in effect, and unfactored load) is applied at the pile head. As shown in Table D-61, this load is 109 kips.

Note that the drag force from negative shaft resistance increases the axial compression force in the pile. Negative shaft resistance above the neutral plane acts to increase axial compression force in the pile, whereas below the neutral plane, positive shaft resistance reduces the axial compression force in the pile. This effect must be accounted for in the elastic compression calculation. Accordingly, the unfactored axial load used to compute elastic compression, Q, changes for each pile

segment length, increasing equal to the unfactored permanent load plus the shaft resistance down to the neutral plane. In this example, the unfactored axial load is equal to the resistance distribution from 100% toe mobilization. The highest drag force magnitude results from this curve and represents the worst case.

Equation 7-48 is used to illustrate this example for the first 12 inch increment of the HP 12x74 pile section. The average shaft resistance and average load for each respective depth interval is used to estimate the elastic compression. For each 12 inch segment, the elastic modulus remains constant, and was evaluated as 29,000 ksi. The pile cross sectional area likewise remains constant as 21.8 in². Remaining calculations were performed using a spreadsheet; Table D-81 summarizes the elastic compression with depth.

Determine the unfactored axial load, Q , in segment.

$$Q_d = \text{unfactored permanent load, } 109.0 \text{ kips.}$$

$$R_s^- = \text{average (negative) shaft resistance, } 1.1 \text{ kips.}$$

$$Q = Q_d + R_s^- = 109.0 \text{ kips} + 1.1 \text{ kips}$$

$$Q = 110.1 \text{ kips}$$

Calculate elastic compression of segment with unfactored axial load from combined unfactored permanent load and negative shaft resistance.

$$L = \text{segment length, } 12 \text{ inches.}$$

$$A = \text{cross sectional area of pile material, } 21.8 \text{ in}^2.$$

$$E = \text{elastic modulus of pile, } 29,000 \text{ ksi.}$$

$$\Delta = \frac{QL}{AE} \quad [\text{Eq. 7-48}]$$

$$\Delta = \frac{(110.1 \text{ kips}) * (12 \text{ inches})}{(21.8 \text{ in}^2) * (29,000 \text{ ksi})}$$

$$\Delta = 0.00209 \text{ inches}$$

Table D-81 Elastic Compression Calculation

Depth Below Pile Head (feet)	Average Shaft Resistance (kips)	Average Unfactored Axial Load (kips)	Δ (inches)
0	0.0	109.0	0.00000
0-1	0.6	110.1	0.00209
1-2	1.8	110.8	0.00210
2-3	3.1	112.1	0.00213
3-4	4.7	113.7	0.00216
4-5	6.4	115.4	0.00219
5-6	8.2	117.2	0.00222
6-7	10.2	119.2	0.00226
7-8	12.4	121.4	0.00230
8-9	14.8	123.8	0.00235
9-10	17.3	126.3	0.00240
10-11	20.5	129.5	0.00246
11-12	24.4	133.4	0.00253
12-13	28.4	137.4	0.00261
13-14	32.5	141.5	0.00269
14-15	36.8	145.8	0.00277
15-16	41.1	150.1	0.00285
16-17	45.3	154.3	0.00293
17-18	49.6	158.6	0.00301
18-19	54.1	163.1	0.00310
19-20	58.7	167.7	0.00318
20-21	63.3	172.3	0.00327
21-22	67.8	176.8	0.00336
22-23	72.5	181.5	0.00344
23-24	77.3	186.3	0.00354
24-25	82.2	191.2	0.00363
25-26	87.1	196.1	0.00372
26-27	92.0	201.0	0.00381
27-28	97.0	206.0	0.00391
28-29	102.1	211.1	0.00401
29-30	107.4	216.4	0.00411
30-31	112.6	221.6	0.00421
31-32	117.7	226.7	0.00430
32-33	122.8	231.8	0.00440
33-34	127.9	236.9	0.00450
34-35	133.0	242.0	0.00459
35-36	138.1	247.1	0.00469
36-37	143.2	252.2	0.00479
37-38	148.3	257.3	0.00488

Table D-81 Elastic Compression Calculation (continued)

Depth Below Pile Head (feet)	Average Shaft Resistance (kips)	Average Unfactored Axial Load (kips)	Δ (inches)
38-39	153.4	262.4	0.00498
39-40	158.5	267.5	0.00508
40-41	164.8	273.8	0.00520
41-42	172.3	281.3	0.00534
42-43	180.1	289.1	0.00549
43-44	187.9	296.9	0.00564
44-45	196.0	305.0	0.00579
45-46	204.2	313.2	0.00594
46-47	212.5	321.5	0.00610
47-48	221.0	330.0	0.00626
48-49	228.4	337.4	0.00640
49-50	227.1	336.1	0.00638
50-51	218.2	327.2	0.00621
51-52	209.1	318.1	0.00604
52-53	199.9	308.9	0.00586
53-54	190.5	299.5	0.00568
54-55	180.9	289.9	0.00550
55-56	171.2	280.2	0.00532
		Total	0.23

For the pile head load of 109 kips (Group Configuration 1 loads), estimated elastic compression of the HP 12x74 pile section driven to 56 feet is 0.23 inches. Combined with 0.76 inches of deformation from settlement (Table D-78), it is estimated that total vertical deformation at Pier 2 is 0.99 inches.

D.28 Block 16: Pier 2 – Check pile drivability to maximum pile penetration depth requirements established in Blocks 12 through 15

Preliminary pile drivability was previously evaluated for the 5 candidate pile sections in Block 10. The plots of nominal resistance, blow count and compression stress versus depth in Figure D-67 should now be reviewed considering the established minimum pile penetration depths. A candidate pile section must be capable of being driven to the penetration depth necessary to achieve the nominal geotechnical resistance in axial compression and tension, and to a penetration depth necessary to satisfy lateral load demands as well as axial and lateral deformation requirements. However, no minimum pile penetration depth is required to satisfy axial tension

resistance. Minimum pile penetration depths have been previously established in Blocks 14 and 15 and are presented in Tables D-73 and D-80.

Although a minimum pile penetration depth is not established for nominal geotechnical resistance in axial compression, the pile should also be capable of being driven close to this estimated pile penetration depth. If the pile cannot be driven to the required depth within driving stress limits and at reasonable blow counts, a larger pile hammer, a pile section with greater impedance, or pile installation aids such as predrilling or jetting may be required to satisfy or improve drivability. Alternatively, substructure design modifications should be considered.

For the HP 12x74 candidate pile section in Group Configuration 6 the pile penetration depth for axial compression loading was estimated at 41 feet (Table D-64). There is no minimum penetration depth for axial tension loading since there was no tension demand. The minimum penetration depth for lateral loading was 20 feet (Table D-73). A minimum pile penetration depth of 56 feet is required to satisfy vertical deformation limits (Table D-80). Accordingly, candidate pile section must have sufficient drivability to the maximum of these depths, 56 feet.

A review of Figure D-67 indicates that the HP 12x74 pile section can be driven to bedrock at approximately 60 feet, if required. In addition, from the preliminary drivability evaluation (with soil and hammer model assumptions described in Block 10), it is estimated that the blow count will not exceed 120 blows per foot or 10 blows per inch before this penetration depth. Compression driving stresses are estimated to remain below driving-stress limits. Therefore, it is concluded that pile drivability to the estimated pile penetration depth is achievable.

D.29 Block 17: Pier 2 – Determine the Neutral Plane Location and Resulting Drag Force. Check Structural Strength Limit State for Pile Penetration Depth From Block 16

Previously in Block 15, the neutral plane was evaluated as part of the settlement calculations. The neutral plane and resulting drag force are now evaluated to check the structural strength limit state for candidate pile sections. Section 7.3.6 of Chapter 7 provides a reference for evaluating the neutral plane location and the magnitude of the drag force. Load using the Service I, without live load limit state (which are, in effect, unfactored loads) are considered. This example again utilizes the load for Group Configuration 6 ($Q = 109$ kips, Table D-61).

It is good practice that the drag force be evaluated for 100 percent toe mobilization. This results in the neutral plane at its lowest potential location, and thus the highest drag force magnitude. Accordingly, this is the toe mobilization curve that should be used to check the pile section's structural resistance. Figure D-79 presents a graphical interpretation of the neutral plane location for the HP 12x74 pile section driven to the estimated pile penetration depth of 56 feet. In this case the neutral plane is located 49 feet below pile head with a resulting maximum axial compression force in the pile of 342 kips.

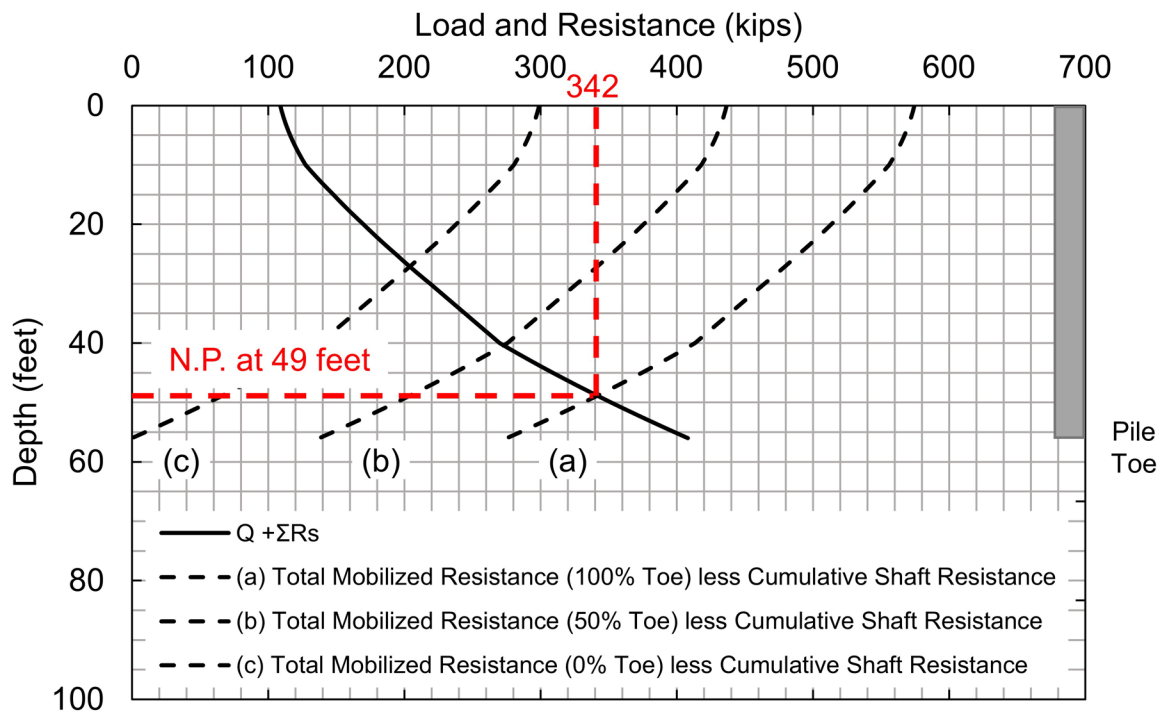


Figure D-79 Neutral plane location considering 100 percent toe mobilization for HP 12x74 at Pier 2.

The resulting unfactored drag force, DF , is the difference between the maximum unfactored axial compression force in the pile, Q_{max} , minus the unfactored permanent load (Q). In this case, the drag force is evaluated for 100 percent toe mobilization.

$$DF = Q_{max} - Q$$

$$DF = (342 \text{ kips}) - (109 \text{ kips}) = 233 \text{ kips}$$

Following this calculation, the structural resistance was checked with Equation 7-70. As discussed in Section 7.3.6 of Chapter 7, a load factor of 1.25 is applied to

the permanent load while a load factor of 1.1 is applied to the drag force. For H-piles which may be subject to damage during driving and require pile toe protection (i.e., as for piles driven to bedrock or through dense gravel, cobbles, etc.), the structural resistance factor in axial compression, ϕ_c , is 0.5. This applied to the nominal structural resistance of 1043 kips (Table D-46). For this assumption, the factored structural resistance, P_r , for the HP 12x74 section is 521 kips.

$$P_u = 1.25 (Q) + \gamma_p (DF) < P_r \quad [\text{Eq. 7-70}]$$

$$P_u = 1.25 (109 \text{ kips}) + 1.1(233 \text{ kips}) = 393 \text{ kips}$$

$$393 \text{ kips} < 521 \text{ kips}$$

In this case, the factored structural resistance is greater than the factored load, and therefore the pile section is acceptable. It may be beneficial to review the ratio of the factored load to the nominal structural resistance or, P_u/P_n . The following evaluation serves to back calculate the minimum required structural resistance factor, $\phi_{c(\text{min})}$, for the section to be acceptable considering the factored load.

$$\phi_{c(\text{min})} = \frac{P_u}{P_n}$$

$$\phi_{c(\text{min})} = \frac{(393 \text{ kips})}{(1043 \text{ kips})}$$

$$\phi_{c(\text{min})} = 0.38$$

The neutral plane and drag force analysis was also performed for each candidate pile section considering 100 percent toe mobilization. Factored structural resistance was subsequently evaluated considering the factored loads for group configurations shown in Table D-61. The pile penetration depth utilized for the structural resistance check was the required minimum penetration depth presented in Table D-80. Table D-82 presents the ratio of the factored load to nominal structural resistance, at the pile toe, for the all the candidate piles and group configurations. For H-piles which may be subject to damage during driving and require pile toe protection (i.e., as for piles driven to bedrock or through dense gravel, cobbles, etc.), the structural resistance factor in axial compression, ϕ_c , is 0.5. A review of Table D-82 shows several unacceptable candidate sections and group configurations. The HP 10x42 section is not acceptable in any group configuration, while the HP12x53 is not acceptable in Group Configuration 4 or 5. In a similar manner, the HP 12x74 section is not acceptable in Group Configuration 4.

Table D-82 Ratio of Factored Load to Nominal Structural Resistance in Axial Compression, $\phi_{c(\min)}$, at the Pile Toe

Group Configuration	HP 10x42 $\phi_{c(\min)}$	HP 12x53 $\phi_{c(\min)}$	HP 12x74 $\phi_{c(\min)}$	HP 14x89 $\phi_{c(\min)}$	HP 14x117 $\phi_{c(\min)}$
4	0.74*	0.68*	0.51*	0.50*	0.39*
5	0.66*	0.62*	0.49*	0.46*	0.37*
6	0.66*	0.38	0.26	0.38	0.27

Note: * - pile toe on rock.

For each combination, the ratio of the factored load to nominal structural resistance was also independently evaluated at the neutral plane. Table D-83 presents the ratio of the factored load to nominal structural resistance at the neutral plane

Table D-83 Ratio of Factored Load to Nominal Structural Resistance in Axial Compression, $\phi_{c(\min)}$, at the Neutral Plane

Group Configuration	HP 10x42 $\phi_{c(\min)}$	HP 12x53 $\phi_{c(\min)}$	HP 12x74 $\phi_{c(\min)}$	HP 14x89 $\phi_{c(\min)}$	HP 14x117 $\phi_{c(\min)}$
4	0.74*	0.68*	0.51*	0.50*	0.39*
5	0.66*	0.62*	0.49*	0.46*	0.35
6	0.66*	0.49	0.38	0.36	0.27

Note: * - neutral plane on rock.

Based on results of the drag force analysis, a candidate pile section may be eliminated from consideration if the factored loads are higher than the factored structural resistance. As noted in Table D-84, the larger candidate pile sections remained acceptable including drag force consideration.

Table D-84 Does Candidate Pile Section Meet Structural Resistance Requirement Considering Drag Force Associated with Minimum Pile Penetration Depth?

Group Configuration	HP 10x42	HP 12x53	HP 12x74	HP 14x89	HP 14x117
1	No	No	No	Yes	Yes
2	No	No	Yes	Yes	Yes
3	No	Yes	Yes	Yes	Yes

D.30 Decision 18: Does Estimated Total Settlement and Differential Settlement Between Adjacent Substructure Locations Satisfy Requirements and Angular Distortion Limits?

The North Abutment and Pier 2 have now been preliminarily designed and the estimated vertical deformations computed for both locations.

The vertical deformation limits and construction point concept detailed in Section 7.3 of Chapter 7 was used to first calculate tolerable differential settlement based upon angular distortion. Using Equation D-1, the angular distortion between substructure supports is limited to 0.004 radians, and for a 100 ft span on a multispan bridge, this equates to 4.8 inches of tolerable differential settlement.

Determine tolerable differential settlement, S_d , between substructure supports.

- L_s = span length, 100 feet.
- A = angular distortion limit, 0.004 radians (Table 7-18).

$$S_d = A * L_s \quad \text{Eq. D-7}$$

$$S_d = (0.004) * (100 \text{ feet}) * \left(\frac{12 \text{ inches}}{1 \text{ foot}}\right)$$

$$S_d = 4.8 \text{ inches}$$

Although this differential settlement is tolerable for angular distortion, project performance requirements limited the settlement at each substructure location to a maximum of 1.5 inches, the differential settlement between adjacent substructure locations limited to 1.0 inch, and a maximum angular distortion of 0.0008 radians. These requirements were established for rideability, drainage, and attached utility damage considerations. The estimated substructure performance is summarized in Table D-85.

Table D-85 Summary of Foundation Total Settlement, Differential Settlement, and Angular Distortion

Substructure Location	Settlement Method	Total Settlement (inches)	Differential Settlement (inches)	Span Length (feet)	Angular Distortion
N. Abutment	N. Plane	1.17			
			0.18	100	0.0002
Pier 2	N. Plane	0.99			

The total settlement, differential settlement, and angular distortion for the two substructures designed so far are acceptable.

D.31 Block 19: Pier 2 – Evaluate Economics of Candidate Piles, Preliminary Group Configurations, and Other Factors

Until now, the design process has served to compare strength and service limits for several candidate pile types within trial group configurations. Some candidate pile types have not met all of the strength, service, or drivability requirements. It is useful to quickly review the suitable and unsuitable pile types and group configurations and then assess the cost of the viable foundation solutions.

Table D-86 summarizes the established minimum pile penetration depth based on analysis results from Blocks 12 through 15. For the candidate pile sections and group configurations at Pier 2, the established minimum pile penetration depth was, in some cases, based on meeting tolerable vertical deformations, and in other cases, based on satisfying the geotechnical resistance demand.

Several candidate sections did not meet structural resistance requirements for axial loading, lateral loading or both. The candidate pile sections and/or group configurations not meeting design requirements are identified with an asterisk in Table D-86. For all three group configurations, the HP 10x42 section did not meet all structural resistance requirements. In a similar manner, the HP 12x53 pile section is unacceptable in Group Configuration 4 or 5 and the HP 12x74 pile section is unacceptable in Group Configuration 4. These candidate pile sections and group configurations will therefore be eliminated for the final design.

Table D-86 Established Minimum Pile Penetration Depth at Pier 2

Group Configuration	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
4	60*	60*	60*	60	60
5	60*	60*	60	60	60
6	60*	58	56	52	49

Note: * - Did not meet structural resistance requirement.

Table D-87 presents the estimated minimum penetration depth for each candidate section to meet the factored geotechnical resistance requirements at the Strength V limit state. The larger pile sections require less pile penetration depth than the smaller pile sections to provide the same geotechnical resistance. In addition, the factored load per pile decreases from Group Configuration 4 to Group Configuration 6. However, from the analyses in Blocks 13 through 15, the established minimum penetration depth to preclude unacceptable vertical deformation requires all piles to be driven deeper than the depth needed for their factored geotechnical resistance. The minimum penetration depth requirement results in the additional geotechnical resistance gained by further pile embedment to be essentially wasted and therefore uneconomical.

Table D-87 Estimated Minimum Penetration Depth for Factored Geotechnical Resistance at Strength V Limit State

Group Configuration	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
4	60*	60*	60*	50	47**
5	60*	46*	44**	40**	40**
6	57*	42**	41**	40**	40**

Note: * - Did not meet structural resistance requirement.

Note: ** - Must be driven deeper to meet deformation requirements (Table D-80).

Reference should be made to Block 19 of the North Abutment for a discussion of individual pile cost. However Figure D-80 again presents the individual pile cost versus depth.

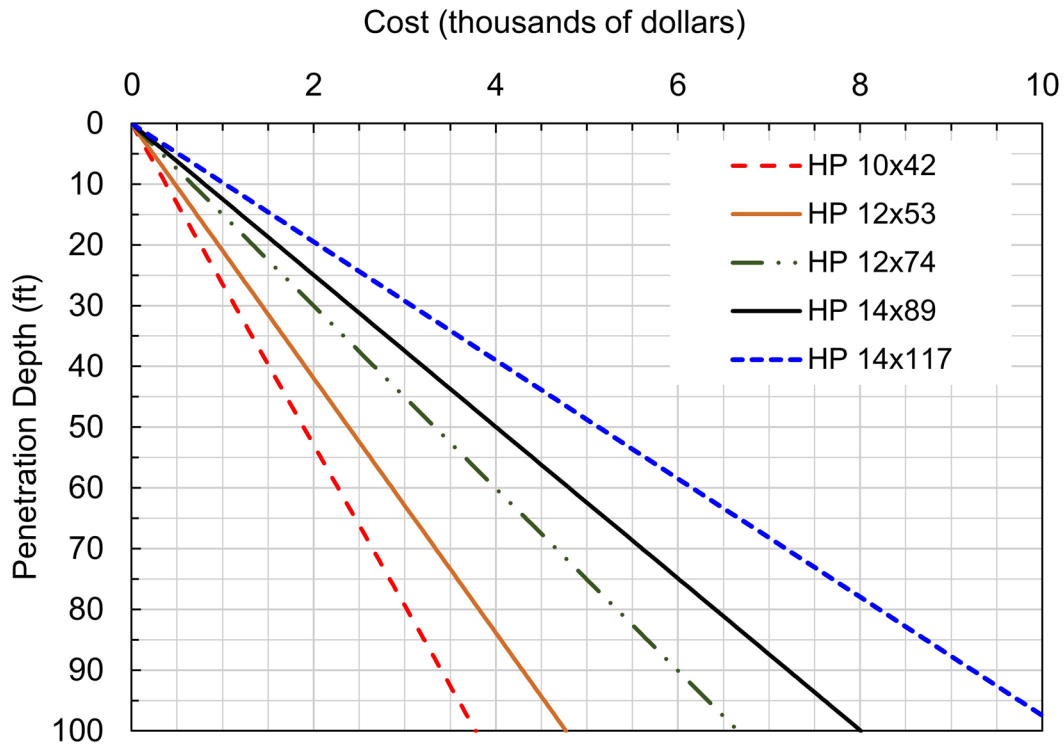


Figure D-80 Pile cost versus penetration depth.

Table D-88 shows the price per pile at Pier 2 with consideration of the established minimum pile penetration depth. For example, for the HP 12x 74 pile section in Group Configuration 6, the individual cost is determined by multiplying the cost per foot by the penetration depth. At the minimum pile penetration depth of 56 feet, the cost per pile is \$3,730. To note, there will be additional pile length embedded in the cap, however for pile cost estimation it can be considered negligible. Furthermore Table D-89 shows the pile group cost reflecting the cost per pile and number of piles in the group.

Table D-88 Cost per Pile at Established Minimum Penetration Depth for Piles Meeting Structural Requirements

Group Configuration	HP 12x53	HP 12x74	HP 14x89	HP 14x117
4	-	-	\$4,806	\$6,156
5	-	\$3,996	4,806	6,156
6	\$2,767	3,730	4,165	5,027

Table D-89 Pile Group Cost at Established Minimum Penetration Depth for Piles Meeting Structural Requirements

Group Configuration	Number of Piles	HP 12x53	HP 12x74	HP 14x89	HP 14x117
4	15	-	-	\$72,090	\$92,340
5	20	-	\$79,920	96,120	123,120
6	20	\$55,332	74,600	83,304	100,548

Based upon this comparison, the HP 14x117 pile section proves to be least economical. Conversely, based upon pile cost alone, the HP 12x53 pile section in Group Configuration 6 appears to be the most economical, followed by the HP 14x89 in Group Configuration 4 and the HP 12x74 pile section in Group Configuration 6. Before selecting the lowest cost option solely considering the results in Table D-89, the cost of the pile cap should also be factored into the foundation cost, as discussed below.

Section 8.9 of Chapter 8 outlines a procedure to estimate the total pile cap thickness. Equation 8-77 is used to estimate this value along using the factored load per pile at the Strength limit state as previously presented in Table D-61. A sample calculation is shown for the HP 12x74 pile in Group Configuration 6, while tabulated results for each pile section and pile group permutation are presented in Table D-90.

Estimate the total pile cap thickness:

P_{ui} = maximum single pile factored axial load, Q =323 kips (Group Configuration 6, Table D-61).

$$t_{cap} = \frac{P_{ui}}{12} + 30 \quad [\text{Eq. 8-80}]$$

$$t_{cap} = \frac{(257 \text{ kips})}{12} + 30$$

$$t_{cap} = 51 \text{ inches}$$

Table D-90 Estimated Total Pile Cap Thickness

Group Configuration	HP 12x53 (inches)	HP 12x74 (inches)	HP 14x89 (inches)	HP 14x117 (inches)
4	-	-	65	65
5	-	55	55	55
6	51	51	51	51

Using the estimated total pile cap thickness, the volume of reinforce concrete required to construct the pile cap is determined. Total pile cap width and length values were previously provided in Table D-59. The resulting volume of reinforced concrete for each pile section and pile group permutation is presented in Table D-91. Note that pile cap volume is shown in cubic yards (CY).

Table D-91 Estimated Volume of Reinforced Concrete in Pile Cap

Group Configuration	HP 12x53 (CY)	HP 12x74 (CY)	HP 14x89 (CY)	HP 14x117 (CY)
4	-	-	24.7	24.7
5	-	28.2	28.2	28.2
6	51.8	51.8	51.8	51.8

To estimate the pile cap cost, past pricing information is generally the best guide; however, similar to estimating the pile cost, due to fluctuations in the market price of material and other factors, pile cap costs are subject to change. The cost of the reinforced concrete pile cap, furnished and constructed, is estimated to be \$500 /CY. Using this estimated value, the volumes presented in Table D-91 were used to estimate the cost of the various reinforced concrete pile caps. The pile cap cost for each candidate section and group configuration permutation is shown in Table D-92.

Table D-92 Estimated Cost of Reinforced Concrete Pile Cap

Group Configuration	HP 12x53	HP 12x74	HP 14x89	HP 14x117
4	-	-	\$12,331	\$12,331
5	-	\$14,110	14,110	14,110
6	\$25,887	25,887	25,887	25,887

By adding the cost of the pile cap and piles for each permutation, the estimated total foundation cost is determined as presented in Table D-93. Additional construction costs should be accounted for such as excavations or required pile installation aides, however an exhaustive analysis is not presented in this design example. Considering both the pile and the pile cap costs, the HP 14x89 pile section in Group Configuration 4 is the more economical option.

Table D-93 Estimated Foundation Cost Including Piles and Pile Cap at Pier 2

Group Configuration	HP 12x53	HP 12x74	HP 14x89	HP 14x117
4	-	-	\$84,421	\$104,671
5	-	\$94,030	110,230	137,230
6	\$81,219	100,479	109,191	126,435

While the cost of pile over/underrun was evaluated at the North Abutment to account for inaccuracies in the assumed soil strength properties and estimated soil resistance, evaluation was not performed at Pier 2. Depending on the pile section and group configuration, piles must be driven to bedrock, or within 5 to 10 feet of bedrock, to satisfy tolerable deformations. These small variations, considering the total number of piles, reduce the risk of length over/underrun differences between the candidate sections. A further economic assessment is therefore not performed.

D.32 Decision 20: Is the Preliminary Design of All Substructure Foundations Complete?

No. The preliminary foundation design has been completed for the North Abutment and the Pier. The preliminary design needs to be completed for the South Abutment. Return to Block 9 and begin the preliminary design for the next substructure location.

D.33 Block 9: South Abutment – For all Candidate Piles, Calculate Nominal and Factored Structural Resistances

The nominal structural resistance is once again reviewed for the five candidate H-pile sections. The H-pile sections selected for evaluation at the South Abutment once again include a HP 10x42, a HP 12x53, a HP 12x74, a HP 14x89 and a HP 14x117. The piles at the South Abutment have the same unsupported length as those at the North Abutment. Therefore, the nominal structural resistances at the South Abutment are unchanged from those determined previously at the North Abutment. A detailed step by step example for calculation of the nominal structural resistance was previously presented in Section 8.5.3 for an HP 14x117 H-pile section. Therefore, this process is not repeated for the five candidate pile sections. Table D-94 once again presents the calculated nominal structural resistances in axial compression, flexure, and shear for the five candidate sections. An unbraced length of 1 foot was assumed in these calculations.

Table D-94 Nominal Structural Resistances in Axial Compression, Flexure and Shear

H-pile Section	HP 10x42	HP 12x53	HP 12x74	HP 14x89	HP 14x117
P_n , Nominal Resistance in Axial Compression (kips)	618	767	1088	1303	1718
M_{ny} , Nominal Resistance in Weak Axis Flexure (kip-ft)	82	114	118	257	380
M_{nx} , Nominal Resistance in Strong Axis Flexure (kip-ft)	176	295	433	592	807
V_n , Nominal Resistance in Shear (kips)	118	149	214	246	331

At this time it has not been determined whether the piles at the South Abutment will be terminated in the very stiff clay or driven to bedrock. If driven to rock, pile shoes may be recommended to reduce the risk of damage. In this scenario, the applicable resistance factor, ϕ_c , would be 0.5 for resistance in axial compression. If terminated in the overlying clay where damage is unlikely to occur during driving, the applicable resistance factor, ϕ_c , would be 0.6 for resistance in axial compression. In any case, for combined resistance in axial compression and flexure (typically above fixity), the applicable resistance factor, ϕ_c , is 0.7. A resistance factor of $\phi_v = 1.0$ is used for shear, and a resistance factor of $\phi_f = 1.0$ is applicable for flexure only. Table D-95 summarizes the calculated factored structural resistances in axial compression, combined axial compression and flexure, flexure, and shear.

Table D-95 Factored Structural Resistance in Axial Compression, Flexure and Shear

H-pile Section	HP 10x42	HP 12x53	HP 12x74	HP 14x89	HP 14x117
P_r , Factored Resistance in Axial Compression, $\phi_c = 0.5$ (kips)	309	383	544	652	859
P_r , Factored Resistance in Axial Compression and Flexure, $\phi_c = 0.7$ (kips)	433	537	762	912	1203
M_{ry} , Factored Resistance in Weak Axis Flexure, $\phi_f = 1.0$ (kip-ft)	82	114	118	257	380
M_{rx} , Factored Resistance in Strong Axis Flexure, $\phi_f = 1.0$ (kip-ft)	176	295	433	592	807
V_r , Factored Resistance in Shear $\phi_v = 1.0$ (kips)	118	149	214	246	331

D.34 Block 10: South Abutment – For All Candidate Piles, Calculate Nominal and Factored Geotechnical Resistances in Axial Compression and Tension versus Depth. Perform Preliminary Pile Drivability Analyses

The engineering properties of the subsurface materials at the South Abutment were determined in Block 5. The results of the boring program and laboratory tests are now used to develop a design profile the South Abutment. Engineering judgement was used in developing the design profile to delineate the subsurface conditions into layers with similar properties. An effective stress diagram, depicted in Figure D-81, was then developed for this soil profile. This diagram includes the total stress, porewater pressure and effective stress versus depth. Figure D-81 also presents the basic soil profile for quick reference to the relevant soil layers. The effective stress diagram was computed using Equations 5-7 through 5-9 from Chapter 5. These equations are repeated below. For Figure D-81, the total stress, porewater pressure and effective stress were calculated at 1 foot increments using a spreadsheet.

$$\sigma_{vo} = \sum_i^n (\gamma_i h_i) \quad [\text{Eq. 5-7}]$$

$$u = \gamma_w h_w \quad [\text{Eq. 5-8}]$$

$$\sigma'_{vo} = \sum_i^n (\gamma_i h_i) - \gamma_w h_w \quad [\text{Eq. 5-9}]$$

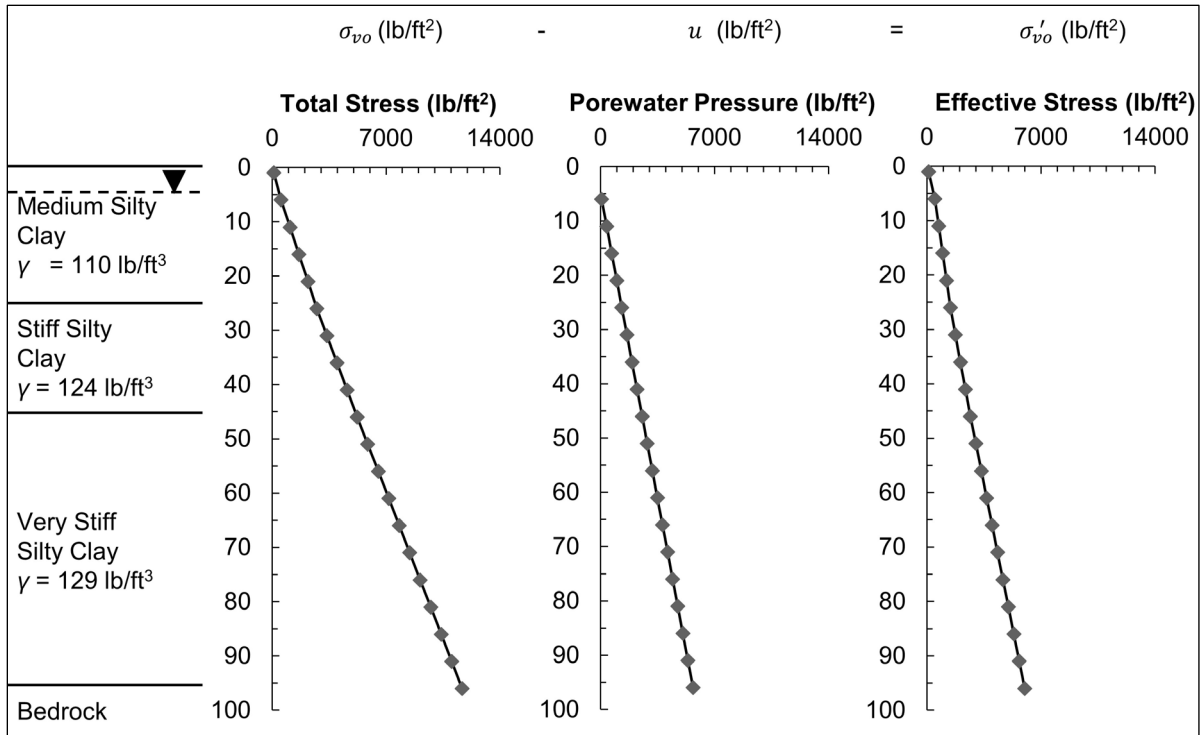


Figure D-81 Effective stress diagram for the South Abutment using Boring S-3.

As indicated by soil boring S-3 (Figures D-4), the soil conditions at the South Abutment consist of silty clay to a depth of 96 feet where bedrock is encountered. Unconfined compression strength tests were performed on each undisturbed cohesive soil sample. The undrained shear strength was then calculated by dividing the unconfined compression strength by 2.

Figure D-82 presents a plot of the undrained shear strength versus depth for the South Abutment soils. As illustrated, the undrained shear strength has distinct changes in undrained shear strength that can be used to identify three distinct soil layers and the layer boundaries. The undrained shear strength also increases approximately linearly with depth within each layer.

Table D-96 summarizes the undrained shear strength values for Soil Layer 1 as well as sublayers a through e. Similarly, Table D-97 and Table D-98 present the undrained shear strength values for Soil Layer 2 and Soil Layer 3 respectively, as well as the individual sublayers.

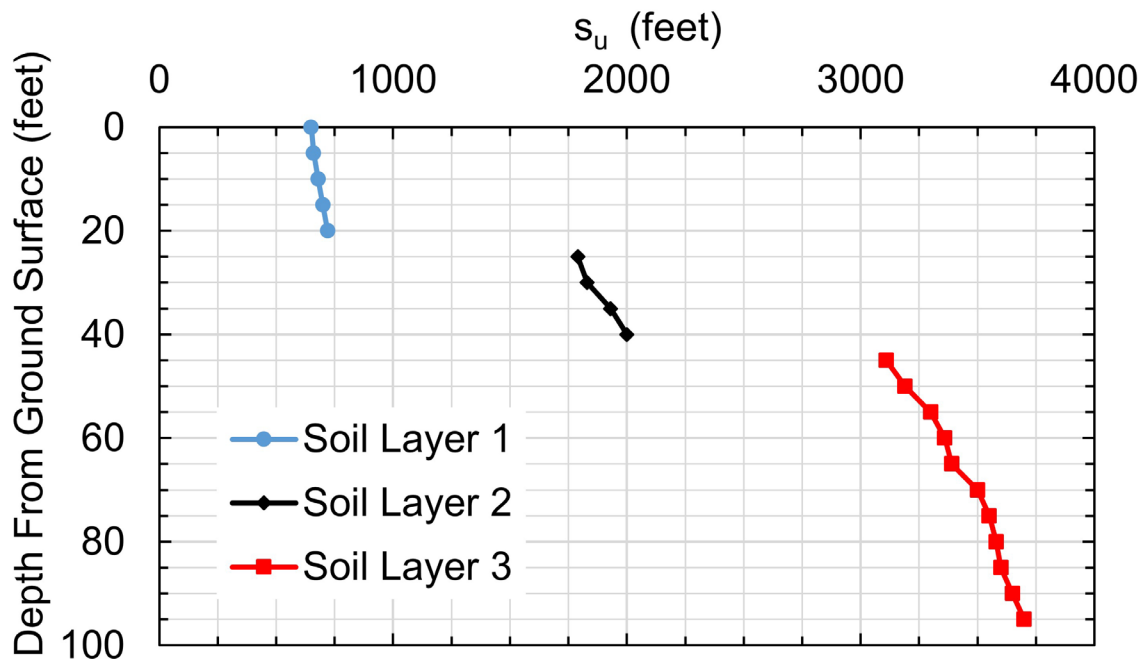


Figure D-82 Undrained shear strength, s_u , versus depth at the South Abutment.

Table D-96 Undrained Shear Strength, s_u , for Soil Layer 1 at the South Abutment

Soil Layer	Sample Depth (ft)	σ'_{vo} (ksf)	q_u (tsf)	s_u (ksf)
1a	1	0.110	0.650	0.650
1b	6	0.488	0.660	0.660
1c	11	0.726	0.680	0.680
1d	16	0.964	0.700	0.700
1e	21	1.202	0.720	0.720

Table D-97 Undrained Shear Strength, s_u , for Soil Layer 2 at the South Abutment

Soil Layer	Sample Depth (ft)	σ'_{vo} (ksf)	q_u (tsf)	s_u (ksf)
2a	26	1.440	1.79	1.79
2b	31	1.748	1.83	1.83
2c	36	2.056	1.93	1.93
2d	41	2.364	2.00	2.00

Table D-98 Undrained Shear Strength, s_u , for Soil Layer 3 at the South Abutment

Soil Layer	Sample Depth (ft)	σ'_{vo} (ksf)	q_u (tsf)	s_u (ksf)
3a	46	2.672	3.11	3.11
3b	51	3.005	3.19	3.19
3c	56	3.338	3.30	3.30
3d	61	3.671	3.36	3.36
3e	66	4.004	3.39	3.39
3f	71	4.337	3.50	3.50
3g	76	4.670	3.55	3.55
3h	81	5.003	3.58	3.58
3i	86	5.336	3.60	3.60
3j	91	5.669	3.65	3.65
3k	96	6.002	3.70	3.70

The design soil profile for the South Abutment is presented in Figure D-83. The bottom of pile cap (Elev. 305 feet) is 5 feet below ground surface. Some soil properties in the design soil profile such as the initial cyclic modulus of subgrade reaction, k_c , have been selected based on published correlations in the absence of laboratory testing. For this particular soil profile, the initial cyclic moduli of subgrade reaction was selected based on the values given in Table 7-22 which are delineated by soil shear strength. Similarly, the 50% strain factor, ϵ_{50} , was estimated based upon the values given in Table 7-21 which vary by soil shear strength. If piles will be driven to bedrock, strength properties of the limestone bedrock are needed. These rock parameters obtained from laboratory tests on recovered rock core samples are summarized in Table D-99.

Table D-99 Laboratory Determined Properties of Limestone Bedrock

Effective Unit Weight (kcf)	Undrained Shearing Resistance, s_u (ksf)	Friction Angle, ϕ (degrees)
0.165	476	35.5

D.34.1 Geotechnical Resistance in Axial Compression

The nominal geotechnical resistance in axial compression is now calculated for the selected candidate H-pile sections. The nominal geotechnical resistance can be calculated by hand or with computer software using an appropriate analysis method

		γ_{sat}	γ'	N	$(N_1)_{60}$	E_s	ϕ'	s_u	k_c	ϵ_{50}
		pcf	pcf			ksf		psf	lb/in ³	
1. Medium Silty Clay (CL)	5 ft							650		
								660		
		110	47.6	--	--		--	680	-	0.01
	20 ft							700		
								720		
2. Stiff Silty Clay (CL)	20 ft							1790		
								1830		
		124	61.6	--	--		--	1930	200	0.007
								2000		
3. Very Stiff Silty Clay (CL)	51 ft							3110		
								3190		
								3300		
								3360		
								3390		
		129	66.6	--	--		--	3500	400	0.005
								3550		
								3580		
								3600		
								3650		
						3700				
4. Limestone Bedrock		REC: 93% RQD: 88%								

Figure D-83 Design soil profile at the South Abutment.

based upon soil, rock, and pile type. Chapter 7 describes appropriate methods for this purpose as well as computer programs available at the time of this manual's publication.

For this abutment, the DrivenPiles computer program was used to calculate the nominal resistance, shaft resistance, and toe resistance each as a function of depth for each of the five candidate H-piles. For nominal geotechnical resistance calculations, each cohesive sample depth in Soil Layers 1, 2, and 3 was treated as an individual layer with respect to shear strength. This software program was selected since it uses the FHWA recommended alpha method to calculate the nominal geotechnical resistance of H-piles in cohesive soils. A summary of the shaft, toe, and nominal geotechnical resistance is presented for the HP 12x74 H-pile section in Table D-100. The depth shown is taken from the bottom of pile cap.

In the DrivenPiles program, the selection of the geomaterial is limited to either cohesionless or cohesive soil. Hence, the program cannot calculate the nominal toe resistance on rock and the only choice is either to model rock as a hard cohesive soil or a very dense cohesionless soil. The nominal toe resistance presented in Table D-100 at depth 91.25 feet was not calculated using the DrivenPiles program but rather calculated using the procedure for hard rock presented in Section 7.2.1.4.2 of Chapter 7.

Table D-100 Nominal Shaft, Nominal Toe and Nominal Geotechnical Resistance for
HP 12x74 at the South Abutment

Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Geotechnical Resistance (kips)	Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Geotechnical Resistance (kips)
0.01	0.02	6.09	6.11	32	116.23	17.81	134.03
1	2.21	6.09	8.3	33	122.25	17.81	140.05
2	4.42	6.09	10.51	34	128.36	17.81	146.16
3	6.63	6.09	12.72	34.99	134.49	17.81	152.3
4	8.85	6.09	14.93	35.01	134.62	18.45	153.07
4.99	11.03	6.09	17.12	36	140.69	18.45	159.14
5.01	11.08	6.27	17.35	37	146.83	18.45	165.28
6	13.35	6.27	19.62	38	152.96	18.45	171.41
7	15.67	6.27	21.95	39	159.1	18.45	177.55
8	18.03	6.27	24.3	39.99	165.17	18.45	183.62
9	20.42	6.27	26.69	40.01	165.28	28.69	193.98
9.99	22.81	6.27	29.08	41	170.34	28.69	199.03
10.01	22.86	6.46	29.32	42	175.44	28.69	204.13
11	25.27	6.46	31.73	43	180.54	28.69	209.24
12	27.75	6.46	34.21	44	185.65	28.69	214.34
13	30.25	6.46	36.71	44.99	190.7	28.69	219.39
14	32.79	6.46	39.25	45.01	190.8	29.43	220.23
14.99	35.34	6.46	41.79	46	195.85	29.43	225.28
15.01	35.39	6.64	42.03	47	200.95	29.43	230.39
16	37.96	6.64	44.6	48	206.06	29.43	235.49
17	40.58	6.64	47.23	49	211.16	29.43	240.59
18	43.24	6.64	49.89	49.99	216.21	29.43	245.64
19	45.94	6.64	52.58	50.01	216.31	30.45	246.76
19.99	48.63	6.64	55.28	51	221.37	30.45	251.81
20.01	48.71	16.51	65.23	52	226.47	30.45	256.92
21	53.96	16.51	70.48	53	231.57	30.45	262.02
22	59.34	16.51	75.85	54	236.68	30.45	267.12
23	64.8	16.51	81.31	54.99	241.73	30.45	272.17
24	70.33	16.51	86.84	55.01	241.83	31	272.83
24.99	75.89	16.51	92.4	56.00	246.88	31	277.88
25.01	76	16.88	92.88	57	251.98	31	282.98
26	81.48	16.88	98.37	58	257.09	31	288.09
27	87.1	16.88	103.99	59	262.19	31	293.19
28	92.8	16.88	109.69	59.99	267.24	31	298.24
29	98.59	16.88	115.47	60.01	267.34	31.28	298.62
29.99	104.39	16.88	121.27	61	272.4	31.28	303.67
30.01	104.51	17.81	122.31	62	277.5	31.28	308.78
31	110.29	17.81	128.1	63	282.6	31.28	313.88

Table D-100 Nominal Shaft, Nominal Toe and Nominal Geotechnical Resistance for HP 12x74 at the South Abutment (Continued)

Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Geotechnical Resistance (kips)	Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Geotechnical Resistance (kips)
64	287.71	31.28	318.98	78	359.15	33.03	392.18
64.99	292.76	31.28	324.03	79	364.25	33.03	397.28
65.01	292.86	32.29	325.15	79.99	369.3	33.03	402.33
66	297.91	32.29	330.2	80.01	369.4	33.21	402.62
67	303.01	32.29	335.31	81	374.46	33.21	407.67
68	308.12	32.29	340.41	82	379.56	33.21	412.77
69	313.22	32.29	345.51	83	384.66	33.21	417.88
69.99	318.27	32.29	350.56	84	389.77	33.21	422.98
70.01	318.37	32.75	351.13	84.99	394.82	33.21	428.03
71	323.43	32.75	356.18	85.01	394.92	33.68	428.59
72	328.53	32.75	361.28	86	399.97	33.68	433.65
73	333.63	32.75	366.39	87	405.07	33.68	438.75
74	338.74	32.75	371.49	88	410.18	33.68	443.85
74.99	343.79	32.75	376.54	89	415.28	33.68	448.96
75.01	343.89	33.03	376.92	90	420.33	33.68	454.01
76	348.94	33.03	381.97	91	420.43	33.68	454.11
77	354.04	33.03	387.07	91.25	420.43	1527.00	1947.43

For piles driven to hard rock, the nominal toe resistance should be determined using Equation 7-34 as described in Section 7.2.1.4.2. The strength properties of the limestone bedrock were presented in Table D-99. The unit toe resistance, q_p , of the hard rock can then be calculated using Equation 7-34.

$$q_p = P_s s_u N_c + \gamma D N_q + P_t \gamma \left(\frac{b N_\gamma}{2} \right) \quad [\text{Eq. 7-34}]$$

This computation requires determination of the bearing capacity factors N_c , N_q , and N_γ from Figure D-84. Entering the figure with a friction angle of 35.5 degrees, the bearing capacity factors N_c , N_q , and N_γ , were determined to be 17, 14, and 24, respectively. Because piles will likely be seated on bedrock, the pile penetration into the rock surface is assumed as 0.25 feet.

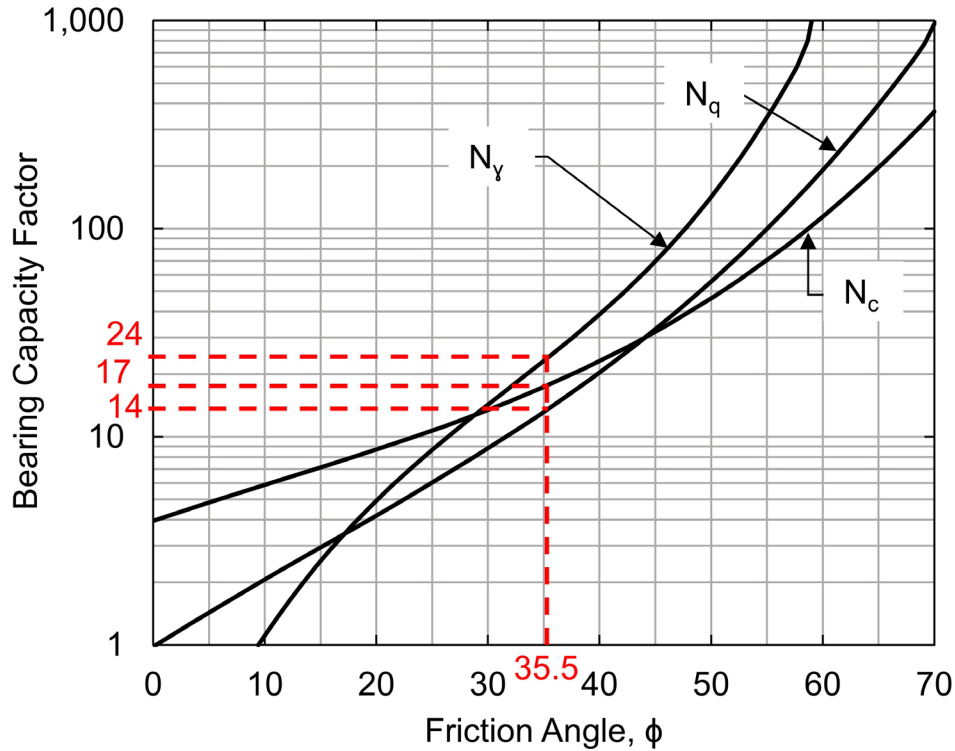


Figure D-84 Bearing capacity factors for foundations on rock.

Equation 7-34 is now used to determine the unit resistance, q_p , of the rock.

- s_u = undrained shearing resistance of the rock, 476 ksf.
- γ = effective density of the rock mass, 0.165 kcf.
- D = pile penetration below the rock surface, assume 0.25 feet.
- b = pile width or diameter 1 foot (HP 12x74).
- P_s = pile shape factor of 1.25 for square pile toe.
- P_t = pile base factor of 0.80 for a square pile toe.
- N_c = bearing capacity factor, 17 (Figure D-84).
- N_q = bearing capacity factor, 14 (Figure D-84).
- N_γ = bearing capacity factor, 24 (Figure D-84).

$$q_p = P_s s_u N_c + \gamma D N_q + P_t \gamma \left(\frac{b N_\gamma}{2} \right) \quad [\text{Eq. 7-34}]$$

$$\begin{aligned} q_p &= (1.25) * (476 \text{ ksf}) * (17) \\ &\quad + (0.165 \text{ kcf}) * (0.25 \text{ feet}) * (14) \\ &\quad + (0.80) * (0.165 \text{ kcf}) * \left(\frac{(1 \text{ foot}) * (24)}{2} \right) \end{aligned}$$

$$q_p = 10,117 \text{ ksf}$$

For the HP 12x74 pile section with toe area (area of steel, A_s) of 0.151 ft², the nominal toe resistance is then calculated as follows:

$$R_p = q_p * A_s$$

$$R_p = (10,117 \text{ ksf}) * (0.151 \text{ ft}^2)$$

$$R_p = 1,528 \text{ kips}$$

The nominal shaft, nominal toe, and nominal geotechnical resistance are also presented graphically in Figure D-85. A review of Table D-94 indicates the nominal structural resistance for the HP 12x74 section is 1088 kips, which is less than the nominal toe resistance when driven to hard limestone bedrock. Therefore, the nominal structural resistance will control the design. The penetration depth indicated in both Table D-100 and Figure D-85 is referenced from the bottom of footing, which is 5 feet below the original ground surface elevation. The nominal shaft, nominal toe, and nominal geotechnical resistances were calculated for all of the candidate pile sections in a similar manner. The nominal geotechnical resistances in axial compression versus pile penetration depth for all candidate pile types are presented in Figure D-86.

Once the nominal geotechnical resistance in axial compression versus pile penetration depth has been calculated, the factored geotechnical resistance in axial compression versus pile penetration depth can be determined. The factored geotechnical resistance depends on the resistance determination method selected for the design.

If the nominal resistance will be confirmed by a field determination method, the factored geotechnical resistance in axial compression as a function of pile penetration depth can be determined by multiplying the calculated nominal geotechnical resistance at a given depth by the resistance factor associated with the field determination method, ϕ_{dyn} . AASHTO (2014) resistance factors for field resistance determination methods are presented in Table 7-2.

If pile installation will be controlled by driving to a depth determined by static analysis method, then the factored geotechnical resistance in axial compression as a function of pile penetration depth can be determined by multiplying the calculated nominal resistance at a given depth by the resistance factor associated with the static analysis method, ϕ_{stat} . AASHTO (2014) resistance factors for static analysis methods are presented in Table 7-1 of this manual.

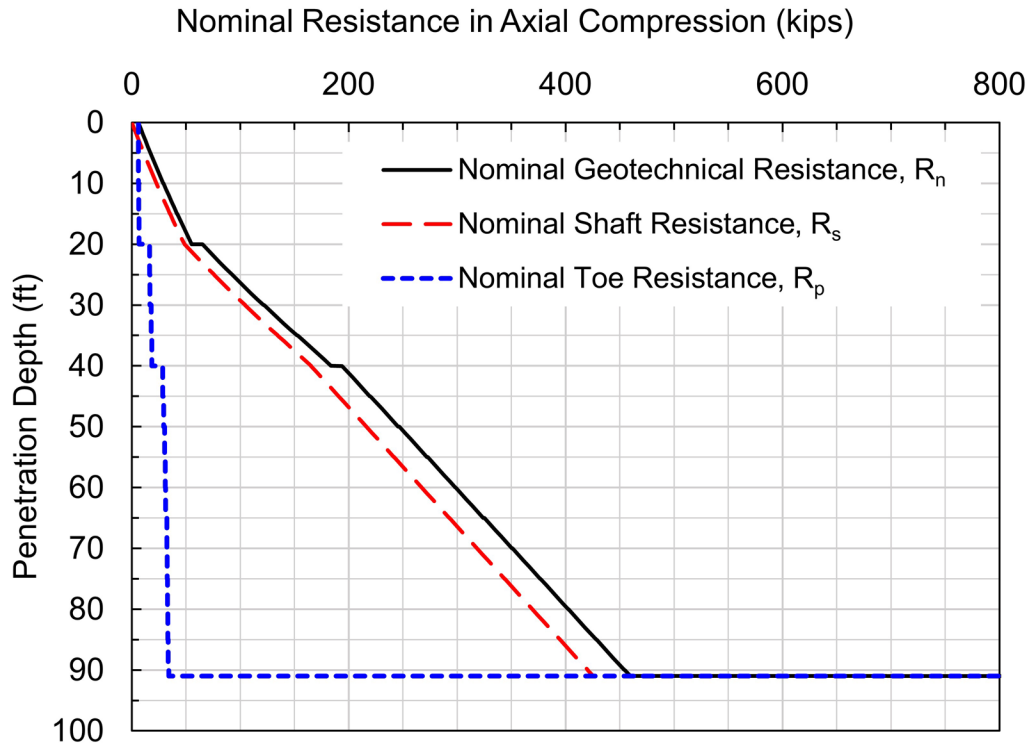


Figure D-85 Nominal geotechnical resistance in axial compression versus pile penetration depth for HP 12x74 at the South Abutment.

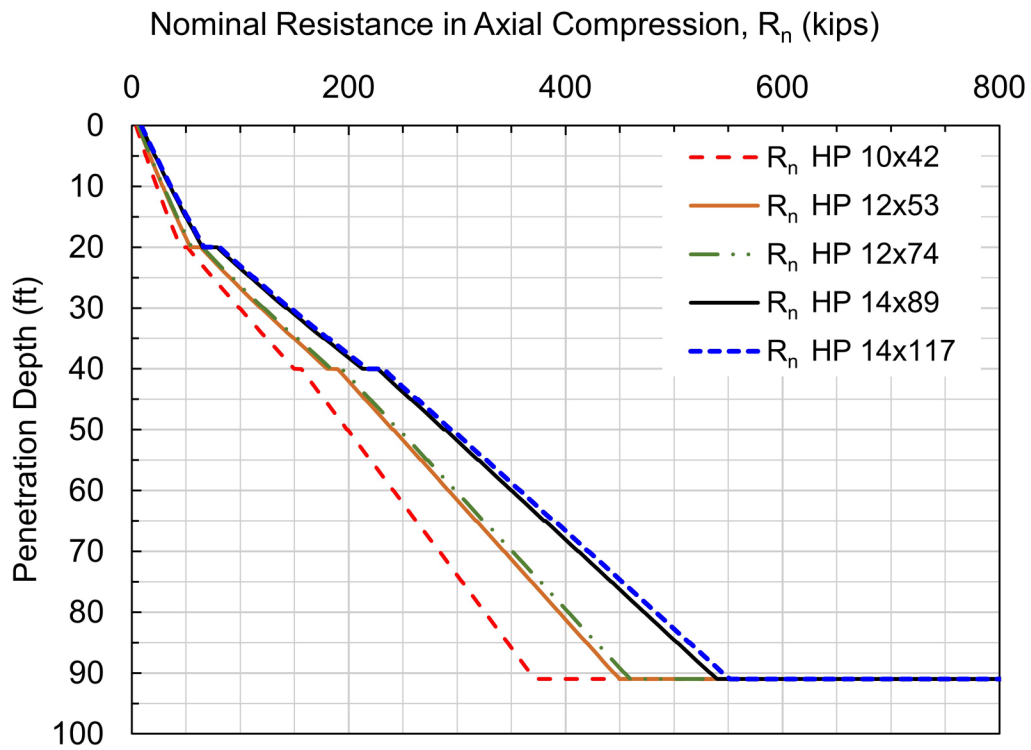


Figure D-86 Nominal geotechnical resistance in axial compression versus pile penetration depth for all candidate pile sections at South Abutment.

Figure D-87 presents a design chart of the nominal and factored geotechnical resistance in axial compression versus depth for the HP 12x74 candidate H-pile section. The design chart includes the nominal geotechnical resistance as well as factored geotechnical resistances based on several resistance determination methods. The factored geotechnical resistance versus penetration depth is plotted for resistance determination by a static load test with dynamic testing of 2% of the piles ($\phi_{dyn}=0.80$), by dynamic testing of at least two piles per site condition, but no less than 2% of the production piles, with signal matching ($\phi_{dyn}=0.65$), and by wave equation analysis ($\phi_{dyn}=0.50$). Also included is the factored geotechnical resistance based on the static analysis method used at the abutment, the alpha method ($\phi_{stat}=0.35$). An AASHTO resistance factor is not currently available for the static analysis method used for hard rock. Therefore, the field resistance determination method would control pile installation.

For the HP 12x74 section, Figure D-87 illustrates the effects that the various resistance determination methods have on the pile length required for a given factored compression resistance, the range of factored resistances available from a given pile section, and the potential impact of these factors on the number of piles needed to resist axial compression loads.

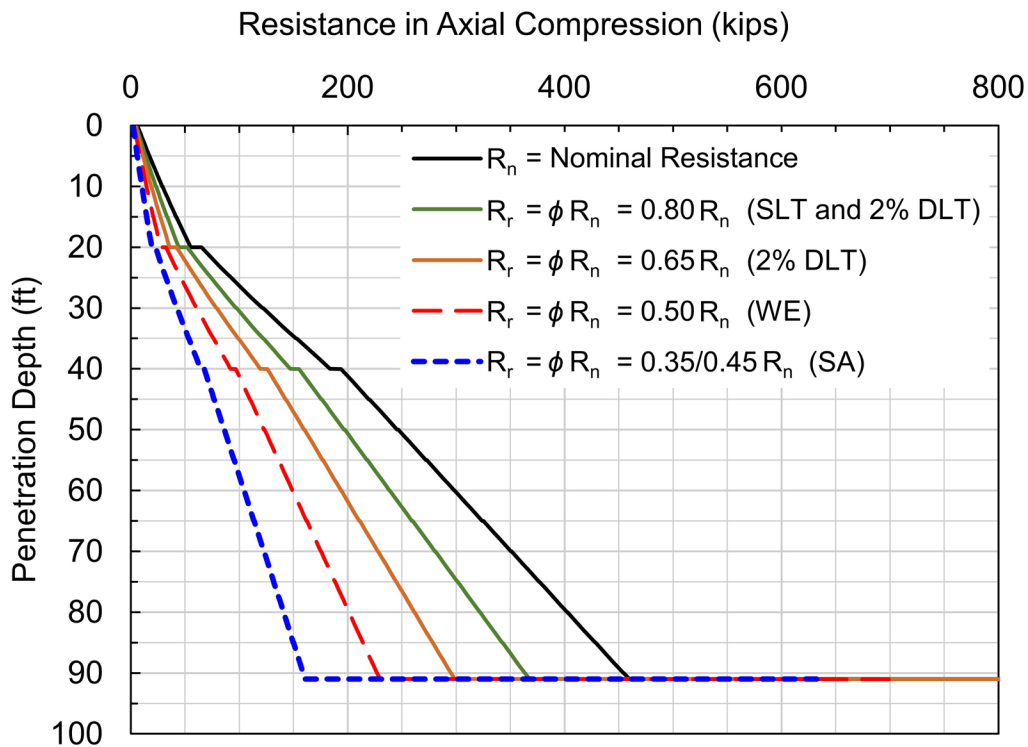


Figure D-87 Design chart of nominal and factored geotechnical resistance in axial compression versus pile penetration depth for HP 12x74 at the South Abutment.

The factored geotechnical resistance in axial compression for all candidate pile sections is presented in Figures D-88 through D-91 based on the same resistance determination method. For all the candidate pile sections, these figures illustrate the effects the various resistance determination methods have on the pile length required for a given factored resistance, the factored resistance available from a given pile section, and the potential impact of these factors on the number of piles needed to resist axial compression loads.

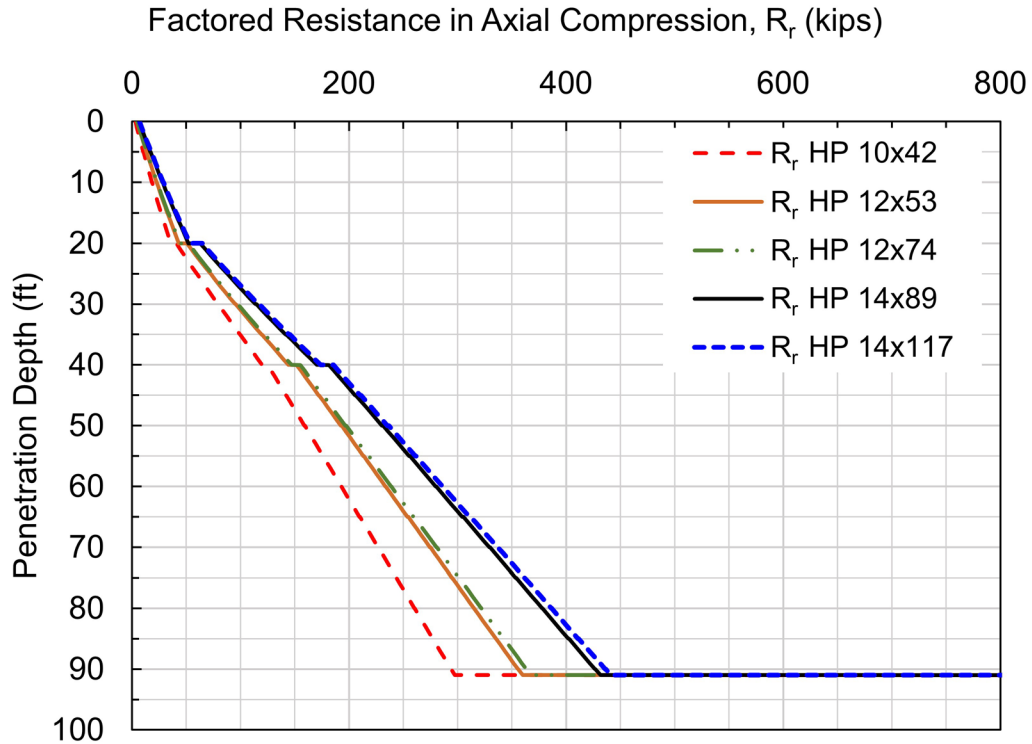


Figure D-88 Factored geotechnical resistance, R_r , in axial compression based on field determination by static load test and dynamic testing 2% of the piles, $\phi_{dyn}=0.80$.

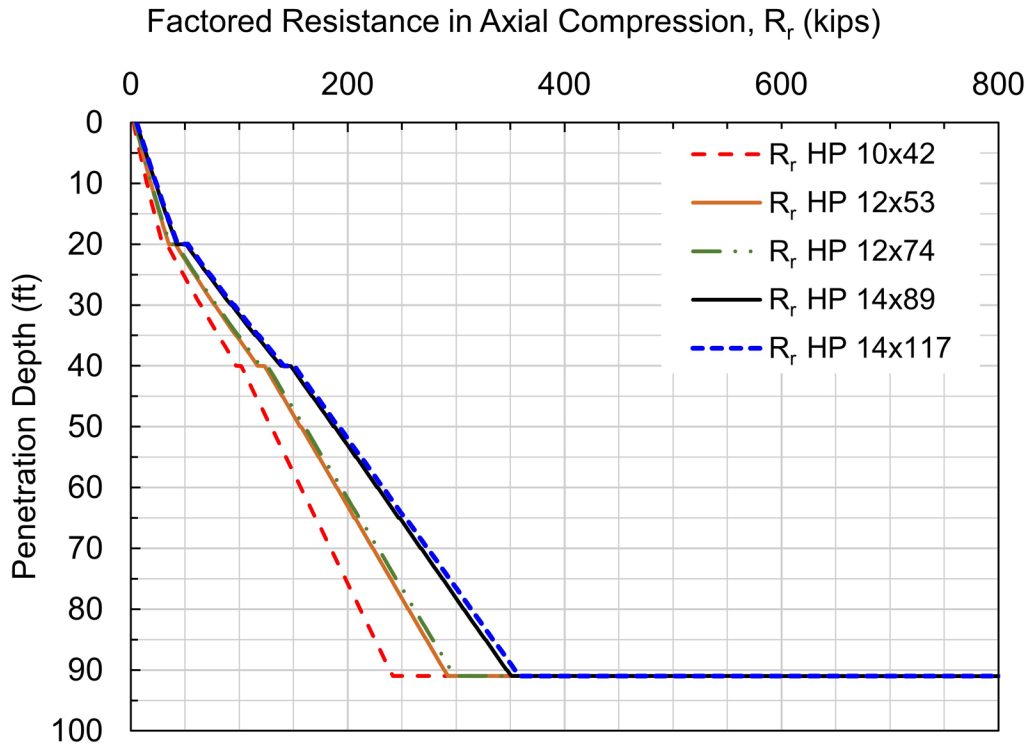


Figure D-89 Factored geotechnical resistance, R_r , in axial compression based on field determination by dynamic testing 2% of the piles, $\phi_{dyn}=0.65$.

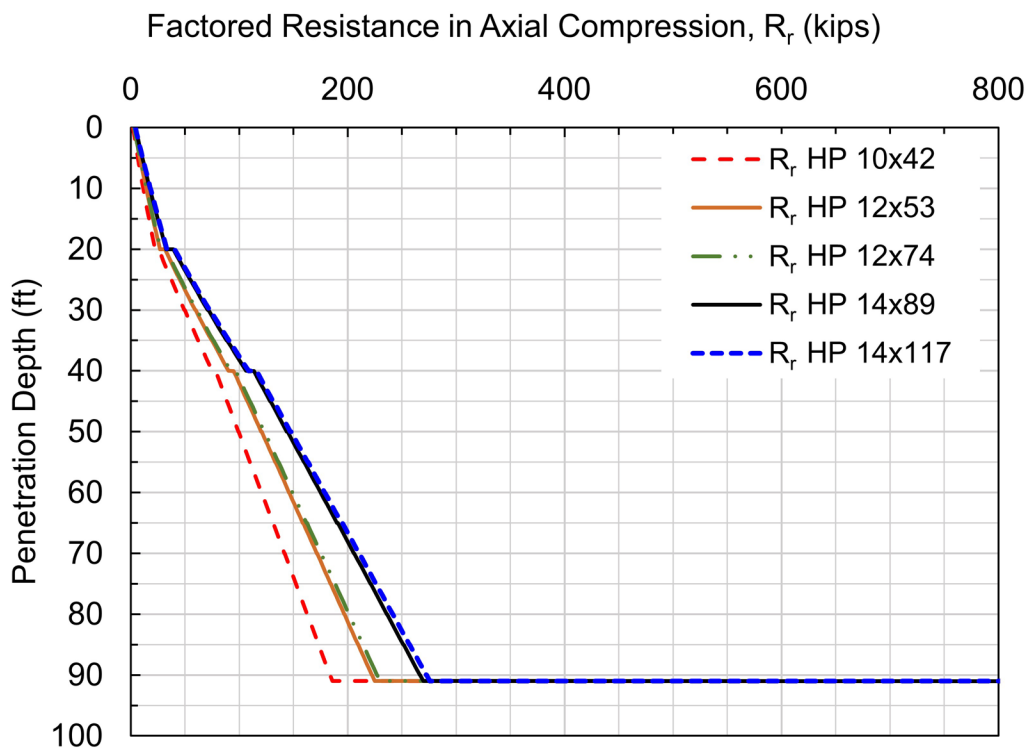


Figure D-90 Factored geotechnical resistance, R_r , in axial compression based on field determination by wave equation analysis, $\phi_{dyn}=0.50$.

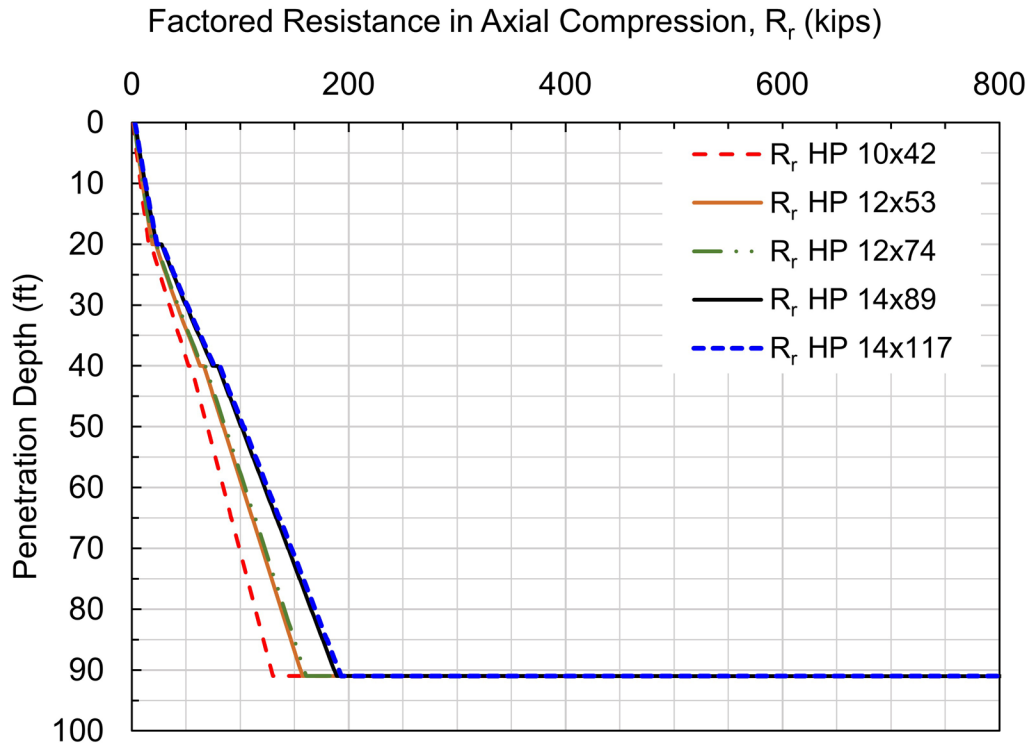


Figure D-91 Factored geotechnical resistance, R_r , in axial compression based on determination using alpha method static analysis, $\phi_{stat}=0.35$.

D.34.2 Geotechnical Resistance in Axial Tension (uplift)

In a similar manner, the nominal and factored geotechnical resistances in axial tension (uplift) were calculated using the alpha method and with the DrivenPiles computer program. Figure D-92 presents the nominal shaft resistance versus penetration depth for all the candidate pile sections. As outlined in Section 7.2.3.2.1, the factored resistance in axial tension is the shaft resistance multiplied by the resistance factor, ϕ_{up} , for the resistance determination method.

Figure D-93 presents a design chart of the nominal and factored geotechnical resistance in axial tension versus pile penetration depth for HP 12x74 H-pile section. A resistance factor of 0.60 is used when the tension resistance is determined by a static load test, 0.50 when determined by a dynamic test with signal matching, and 0.25 when determined by the alpha method static analysis. For a single pile section (HP 12x74), this figure illustrates the effects the various resistance determination methods have on the pile length required for a given factored resistance, the factored resistance available from a given pile section, and the potential impact of these factors on the number of piles needed to resist axial tension loads.

Figures D-94 to D-96 present the factored geotechnical resistance in axial tension versus penetration depth based on the field determination method for each of the candidate pile sections. For all the candidate pile sections, these figures illustrate the effects the various resistance determination methods have on the pile length required for a given factored resistance, the factored resistance available from a given pile section, and the potential impact of these factors on the number of piles needed to resist axial tension loads.

A review of the soil profile at the South Abutment indicates no unsuitable soil layers are present that should be ignored for load support. Abutment scour and liquefaction due to a seismic event are also not considerations.

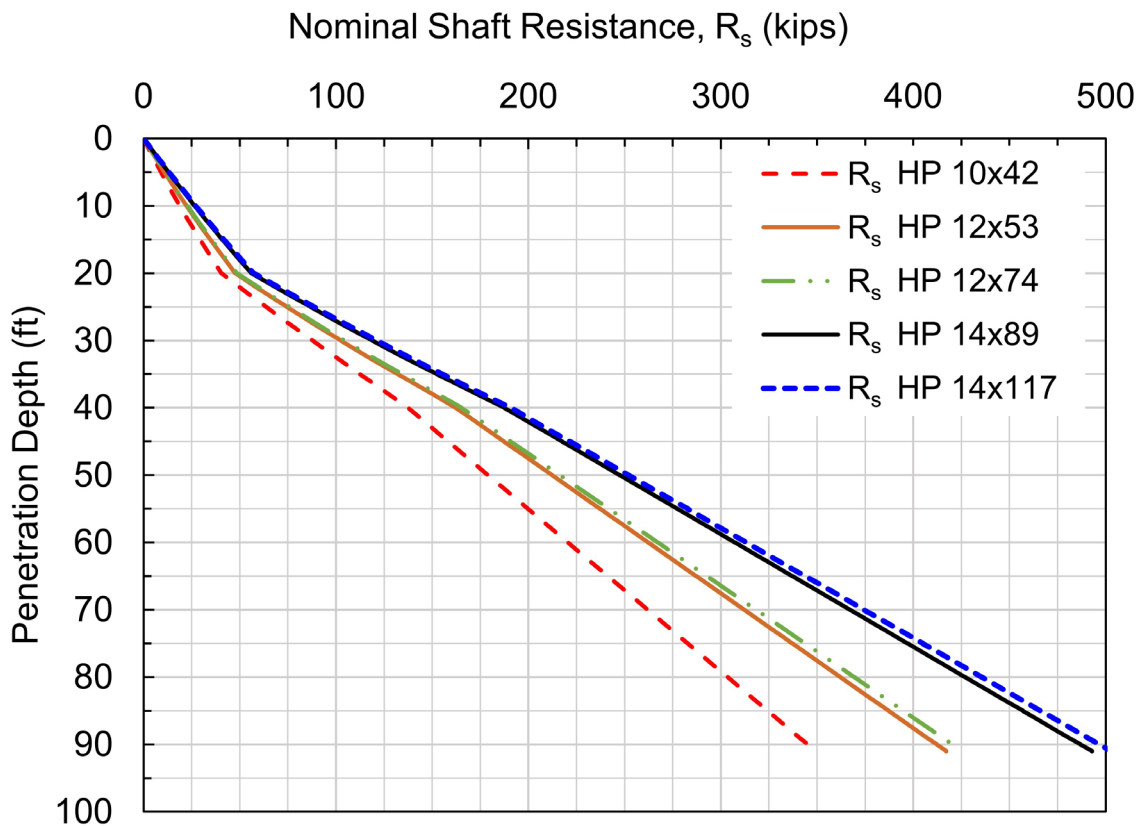


Figure D-92 Nominal shaft resistance versus penetration depth for all candidate pile sections at the South Abutment.

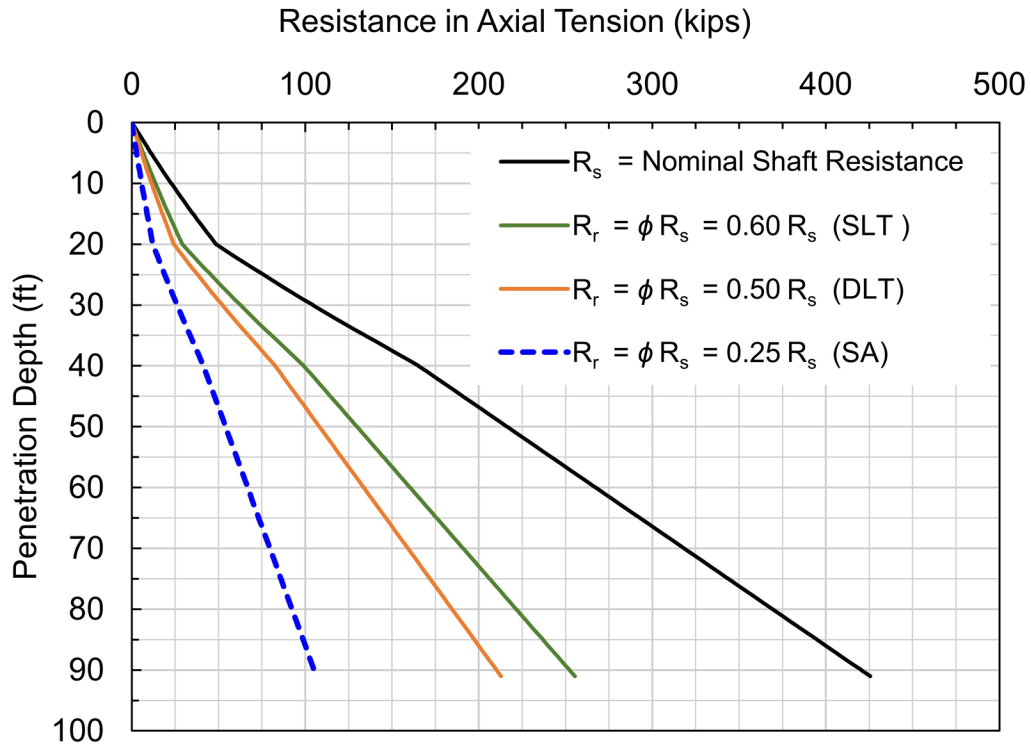


Figure D-93 Design chart of nominal and factored geotechnical resistance in axial tension versus pile penetration depth for HP 12x74 at the South Abutment.



Figure D-94 Factored geotechnical resistance, R_r , in axial tension based on field determination by static load test, $\phi_{dyn}=0.60$.

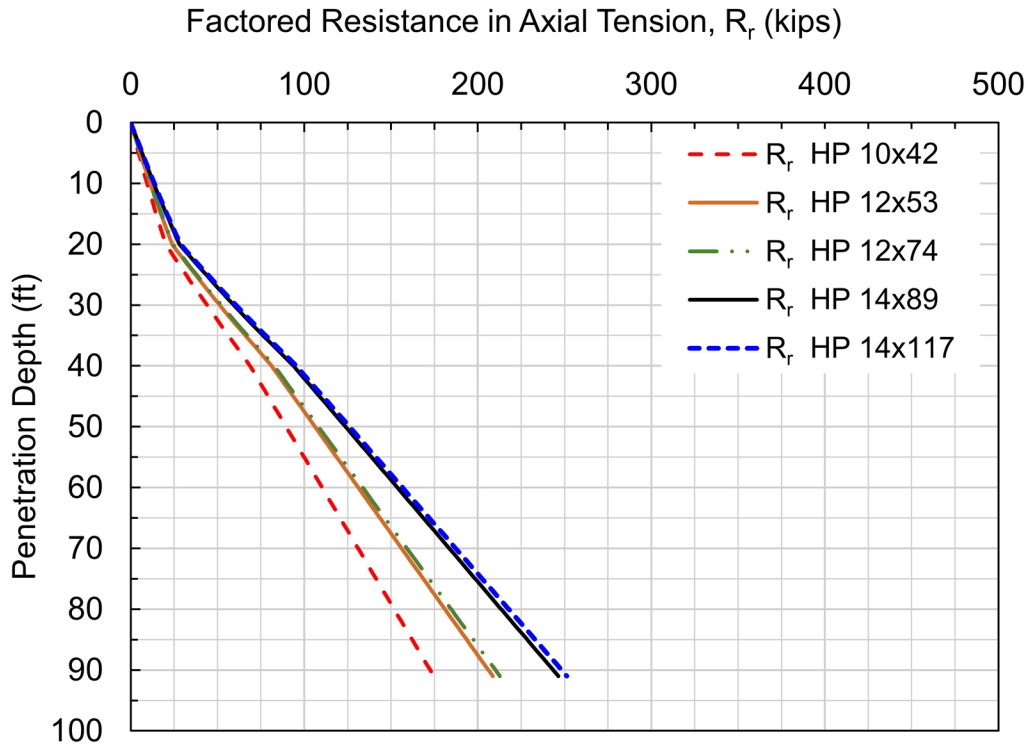


Figure D-95 Factored geotechnical resistance, R_r , in axial tension based on field determination by dynamic testing with signal matching, $\phi_{dyn}=0.50$.

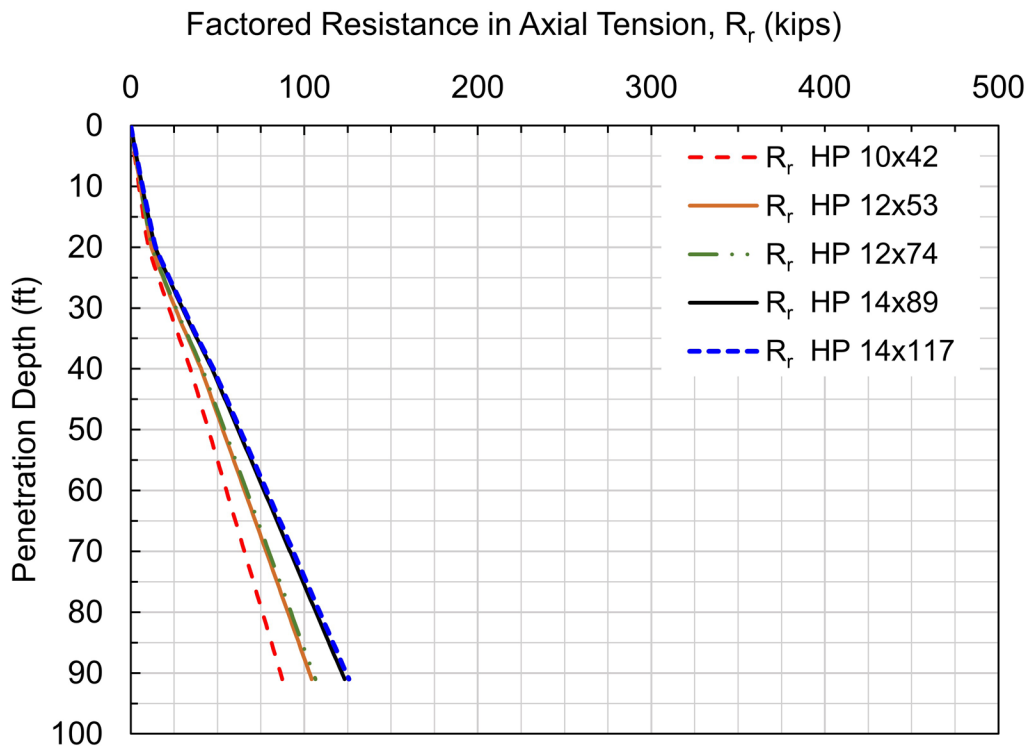


Figure D-96 Factored geotechnical resistance, R_r , in axial tension based on determination using alpha method static analysis, $\phi_{stat}=0.25$.

D.34.3 Preliminary Pile Drivability Assessment

Preliminary assessments of pile drivability are now performed at this stage of the design. A drivability check at this time is essential to assess the constructability of candidate pile types and/or sections and to eliminate sections with insufficient drivability. Section 12.4 provides a detailed discussion of wave equation drivability analyses and their applications.

A candidate pile section must be capable of being driven to the nominal driving resistance. In this example, the nominal driving resistance is less than the nominal geotechnical resistance since soil set up will be incorporated into the design. In addition, a candidate pile section must be capable of being driven to the penetration depth necessary to achieve the nominal geotechnical resistance in axial tension, and to a penetration depth necessary to satisfy lateral load demands as well as axial and lateral deformation requirements. A suitably sized pile hammer must be capable of driving the pile to its established minimum penetration depth and to the nominal resistance at a reasonable blow count without exceeding material stress limits. As detailed in Chapter 12, the blow count should be between 30 and 120 blows per foot at the nominal resistance. If the pile cannot be driven within these requirements, a larger pile hammer, a pile section with greater impedance, or pile installation aids such as predrilling or jetting may be required to satisfy or improve drivability.

Driving stresses during pile installation should remain below the driving stress limits tied to pile type and material strength. For the candidate steel H-piles, compression driving stress limits are given by Equation 8-33. As per ASTM A-572 requirements, new steel H-piles are rolled with a minimum yield stress of 50 ksi.

The driving stress limit, σ_{dr} , for candidate pile sections is then calculated as follows:

$$\sigma_{dr} = \phi_{da} (0.9 F_y) \quad [\text{Eq. 8-33}]$$

Where: ϕ_{da} = resistance factor, 1.0 for steel piles.
 F_y = yield stress, 50 ksi.

Therefore, the driving stress limit, σ_{dr} , is 45 ksi.

$$\sigma_{dr} = (1.0)(0.9 (50 \text{ ksi})) = 45 \text{ ksi}$$

Drivability analyses were performed for all five candidate pile sections. Since the specific pile hammer is often unknown at this point in the design, a reasonably sized,

commonly available single acting diesel hammer was chosen for each of the candidate pile sections. As noted in Section 15.19, a hammer having a ram weight of 1 to 2% of the larger of the required nominal resistance or required nominal driving resistance often provides a reasonable initial estimate of hammer size for wave equation analysis.

Table D-101 summarizes the factored structural resistance in axial compression, P_r , and the corresponding minimum and maximum nominal driving resistance associated with full section utilization and field determination methods ranging from a static load test with dynamic testing ($\phi_{dyn}=0.80$) to the FHWA modified Gates dynamic formula ($\phi_{dyn}=0.40$). Given that utilization of the full structural resistance is uncommon for piles driven in soil, a reasonable initial estimate of the trial hammer size for a wave equation drivability analysis is 1 to 1.5% of the minimum R_{ndr} . Driving stresses could exceed specified limits by choosing a hammer with a ram weight significantly larger than 2% of the minimum R_{ndr} . For each pile hammer, the wave equation default values were used for the helmet weight, hammer cushion materials, and the hammer cushion material properties.

Table D-101 Summary of Pile Hammers Used in Drivability Analyses

Pile Section	Pile Cross Sectional Area (in ²)	Factored Structural Resistance, P_r (kips)	Minimum R_{ndr} $\phi_{dyn}=0.80$ (kips)	Maximum R_{ndr} $\phi_{dyn}=0.40$ (kips)	Ram Weight 1% of Min R_{ndr} (%)	Diesel Model	Ram Weight (kips)	Rated Energy (ft-kips)
HP 10x42	12.4	309	386	773	3.86	D25-52	5.51	62.0
HP 12x53	15.5	383	478	958	4.78	D30-52	6.62	74.4
HP 12x74	21.8	544	680	1360	6.80	D36-52	7.94	89.3
HP 14x89	26.1	652	815	1630	8.15	D46-52	10.14	114.1
HP 14x117	34.4	860	1075	2150	10.07	D50-52	11.03	124.0

For the soil resistance model, the output from DrivenPiles was converted to unit shaft resistance and unit toe resistance values and then input into the wave equation program. Similar soil resistances are thereby calculated versus depth by both the static analysis and wave equation analysis programs.

The dynamic soil properties for each soil layer were chosen in accordance with wave equation program recommendations. Selection of soil quake and damping parameters is discussed in Section 12.6.7. For the South Abutment profile, setup factors of 2.0 were selected for all of the silty clay layers. No setup was considered

for the hard rock. Soil setup is discussed in Section 7.2.4.2 and a summary of typical soil setup factors is provided in Table 7-16. A summary of the dynamic soil properties chosen for the drivability analyses are summarized in Table D-102.

Table D-102 Dynamic Soil Properties for South Abutment Soil Profile

Soil Layer	Pile Section	Shaft Quake (in)	Toe Quake (in)	Shaft Damping (s/ft)	Toe Damping (s/ft)	Soil Set-Up Factor
1	HP 10 x 42	0.10	0.17	0.20	0.15	2.0
1	HP 12 x 53	0.10	0.20	0.20	0.15	2.0
1	HP 12 x 74	0.10	0.20	0.20	0.15	2.0
1	HP 14 x 89	0.10	0.23	0.20	0.15	2.0
1	HP 14 x 117	0.10	0.23	0.20	0.15	2.0
2	HP 10 x 42	0.10	0.17	0.20	0.15	2.0
2	HP 12 x 53	0.10	0.20	0.20	0.15	2.0
2	HP 12 x 74	0.10	0.20	0.20	0.15	2.0
2	HP 14 x 89	0.10	0.23	0.20	0.15	2.0
2	HP 14 x 117	0.10	0.23	0.20	0.15	2.0
3	HP 10 x 42	0.10	0.17	0.20	0.15	2.0
3	HP 12 x 53	0.10	0.20	0.20	0.15	2.0
3	HP 12 x 74	0.10	0.20	0.20	0.15	2.0
3	HP 14 x 89	0.10	0.23	0.20	0.15	2.0
3	HP 14 x 117	0.10	0.23	0.20	0.15	2.0
4	HP 10 x 42	0.10	0.08	0.05	0.15	1.0
4	HP 12 x 53	0.10	0.10	0.05	0.15	1.0
4	HP 12 x 74	0.10	0.10	0.05	0.15	1.0
4	HP 14 x 89	0.10	0.12	0.05	0.15	1.0
4	HP 14 x 117	0.10	0.12	0.05	0.15	1.0

In soils that exhibit setup, the long term nominal resistance may be higher than the nominal driving resistance. Therefore, a gain/loss factor of 0.50 was used to estimate the nominal driving resistance versus depth in the drivability analyses. This gain/loss factor was determined from the inverse of the highest soil setup factor within the soil model (e.g., 1 divided by 2.0 equals 0.50). A gain/loss factor of 1 would be used if it was desired to model the nominal resistance instead of the nominal driving resistance and not consider the soil strength loss during driving and any subsequent soil setup. Refer to Chapter 12 for more detailed discussion on the selection of dynamic soil properties and soil setup factors.

The DrivenPiles program calculates the nominal driving resistance which models the soil strength lost during driving as well as the geotechnical nominal resistance once setup occurs. Figure D-97 presents the nominal shaft, toe, and driving resistance versus pile penetration depth for the HP 12x74 H-pile section at the South Abutment. These results are also presented numerically in Table D-103. To quantify the expected soil setup at a given pile penetration depth, the values from Table D-103 can be compared against the nominal resistance previously presented in Table D-100. Figure D-98 illustrates the significant difference between the expected nominal driving resistance and the geotechnical nominal resistance after setup for the HP 12x74 candidate pile section. For example, at a depth of 45 feet, the expected setup from shaft resistance is 95 kips. Where significant soil setup is expected, a smaller pile driving hammer may be acceptable for pile installation since the nominal driving resistance will be significantly lower than the nominal geotechnical resistance.

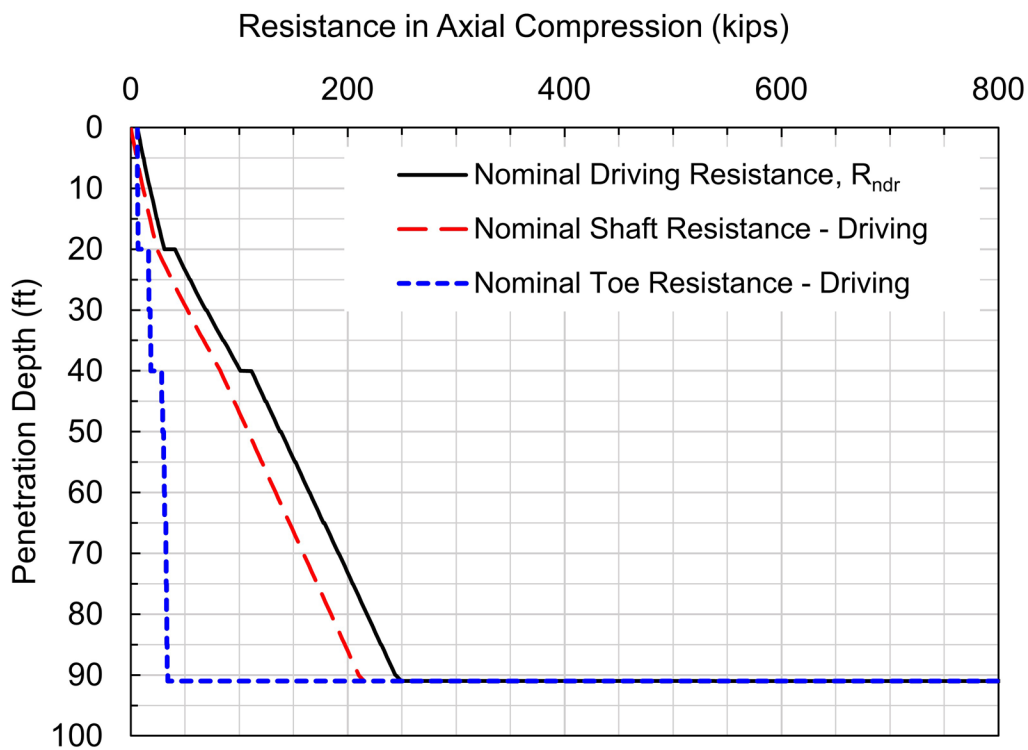


Figure D-97 Nominal driving resistance for HP 12x74 at the South Abutment.

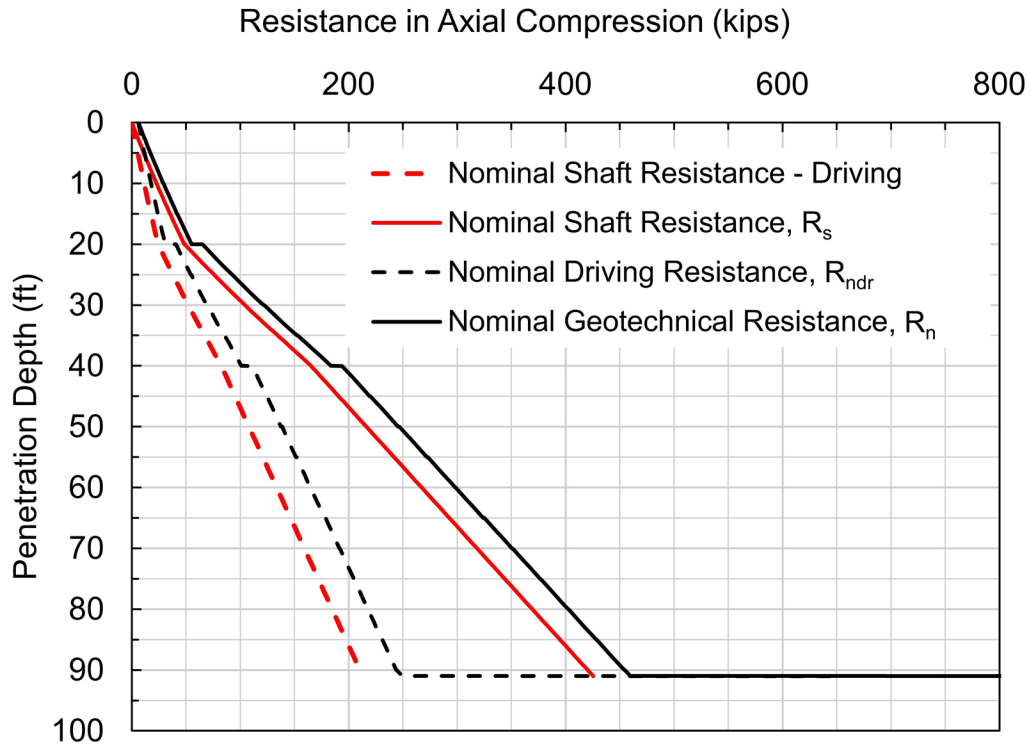


Figure D-98 Comparison of nominal driving resistance and nominal geotechnical resistance in axial compression for HP 12x74 at the South Abutment.

Table D-103 Nominal Shaft, Nominal Toe and Nominal Driving Resistance
for HP 12x74

Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Driving Resistance (kips)	Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Driving Resistance (kips)
0.01	0.01	6.09	6.1	32	58.11	17.81	75.92
1	1.1	6.09	7.19	33	61.12	17.81	78.93
2	2.2	6.09	8.3	34	64.18	17.81	81.99
3	3.3	6.09	9.41	34.99	67.25	17.81	85.05
4	4.4	6.09	10.51	35.01	67.31	18.45	85.76
4.99	5.52	6.09	11.61	36	70.35	18.45	88.8
5.01	5.54	6.27	11.81	37	73.41	18.45	91.87
6	6.67	6.27	12.95	38	76.48	18.45	94.93
7	7.84	6.27	14.11	39	79.55	18.45	98
8	9.01	6.27	15.29	39.99	82.59	18.45	101.04
9	10.21	6.27	16.48	40.01	82.64	28.69	111.34
9.99	11.4	6.27	17.68	41	85.17	28.69	113.86
10.01	11.43	6.46	17.89	42	87.72	28.69	116.41
11	12.64	6.46	19.1	43	90.27	28.69	118.96
12	13.87	6.46	20.33	44	92.82	28.69	121.52
13	15.13	6.46	21.59	44.99	95.35	28.69	124.04
14	16.4	6.46	22.85	45.01	95.4	29.43	124.83
14.99	17.67	6.46	24.13	46	97.93	29.43	127.36
15.01	17.69	6.64	24.34	47	100.48	29.43	129.91
16	18.98	6.64	25.62	48	103.03	29.43	132.46
17	20.29	6.64	26.93	49	105.58	29.43	135.01
18	21.62	6.64	28.26	49.99	108.11	29.43	137.54
19	22.97	6.64	29.61	50.01	108.16	30.45	138.6
19.99	24.32	6.64	30.96	51	110.68	30.45	141.13
20.01	24.36	16.51	40.87	52	113.23	30.45	143.68
21	26.98	16.51	43.5	53	115.79	30.45	146.23
22	29.67	16.51	46.18	54	118.34	30.45	148.78
23	32.4	16.51	48.91	54.99	120.86	30.45	151.31
24	35.16	16.51	51.68	55.01	120.91	31.00	151.91
24.99	37.94	16.51	54.46	56	123.44	31.00	154.44
25.01	38.0	16.88	54.88	57	125.99	31.00	156.99
26	40.74	16.88	57.63	58	128.54	31.00	159.54
27	43.55	16.88	60.44	59	131.1	31.00	162.1
28	46.4	16.88	63.29	59.99	133.62	31.00	164.62
29	49.29	16.88	66.18	60.01	133.67	31.28	164.95
29.99	52.19	16.88	69.08	61	136.2	31.28	167.48
30.01	52.25	17.81	70.06	62	138.75	31.28	170.03
31	55.15	17.81	72.95	63	141.3	31.28	172.58

Table D-103 Nominal Shaft, Nominal Toe and Nominal Driving Resistance
for HP 12x74 (continued)

Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Driving Resistance (kips)	Depth (feet)	Nominal Shaft Resistance (kips)	Nominal Toe Resistance (kips)	Nominal Driving Resistance (kips)
64	143.85	31.28	175.13	78	179.57	33.03	212.6
64.99	146.38	31.28	177.66	79	182.13	33.03	215.15
65.01	146.43	32.29	178.72	79.99	184.65	33.03	217.68
66	148.96	32.29	181.25	80.01	184.7	33.21	217.92
67	151.51	32.29	183.8	81	187.23	33.21	220.44
68	154.06	32.29	186.35	82	189.78	33.21	222.99
69	156.61	32.29	188.9	83	192.33	33.21	225.55
69.99	159.14	32.29	191.43	84	194.88	33.21	228.1
70.01	159.19	32.75	191.94	84.99	197.41	33.21	230.62
71	161.71	32.75	194.47	85.01	197.46	33.68	231.14
72	164.26	32.75	197.02	86	199.99	33.68	233.66
73	166.82	32.75	199.57	87	202.54	33.68	236.21
74	169.37	32.75	202.12	88	205.09	33.68	238.76
74.99	171.89	32.75	204.65	89	207.64	33.68	241.32
75.01	171.94	33.03	204.97	90	210.17	33.68	243.84
76	174.47	33.03	207.5	91	210.24	33.68	243.92
77	177.02	33.03	210.05	91.25	210.24	1527.00	1737.24

Graphical outputs of the preliminary drivability analyses are shown in Figure D-99. The nominal driving resistance, the blow count or pile penetration resistance, and the compression driving stress are presented versus pile penetration depth for each of the five candidate pile sections. As previously noted, the recommended blow count limit is 120 blows per foot (10 blows per inch), and the recommended driving stress limit is 45 ksi. A circular reference marker is indicated on the blow count versus depth plot highlighting the depth where the blow count first exceeds 120 blows per foot. This marker is also shown at the same depth on the nominal driving resistance versus depth plot indicating the nominal driving resistance achieved when practical refusal driving conditions are encountered with the selected hammer in the modeled driving conditions. Similarly, the marker is shown at the same depth on the compression driving stress versus depth plot indicating the compression driving stress when practical refusal driving conditions are encountered.

In Figure D-99, preliminary drivability results illustrate all candidate pile sections can be driven to the hard rock layer without reaching the blow count limit of 120 blows per foot. Compression driving stresses are also well below the driving stress limit of 45 ksi prior to reaching bedrock.

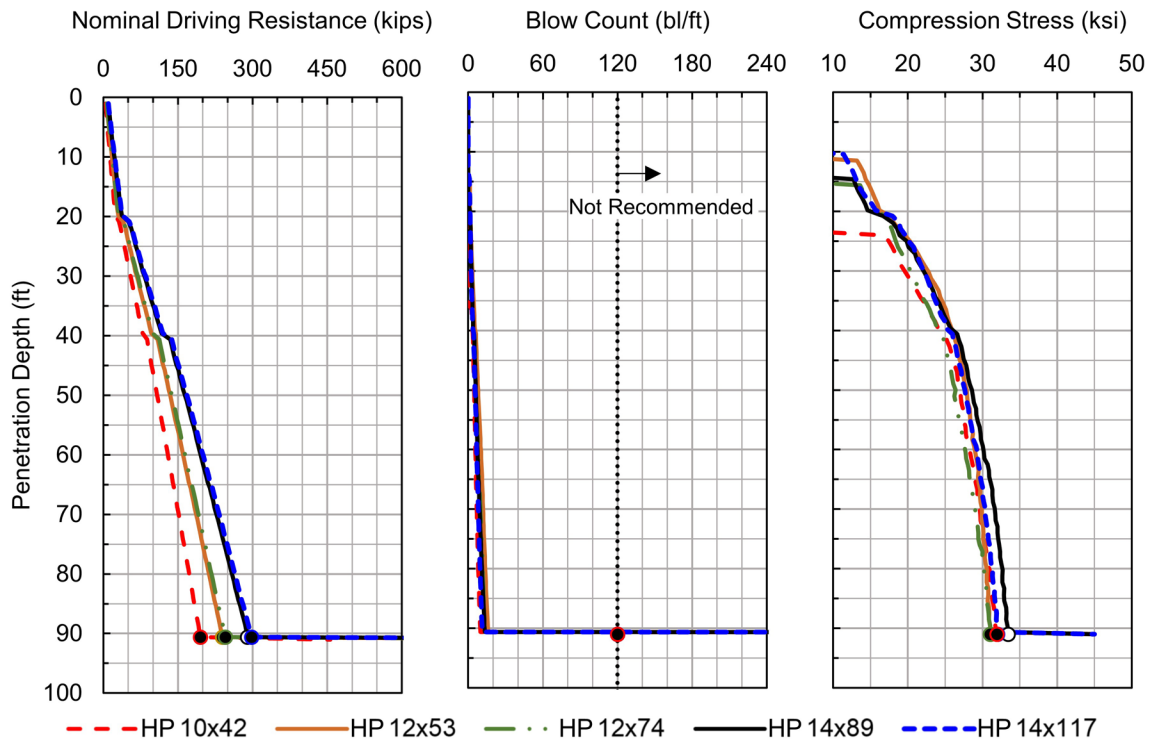


Figure D-99 Preliminary drivability results for five candidate H-pile sections at the South Abutment.

For piles driven to hard rock, the driving criteria should be established to control compression stresses and prevent pile toe damage (e.g., limit the number of blows at refusal driving conditions). A summary of the preliminary drivability results is presented in Table D-104. The anticipated nominal resistance in this table is the expected resistance after soil setup that can be mobilized by the driving system at 10 blows per inch. Since the piles terminate on hard rock, a higher geotechnical nominal resistance, up to the structural resistance of the pile, is actually available.

Once the estimated and/or minimum pile toe elevations are determined in Block 12 through Block 15 of the design process, the drivability results should be reviewed to confirm that the candidate pile section can be driven to the required nominal driving resistance, at the estimated or required pile penetration depth, at a reasonable blow count, and within driving stress limits. Upon determination of the estimated toe elevation, these plots should be reviewed to confirm that the pile can be driven to the estimated toe elevation at reasonable blow counts and stresses.

Table D-104 Summary of Preliminary Drivability Results at South Abutment

Pile Section	Pile Hammer	Fuel Setting	Pile Penetration Depth at Practical Refusal Limit (feet)	Nominal Driving Resistance at Practical Refusal Limit (kips)	Anticipated Nominal Resistance at Depth of Practical Refusal (kips)	Penetration Depth Exceeding Compression Driving Stress Limit (feet)	Maximum Compression Driving Stress (ksi)
HP 10x42	D25-52	4	91.1	362	536	> 91.1	34.1
HP 12x53	D30-52	4	91.1	446	649	> 91.1	34.6
HP 12x74	D36-52	4	91.1	590	806	> 91.1	36.7
HP 14x89	D46-52	4	91.1	729	975	> 91.1	39.5
HP 14x117	D50-52	4	91.1	896	1143	> 91.1	39.2

D.35 Block 11: South Abutment – Estimate Preliminary Number of Piles, Preliminary Pile Group Size, and Resolve Individual Pile Loads for All Limit States

The structural engineer has provided the anticipated loads for the controlling limit states at the South Abutment. These limit state loads are restated in Table D-105. The Strength I limit state loads are used to evaluate geotechnical resistance in axial compression and tension, as well as for lateral loading. Service I limit state loads were also provided by the structural engineer without live loads. The Service I without live load (LL) includes only unfactored permanent loads such as the superstructure and wearing surface, pile cap and stem, utilities, and vertical earth pressure among others. The Service I without live load should be used for evaluating vertical deformation. There are no loads in the transverse direction at this abutment.

Table D-105 Limit State Loads on South Abutment

Limit State	Q (kips)	V _{uy} (kips)	M _{uy} (k-ft)
Strength I	-2815	946	6732
Service I	-2082	629	3931
Service I, without live load	-1783	546	3024

Based on past experience, the agency generally utilizes 2 rows of piles at abutments with a minimum center to center pile spacing of at least 3 pile diameters. Three potential pile group configurations are therefore being considered: 2 rows of 9 piles, 2 rows of 11 piles, and 2 rows of 13 piles. These group configurations are identified as Group Configuration 1, 2, and 3, respectively in Table D-106. Because of site constraints, the pile cap length is limited to 43 feet. Furthermore, the distance from the center of any exterior pile to the pile cap edge must be at least 1.25 feet in both the transverse (x) and longitudinal (y) direction.

Table D-106 Potential Pile Group Configurations

Group Configuration	Piles per Row	Total Number of Piles	S_{bx} [*] (feet)	Total Footing Length (feet)	S_{by} [*] (feet)	Total Footing Width (feet)
1	9	18	5.0	42.5	4.0	6.5
2	11	22	4.0	42.5	4.0	6.5
3	13	26	3.0	38.5	4.0	6.5

Note: * - S_{bx} and S_{by} are illustrated in Figure D-100

The following calculation is for the Group Configuration 3 and all applicable loads. For this alternative, 13 piles per row are used in two separate rows, thus the transverse pile spacing, S_{bx} , is 5'-0" and the total footing length is 38'-6". The longitudinal pile spacing, S_{by} , is 4'-0". Figure D-100 shows the layout for the Group Configuration 3 pile cap.

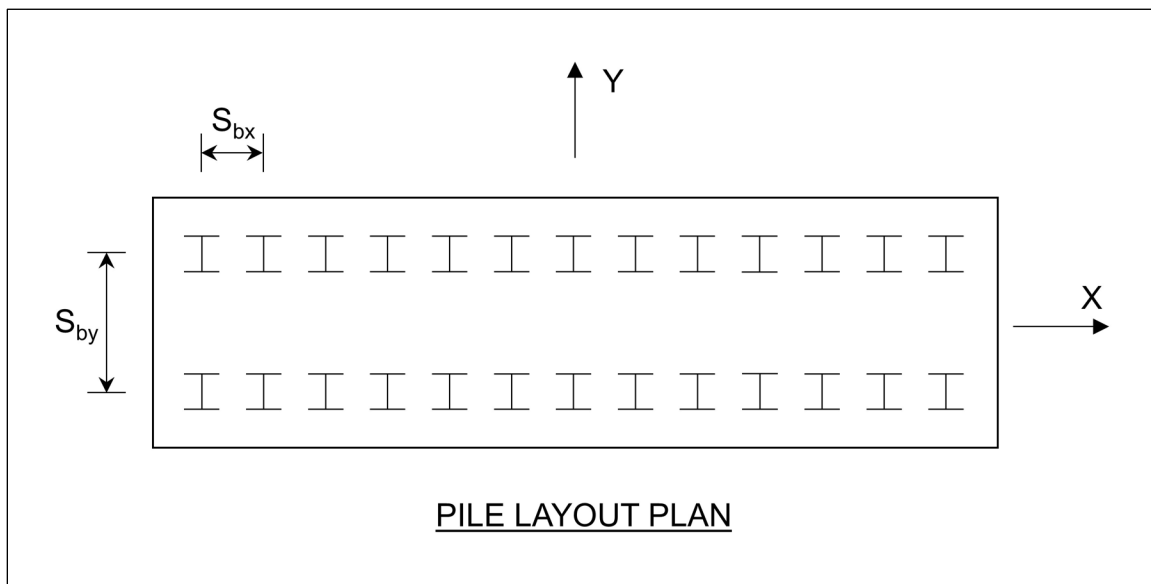


Figure D-100 Group Configuration 3 pile cap plan view.

For the established limit state loads and the trial pile group configuration in Figure D-100, reactions for both the front and the back rows of piles were determined. Compression loads are taken as positive. The maximum factored load applied to each pile was subsequently calculated by dividing the reactions by the number of piles. Figure D-101 shows the free body diagram for determining the factored load for both the front and the back row.

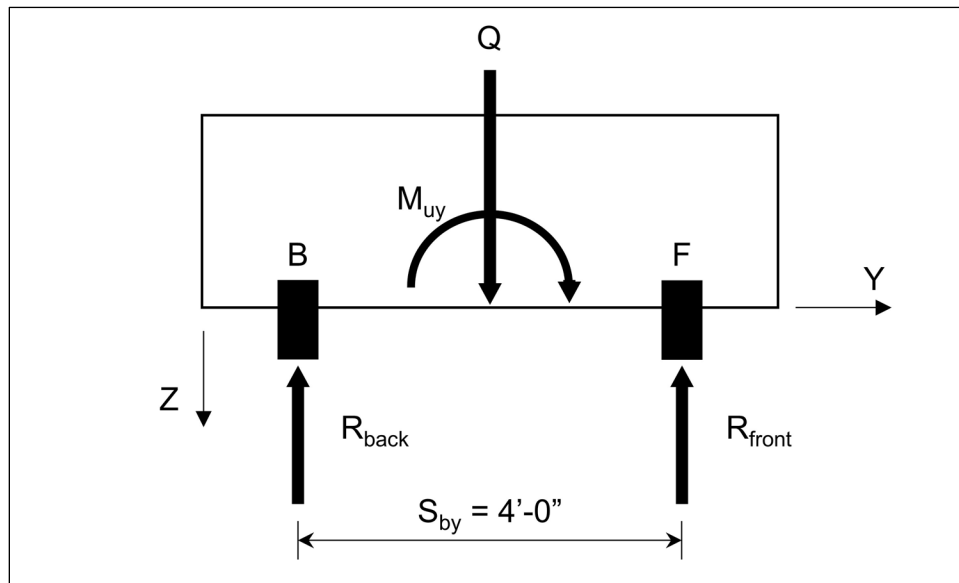


Figure D-101 Elevation view of cap free body diagram.

Table D-107 summarizes the limit state loads and the front and back row reactions. Strength I loads, Service I loads and the Service I loads without live load (LL) are provided. The Service I load without live load is used for settlement calculations.

Table D-107 Factored Loads and Row Reactions

Limit State	Q (kips)	M_{uy} (k-ft)	R_{front} (kips)	R_{back} (kips)
Strength I	-2815	6732	3091	-276
Service I	-2082	3931	2037	46
Service I, without LL	-1783	3024	1648	136

Table D-108 presents the maximum factored load per pile based upon the number of piles in each group configuration. For example, considering Group Configuration 3, the Service I, without live load, front-row reaction of 1648 kips is divided by 13 piles to yield a front-row factored load per pile equal to 127 kips. Table D-108 also provides the factored load per pile for other limit states and group configurations.

Table D-108 Factored Load Per Pile for Alternative Pile Group Configurations

Group Configuration	Strength I, Q (kips)	Strength I, Q (tension) (kips)	Strength I, V_{uy} (kips)	Service I, without LL Q (kips)
1	344	-31	53	183
2	281	-25	43	150
3	238	-21	37	127

D.36 Block 12: South Abutment – Estimate Pile Penetration Depth for Maximum Axial Compression Loads. Check Group Efficiency in Axial Compression

The estimated minimum pile penetration depth necessary to obtain a factored geotechnical resistance that is equal or greater than the maximum factored load per pile is now determined. Note that the factored geotechnical resistance in axial compression and the resulting pile penetration depth is dependent upon the resistance determination method. Therefore, the influence of the field resistance determination method on the design needs to be evaluated at this point in the design process and some resistance determination methods may be eliminated from further design consideration.

Since setup in the cohesive soils will be a significant component of the nominal resistance, it is determined that the nominal geotechnical resistance will be substantiated by dynamic testing 2% of the piles. Figure D-89 illustrates the nominal geotechnical resistance versus penetration depth for the 5 candidate pile sections based on this resistance determination method. From this plot, the estimated penetration depth for a factored geotechnical resistance of 238 kips in axial compression for the Group Configuration 3 ranges from 62 feet for the HP 14x117 H-pile section to 91 feet for the HP 10x42 H-pile section. The estimated pile penetration depth needed for the factored load associated with each group configuration is provided for all candidate pile sections in Table D-109.

Table D-109 Estimated Pile Penetration Depths for the Factored Geotechnical Resistance in Axial Compression at the Strength I Limit State

Group Configuration	Factored Load per Pile (kips)	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
1	344	91	91	91	90	88
2	281	91	90	86	74	72
3	238	91	74	73	63	62

As described in Section 7.2.2.2, the nominal geotechnical resistance of a pile group in cohesive soil can be taken as the sum of the individual pile nominal geotechnical resistances. In a similar manner, the factored geotechnical resistance of the pile group in cohesive soil is taken as the sum of the individual pile factored geotechnical resistances. This is recommended so long as 1) the pile cap is in firm contact with the ground and 2) the piles are not installed at a pile spacing of less than 3 times the pile diameter or 3 feet. Since both conditions are met at the South Abutment, by inspection, the nominal and factored group resistances are satisfactory.

As will be discussed later in Block 15, in order to satisfy tolerable deformations, the piles at the South Abutment will be driven to bedrock. Therefore, piles at the South Abutment are not subject to block failure as outlined in Section 7.2.2.3 of Chapter 7.

D.37 Block 13: South Abutment – Establish Minimum Pile Penetration Depth for Axial Tension Loads. If Conditions Warrant, Modify Design and Return to Block 10

The factored geotechnical resistance in axial tension must also be evaluated as the back row of piles will be loaded in tension (Table D-107). In this case, the minimum required factored geotechnical resistance in axial tension is established using the Strength I limit state and is determined following the procedure outlined in Section 7.2.3.2. The analysis presented in this appendix slightly differs from the procedure outlined in Chapter 7 in that only a single row of piles is providing the tension resistance rather than the entire pile group. Two analyses are performed; one that considers the factored shaft resistances from individual piles, and one that considers the weight of a soil block acting with the piles. The lesser tension resistance determined from either method controls the design.

As noted in Table D-107, the Strength I limit state tension load on the back row of piles for the Group Configuration 3 is 276 kips. Therefore, the minimum factored geotechnical resistance in axial tension required from an individual pile is this factored load divided by the 13 piles in the rear row or 21 kips. In a similar manner, the minimum factored geotechnical resistance required from an individual pile in axial tension is 31 kips for Group Configuration 1, and 25 kips for Group Configuration 2.

As noted earlier, dynamic testing with signal matching will be used as the resistance determination method in the field. Therefore, the AASHTO recommended resistance factor, ϕ_{up} , is 0.5 (Table 7-2 of Chapter 7). Figure D-95 provides plots of the factored geotechnical resistance in axial tension versus depth for all of the candidate pile types based on this resistance determination method. For each candidate section, Figure D-95 should be entered on the x-axis at the required factored axial tension resistance to determine the corresponding pile penetration depth.

Following this procedure, the estimated pile penetration depth to achieve the factored geotechnical resistance in axial tension for each candidate pile section and group configuration is summarized in Table D-110.

Table D-110 Estimated Minimum Pile Penetration Depth Required for Factored Geotechnical Resistance in Axial Tension at the Strength I Limit State

Group Configuration	Required Factored Resistance in Axial Tension (kips)	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
1	31	25	23	23	21	21
2	25	23	21	21	19	18
3	21	21	19	18	16	16

Next, the tension resistance of the pile row when considered as a soil block is calculated. For Group Configuration 3, the required factored tension resistance of the back row remains 276 kips (Table D-107) and the resistance is derived from the weight of soil as a block and an average of the soil shear strength. Equation 7-39 is used to determine the required nominal group resistance in axial tension.

$$R_r = \phi R_n = \phi_{ug} R_{ug} \quad [\text{Eq. 7-39}]$$

Where:

- R_n = nominal resistance (kips).
- ϕ_{ug} = resistance factor for tension per Table 7-1, 0.50.
- R_{ug} = nominal uplift resistance of the pile group (kips).

$$R_{ug} = \frac{R_r}{\phi_{ug}} \quad \text{Eq. [7-39 modified]}$$

$$R_{ug} = \frac{(276 \text{ kips})}{(0.5)}$$

$$R_{ug} = 552 \text{ kips}$$

The minimum required nominal uplift resistance is 552 kips.

Equation 7-41 is then used to determine the minimum embedded length required to satisfy the nominal group tension resistance for soil acting as a block. The length of the pile group will change depending upon the group configuration. For Group Configuration 1 and 2, the pile group length is 41 feet from exterior pile edge to exterior pile edge assuming a 12 inch pile width/diameter. For Group Configuration 3, the pile group length is 37 feet using the same pile dimension. The effect of a smaller pile size on the pile group length is negligible (e.g. for HP 10x42 section, the pile group length is 40.83 feet).

$$R_{ug} = 2 D (B + Z) s_u + W_g \quad [\text{Eq. 7-41}]$$

- D = embedded length of piles (feet).
- B = width of pile group, assume 1 foot (1 row in tension).
- Z = length of pile group, 37 feet (Group Configuration 3).
- s_u = weighted average of the undrained shear strength over the depth of pile embedment along the pile group perimeter (ksf) (Figure D-83).
- W_g = effective weight of the pile/soil block including pile cap weight (kips).

The calculation is performed for the HP 12x74 pile section in Group Configuration 3. The back row contains 13 piles, and the group length is 37 feet. Figure D-102 provides a visual representation of the soil volume used to determine tension resistance. The block of soil being evaluated is only as wide as, and lies between, the piles. Therefore it is not visible in the transverse elevation view in Figure D-102.

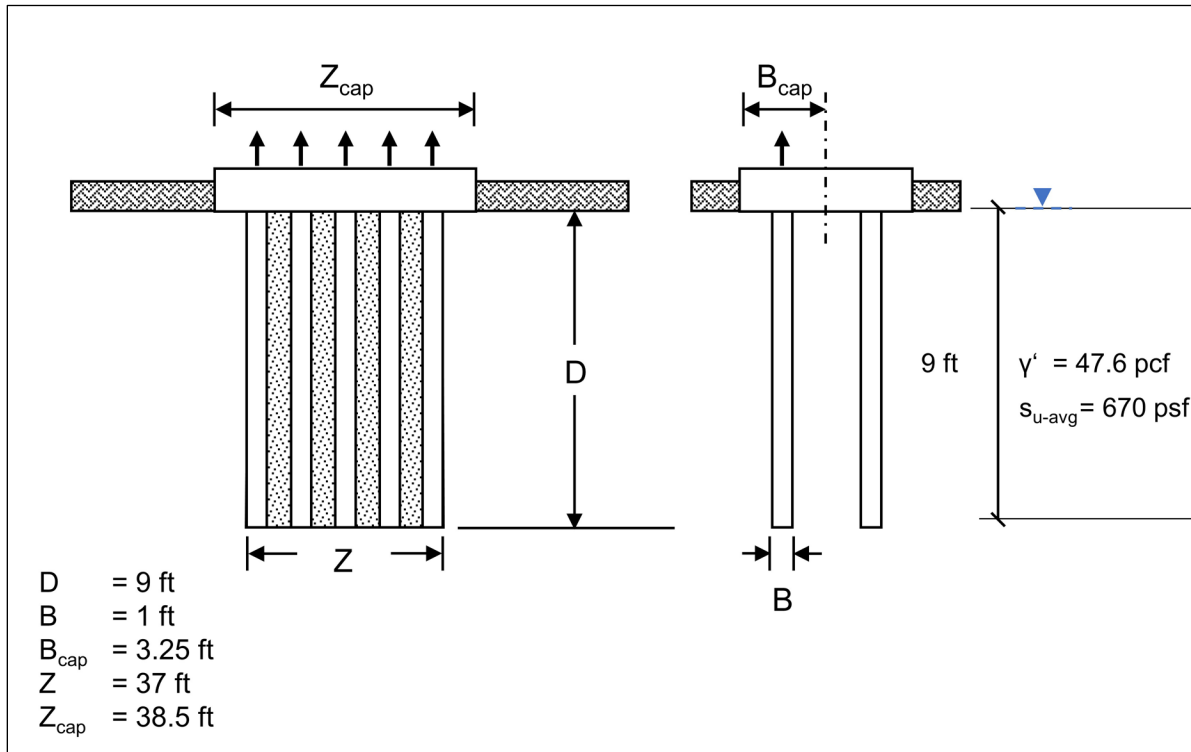


Figure D-102 Tension resistance of pile group in cohesive soil.

Several iterations were performed to incrementally increase the pile length and evaluate the nominal uplift resistance of the pile group. The minimum required embedded length of piles was determined to be 9 feet, and therefore, that calculation is presented below.

Calculate the weight of piles:

$D =$ embedded length of piles, 9 feet.

$W/ft =$ weight per linear foot of HP 12x74 pile section, 0.074 kip/ft.

$$W_{piles} = (\text{Weight/ linear foot}) * D * \text{Number of Piles}$$

$$W_{piles} = \left(0.074 \frac{\text{kip}}{\text{ft}}\right) * (9 \text{ feet}) * (13 \text{ piles})$$

$$W_{piles} = 8.66 \text{ kips}$$

Calculate the weight of soil:

- D = embedded length of piles, 9 feet.
- B = width of pile group, assume 1 foot (1 row in tension).
- Z = length of pile group, 37 feet (Group Configuration 3).
- γ'_{soil} = effective unit weight of soil, 47.6 pcf (Figure D-83).

$$W_{soil} = D * B * Z * \gamma'_{soil}$$

$$W_{soil} = (9 \text{ feet}) * (1 \text{ feet}) * (37 \text{ feet}) * (47.6 \text{ pcf}) * \left(\frac{1 \text{ kip}}{1000 \text{ lbs}}\right)$$

$$W_{soil} = 15.85 \text{ kips}$$

Calculate weight of the pile cap, conservatively taken as the tributary area over piles in tension:

- B_{cap} = width of pile cap, half of cap width 6.5 feet / 2 = 3.25 ft (Table D-106).
- Z_{cap} = length of pile group, 38.5 feet (Table D-106).
- t_{cap} = thickness of pile cap, preliminary estimate, 4.2 feet.
- $\gamma_{concrete}$ = unit weight of reinforced concrete, 150 pcf.

$$W_{cap} = B_{cap} * Z_{cap} * t_{cap} * \gamma_{concrete}$$

$$W_{cap} = (3.25 \text{ feet}) * (38.5 \text{ feet}) * (4.2 \text{ feet}) * (150 \text{ pcf}) * \left(\frac{1 \text{ kip}}{1000 \text{ lbs}}\right)$$

$$W_{cap} = 78.83 \text{ kips}$$

Calculate the effective weight of the pile/soil block including pile cap weight:

$$W_g = W_{piles} + W_{soil} + W_{cap}$$

$$W_g = (8.66 \text{ kips}) + (15.85 \text{ kips}) + (78.83 \text{ kips})$$

$$W_g = 103.3 \text{ kips}$$

Equation 7-41 is next used to calculate the group tension resistance for soil acting as a block. A weighted average shear strength value of 0.67 ksf was used for the first 10 feet below the footing in the silty clay soil layer (Figure D-83).

- D = embedded length of piles, 9 feet.
- B = width of pile group, assume 1 foot (1 row in tension).
- Z = length of pile group, 37 feet (Group Configuration 3).
- s_u = weighted average undrained shear strength, 0.67 ksf (Figure D-83).
- W_g = effective weight of the block, 103.3 kips.

$$R_{ug} = 2 D (B + Z)s_u + W_g \quad [\text{Eq. 7-41}]$$

$$R_{ug(9\ ft)} = 2 * (9\ feet) * ((1\ foot) + (37\ feet)) * (0.67\ ksf) + (103.3\ kips)$$

$$R_{ug(9\ ft)} = 561.6\ kips$$

Therefore:

$$R_{ug(9\ ft)} > R_{ug}$$

$$561.6\ kips > 552\ kips$$

The required soil block weight is achieved at approximately 9 feet of pile penetration. Accordingly, any additional pile penetration would satisfy this requirement. For a pile group length of 41 feet (i.e., Group Configuration 1 and 2), the minimum pile penetration depth to achieve a soil block weight in excess of the required 552 kips is also 9 feet.

As noted earlier, two analyses are performed to determine the minimum pile penetration depth for nominal tension resistance. The first one considers the shaft resistances from individual piles, and the second one considers the weight of a soil block acting with the piles. The required minimum pile penetration depth to achieve geotechnical resistance in axial tension is the greater depth of the above calculated resistances. The estimated minimum penetration depth required for the factored geotechnical resistance in axial tension was summarized in Table D-110. For the HP 12x74 pile section in Group Configuration 3, the minimum pile penetration depth is 18 feet for the factored tension load of 21 kips.

$$D_{(individual)} = \text{minimum pile penetration depth based on sum of individual pile resistance} = 18\ \text{feet (Table D-110)}.$$

$D_{(block)}$ = minimum pile penetration depth based on weight of soil block
 = 9 feet. (calculated above).

$$D_{(individual)} > D_{(block)}$$

$$18 \text{ feet} > 9 \text{ feet}$$

Therefore, the minimum penetration depth $D_{(min-tension)}$ is as follows:

$$D_{(min-tension)} = 18 \text{ feet}$$

In a similar manner, this check was performed for all candidate pile sections and group configurations. The resulting established minimum required pile penetration depth to meet axial tension requirements for each candidate pile section within the specified group configuration is presented in Table D-111.

Table D-111 Established Minimum Pile Penetration Depth Required for Factored Geotechnical Resistance in Axial Tension at South Abutment

Group Configuration	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
1	25	23	23	21	21
2	23	21	21	19	18
3	21	19	18	16	16

D.38 Block 14: South Abutment – Establish Minimum Pile Penetration Depth for Lateral Loads. Determine p-y Models, Required Geomaterial Parameters, and Perform Lateral Load Analysis. If Conditions Warrant, Modify Design and Return to Block 10

Next, lateral analyses are performed to establish the required minimum pile penetration depth for lateral loading and to evaluate pile deflection and structural resistance for the applied limit state loads. The minimum required pile penetration depth required to satisfy the nominal geotechnical resistance requirements in axial tension was determined to be 18 feet in Block 13. A deeper minimum required pile penetration depth for lateral loading can result based on the combination of factored lateral loads and structural resistances, or deflection limits. Excessive deflections and moments develop at relatively short pile lengths, where a depth to fixity is not

achieved. Furthermore, the structural resistance of pile sections must be evaluated based upon the axial, lateral and moment loads. Factored structural resistances were presented in Table D-95 while a lateral deformation limit of 1 inch was established as a global performance requirement in Block 1 and confirmed in Block 4 as the design progressed.

The soil profile at the South Abutment was presented in Figure D-83. For lateral load analyses, appropriate p-y models must be selected for each soil layer. The input parameters necessary for lateral load analysis using the LPILE computer program are included in the South Abutment soil profile.

As discussed in Section 7.3.7.6, p-multipliers are applied to the p-y curves to model pile group behavior. The p-multipliers depend on the center to center pile spacing within the pile group. For all group configurations at the South Abutment, the pile spacing in the longitudinal direction is 4 feet. Therefore, per Section 7.3.7.6 and AASHTO (2014) design specifications, interpolation was used to determine p-multipliers for a pile spacing of 4b. In this case, the front row p-multiplier is 0.90, while the second row is 0.625.

Cyclic loading was performed for both rows using LPILE's Load Type 2 option, which uses shear and slope to model a fixed head condition. Considering loading conditions at this abutment, lateral analyses in the longitudinal (y-direction) were performed about the pile section's strong axis. Figure D-100 shows the pile orientation within the trial pile cap design.

The following example is presented for the HP 12x74 pile section using a range of factored axial and lateral loads for Group Configuration 3. Tables D-112 and D-113 provide LPILE output summaries for both rows at the pile head considering a pile penetration for lateral loading of 30 feet. The pile head is assumed to terminate at the ground surface (i.e. no stickup).

Table D-112 LPILE Summary Output at Pile Head for Front Row, $p_m=0.90$

Load Case	Load Type No.	Pile-Head Condition 1 V (kips)	Pile-Head Condition 2 S (rad)	Axial Load (kips)	Pile-Head Deflection (inches)	Maximum Moment in Pile (kip-ft)	Maximum Shear in Pile (kips)	Pile-Head Rotation (radians)
1	2	0	0	238	0.000	0.0	0	0
2	2	15	0	238	0.116	-56.6	15	0
3	2	30	0	238	0.451	-141.7	30	0
4	2	35	0	238	0.611	-173.8	35	0
5	2	36	0	238	0.646	-180.4	36	0
6	2	37	0	238	0.682	-187.1	37	0
7	2	38	0	238	0.719	-193.9	38	0
8	2	40	0	238	0.796	-207.6	40	0
9	2	42	0	238	0.876	-221.5	42	0
10	2	50	0	238	1.344	-292.1	50	0

Table D-113 LPILE Summary Output at Pile Head for Second Row, $p_m=0.625$

Load Case	Load Type No.	Pile-Head Condition 1 V (kips)	Pile-Head Condition 2 S (rad)	Axial Load (kips)	Pile-Head Deflection (inches)	Maximum Moment in Pile (kip-ft)	Maximum Shear in Pile (kips)	Pile-Head Rotation (radians)
1	2	0	0	238	0.000	0.0	0	0
2	2	15	0	238	0.196	-67.5	15	0
3	2	30	0	238	0.769	-169.8	30	0
4	2	35	0	238	1.060	-211.2	35	0
5	2	36	0	238	1.138	-221.2	36	0
6	2	37	0	238	1.223	-231.8	37	0
7	2	38	0	238	1.317	-242.9	38	0
8	2	40	0	238	1.531	-266.6	40	0
9	2	42	0	238	1.793	-292.7	42	0

The pile group deflection can be estimated from the above LPILE's deflection results for the front and second rows. The factored load versus pile head deflection for each row is plotted in Figure D-103 along with the group average. The average lateral load per pile for a given group deflection is shown. A step by step discussion of this procedure is provided in Section 7.3.7.6.

The rigid cap method assumes piles move together, and therefore experience the same shear and lateral load. Accordingly, at the resulting factored lateral load per pile, V_{uy} , of 37 kips (Table D-108), the estimated lateral group deflection at the pile head is determined as 0.96 inches. This lateral deflection is less than the 1 inch tolerance based upon project specific requirements.

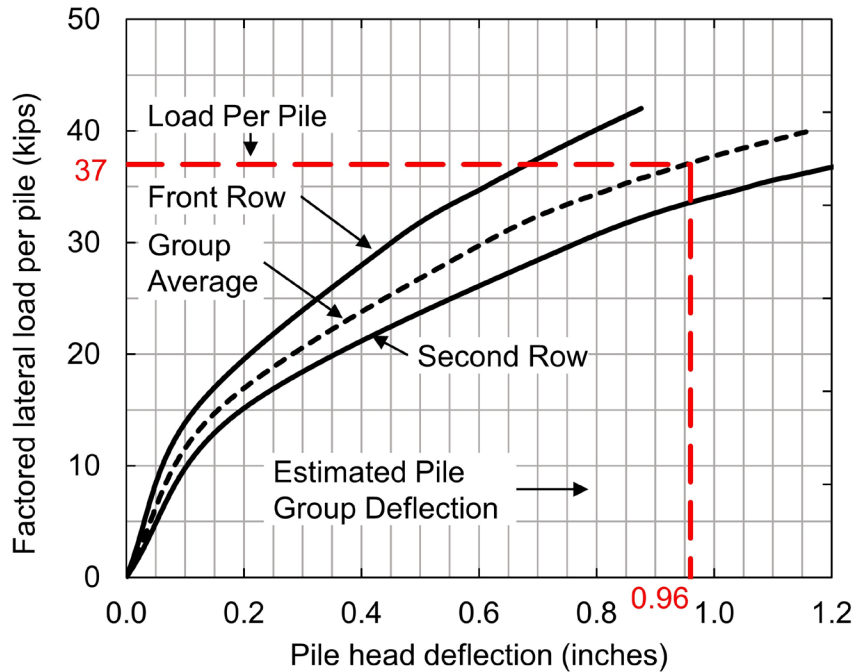


Figure D-103 Factored load versus deflection for HP 12x74 at South Abutment.

In addition to the lateral deflection limit, stresses from the resulting bending moment and shear must be evaluated to check that the pile section does not fail structurally. Using the LPILE tabular results, Figure D-104 plots the front row bending moment versus depth for a deflection of 0.96 inches.

Figure D-105 plots the maximum bending moment versus pile head deflection for both the front and back rows, however only the maximum bending moment for the front row is used as a “worst case” evaluation of the structural resistance in combined axial compression and flexure. As illustrated in Figure D-105 for the front row piles, at the estimated pile head deflection of 0.96 inches, the maximum bending moment, M_{uy} , is 235 kip-ft.

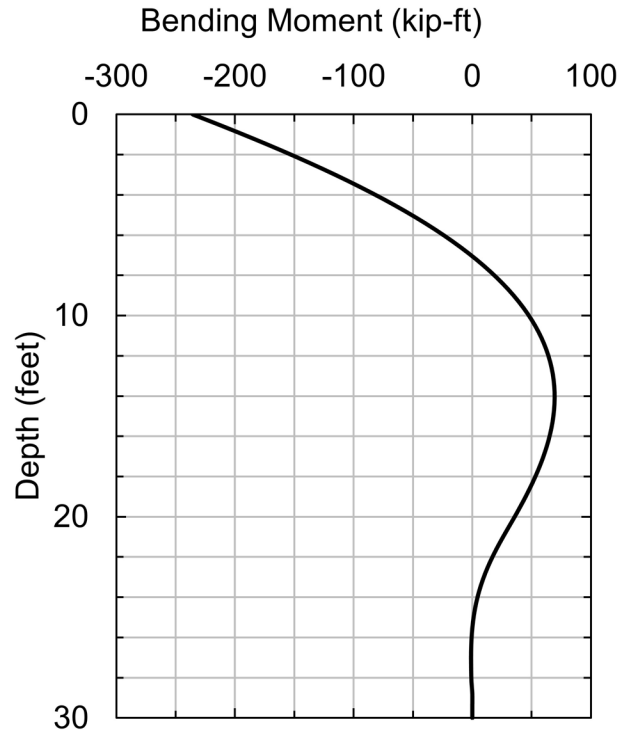


Figure D-104 Front row bending moment versus depth.

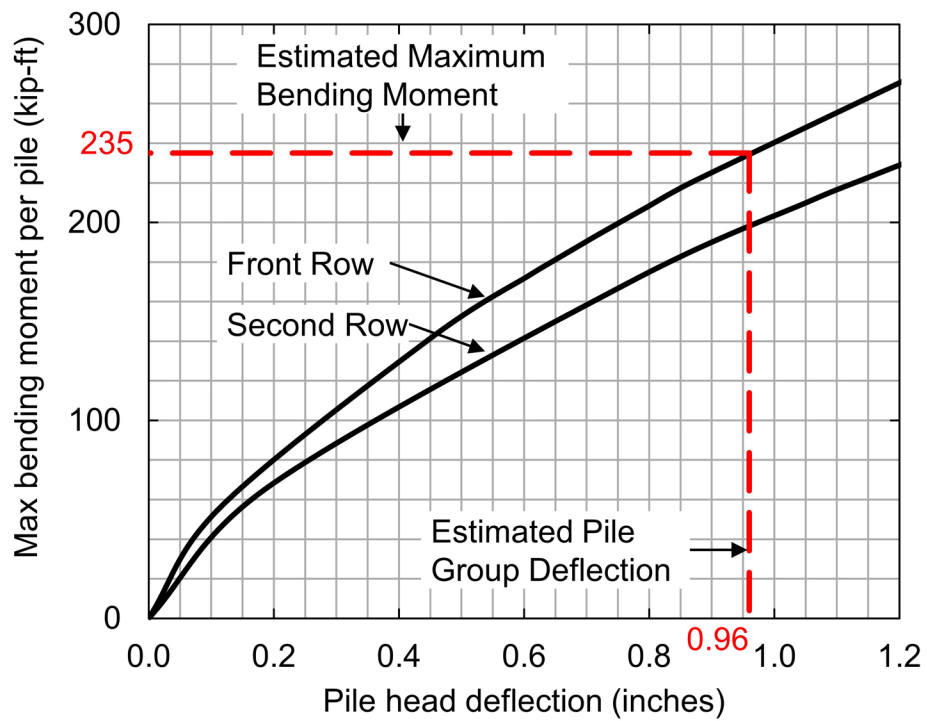


Figure D-105 Bending moment versus deflection for HP 12x74 at South Abutment.

The interaction shown in Equation 8-58 must be satisfied for the factored axial compression load and moment in the pile. Using results of the lateral analysis, the factored structural resistance was evaluated at the pile head using the factored axial compression load and maximum bending moment (determined using factored loads). The factored structural resistances were determined as shown in Table D-95.

Equation 8-58 must be satisfied for the pile section to be acceptable.

- P_u = factored axial load, 238 kips (Table D-108).
- P_r = factored axial resistance, 762 kips (Table D-95).
- M_{ux} = factored moment about x-axis, 0 kip-ft (Block 11).
- M_{rx} = factored flexural resistance about x-axis, 118 kip-ft (Table D-95).
- M_{uy} = factored moment about y-axis, 235 kip-ft (Figure D-105).
- M_{ry} = factored flexural resistance about y-axis, 433 kip-ft (Table D-95).

$$\frac{P_u}{P_r} + \frac{8.0}{9.0} \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \right) \leq 1.0 \quad [\text{Eq. 8-58}]$$

$$\frac{238 \text{ kips}}{762 \text{ kips}} + \frac{8.0}{9.0} \left(\frac{0}{118 \text{ kip-ft}} + \frac{235 \text{ kip-ft}}{433 \text{ kip-ft}} \right) \leq 1.0$$

$$0.79 \leq 1.0$$

The maximum shear from factored lateral loading was then compared to the factored shear resistance from Table D-95. Based on the factored loads, the factored shear resistance is acceptable.

- V_r = factored shear resistance, 214 kips (HP 12x74, Table D-95).
- V_{uy} = factored shear load, 37 kips (Group Configuration 3, Table D-108).

$$V_{uy} < V_r$$

$$37 \text{ kips} < 214 \text{ kips}$$

The lateral analysis was also performed for each alternative pile section. The deflection and factored structural resistance were evaluated considering the factored loads for group configurations shown in Table D-108. Pile head deflection must be limited to 1 inch, and based upon the applied loads and pile section, the factored structural resistance of the pile must also satisfy the structural resistance interaction

equation presented as Equation 8-58. Pile sections satisfying both criteria were deemed acceptable, and furthermore as summarized in Table D-114, a minimum required pile penetration depth was established based on lateral load resistance considerations. The minimum penetration depth is identified as “- - -” for candidate pile sections not meeting the lateral deformation or structural resistance requirements.

Several of the larger pile sections provided sufficient stiffness to resist the applied loads, while less stiff sections did not (they failed the structural resistance check in Equation 8-58). Factored axial compression loads, in combination with moments caused by factored lateral loads, resulted in some sections’ factored structural resistance being exceeded. Although the HP 12x53 pile section satisfied structural resistance requirements considering Group Configuration 3 factored loads, the pile section did not satisfy the 1 inch pile head deflection requirement for any pile penetration depth.

Table D-114 Established Minimum Required Pile Penetration Depth for Lateral Loading at the South Abutment

Group Configuration	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
1	- - -	- - -	- - -	- - -	40
2	- - -	- - -	- - -	35	35
3	- - -	- - -	35	35	35

D.39 Block 15: South Abutment – Establish Pile Penetration Depths that Satisfy Tolerable Deformations. Estimate Group Settlement over the Minimum and Maximum Range of Pile Penetration Depths From Blocks 12 to 14 and Identify All Pile Toe Elevations Which Result in Intolerable Deformations. If Conditions Warrant, Modify Design and Return to Block 10

For the cohesive soils at the South Abutment, pile group settlement was estimated using two methods, classic consolidation theory in cohesive soils and the Janbu Tangent modulus method. Ideally, the settlement method chosen by the designer is one that has shown good correlation with observed results. The pile group settlement at the South Abutment was first calculated using classic consolidation theory with an equivalent footing per AASHTO (2014) design specifications. However, the soil conditions across the bridge substructure locations are quite variable and a settlement method that could be used at all substructure locations was also desired. Therefore, group settlement was also computed with the Janbu Tangent modulus approach using an equivalent footing placed at the neutral plane.

As established by global project performance requirements, vertical deformation (including both settlement and elastic compression) should be limited to 1.5 inches at each substructure location.

The deformation analysis must consider all loads and resulting changes in vertical effective stress. Thus the analysis begins considering construction of the new embankment and the subsequent increase in vertical effective stresses below the abutment. For simplification, the vertical effective stress increase is determined by treating the embankment surcharge as a strip load. Figure D-106 demonstrates this concept and defines symbols, while the change in vertical effective stress with depth is determined using Equation D-2 through D-4.

$$\Delta\sigma_v = \frac{q}{\pi} [\beta + \sin(\beta) * \cos(\beta + 2\delta)] \quad [\text{Eq. D-2}]$$

Where:

$$\beta = \tan^{-1} \left(\frac{x+b}{z} \right) - \tan^{-1} \left(\frac{x-b}{z} \right) \quad [\text{Eq. D-3}]$$

and

$$\delta = \tan^{-1} \left(\frac{x-b}{z} \right) \quad [\text{Eq. D-4}]$$

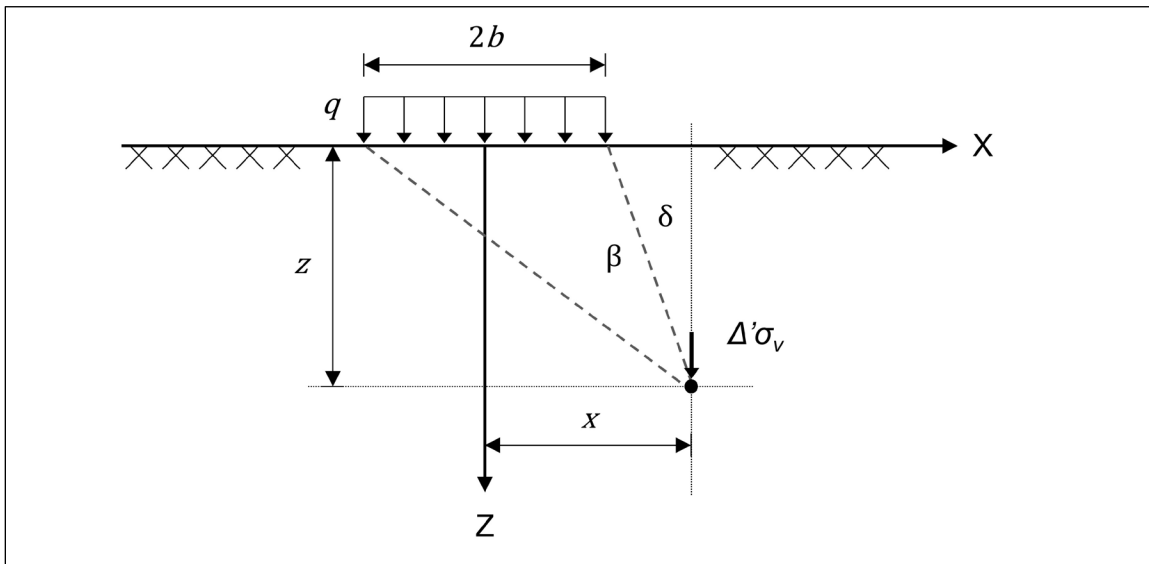


Figure D-106 Vertical effective stress increase due to strip load.

The 15 foot high embankment, with a soil unit weight 120 pcf, results in a surcharge stress at the embankment base of 1.8 ksf, and is assumed to extend 100 feet behind

the abutment. Fill directly above the footing is already included in design as a permanent vertical load, EV, and therefore the embankment surcharge is assumed to act as a strip load beginning at the footing edge. The change in vertical effective stress from the embankment surcharge, $\Delta\sigma'_{v(e)}$, is determined under the footing centerline as depicted in Figure D-107.

An example calculation is shown for a depth below footing, z , of 2.5 feet, while complete calculations were performed using a spreadsheet and are shown in Table D-115. The bottom of footing is at Elevation 305 feet. Therefore, although the value of z in the following series of analyses may vary based upon the equivalent footing or neutral plane location, the elevation is used to provide a comparison of effective stress change with depth.

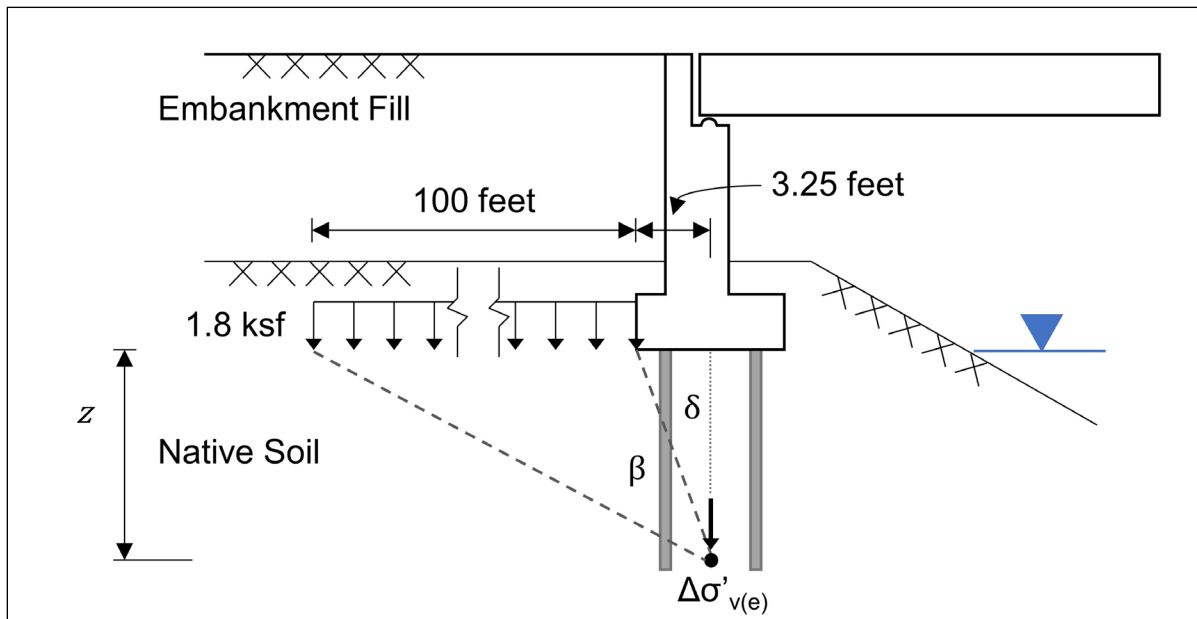


Figure D-107 Profile of vertical stress increase due to embankment surcharge.

Determine geometry of profile.

$$x = 3.25 \text{ feet} + \frac{100 \text{ feet}}{2} = 53.25 \text{ feet}$$

$$b = \frac{100 \text{ feet}}{2} = 50 \text{ feet}$$

Determine angle β .

$$\beta = \tan^{-1}\left(\frac{x+b}{z}\right) - \tan^{-1}\left(\frac{x-b}{z}\right) \quad [\text{Eq. D-3}]$$

$$\beta = \tan^{-1} \left(\frac{53.25 \text{ feet} + 50 \text{ feet}}{2.5 \text{ feet}} \right) - \tan^{-1} \left(\frac{53.25 \text{ feet} - 50 \text{ feet}}{2.5 \text{ feet}} \right)$$

$$\beta = 0.63$$

Determine angle δ .

$$\delta = \tan^{-1} \left(\frac{x-b}{z} \right) \quad [\text{Eq. D-4}]$$

$$\delta = \tan^{-1} \left(\frac{53.25 \text{ feet} - 50 \text{ feet}}{2.5 \text{ feet}} \right)$$

$$\delta = 0.92$$

Calculate the change in vertical effective stress due to embankment surcharge.

q = stress per unit length, 1.8 ksf.

$$\Delta\sigma'_{v(e)} = \frac{q}{\pi} [\beta + \sin(\beta) * \cos(\beta + 2\delta)] \quad [\text{Eq. D-2}]$$

$$\Delta\sigma'_{v(e)} = \frac{1.8 \text{ ksf}}{\pi} [0.63 + \sin(0.63) * \cos(0.63 + 2 * (0.92))]$$

$$\Delta\sigma'_{v(e)} = 0.10 \text{ ksf}$$

Table D-115 Vertical Effective Stress Increase from Embankment Surcharge

Elevation (feet)	Soil Layer	z (feet)	δ	β	$\Delta\sigma'_{v(e)}$ (ksf)
304.99	1	0.01	1.57	0.00	0.00
302.5	1	2.5	0.92	0.63	0.10
297.5	1	7.5	0.41	1.09	0.46
292.5	1	12.5	0.25	1.20	0.61
287.5	1	17.5	0.18	1.22	0.69
282.5	2	22.5	0.14	1.21	0.73
277.5	2	27.5	0.12	1.19	0.76
272.5	2	32.5	0.10	1.17	0.78
267.5	2	37.5	0.09	1.14	0.79
262.5	3	42.5	0.08	1.10	0.79
257.5	3	47.5	0.07	1.07	0.79
252.5	3	52.5	0.06	1.04	0.79
247.5	3	57.5	0.06	1.01	0.79
242.5	3	62.5	0.05	0.97	0.78
237.5	3	67.5	0.05	0.94	0.78
232.5	3	72.5	0.04	0.91	0.77
227.5	3	77.5	0.04	0.88	0.76
222.5	3	82.5	0.04	0.86	0.75
217.5	3	87.5	0.04	0.83	0.74
214.5	3	90.5	0.04	0.82	0.73

The increase in vertical effective stress with depth is relatively sustained due to the long embankment length, and, as a result, settlement will be adversely affected. Therefore, the designer should determine if the construction schedule can accommodate the time required for embankment induced settlements to occur before the start of pile driving and superstructure construction. This example calculation assumes construction cannot be delayed, and therefore, the stress increase from embankment construction and foundations loads are applied concurrently.

Compressibility properties for the three silty clay layers were determined from one dimensional consolidation tests which were performed on undisturbed samples collected near the middle of each respective soil layer. Table D-116 presents the void ratio, e_0 , overconsolidation ratio, OCR, compression index, C_c , and recompression index, C_r for the three silty clay layers. The OCR of the various soil layers were used to calculate the preconsolidation stress, σ'_p , at discrete depths.

Table D-116 Soil Properties Determined from One Dimensional Consolidation Test

Soil Layer	e_o	OCR	C_c	C_r
1	0.94	1.68	0.34	0.03
2	0.80	2.01	0.30	0.03
3	0.54	1.71	0.20	0.02

The settlement at the South Abutment was first evaluated using an equivalent footing with stress distribution considering piles supported by shaft resistance in clay as depicted by Figure D-108. A discussion of this approach is provided in Section 7.3.5.3 of Chapter 7. Classic consolidation settlement equations were used to estimate pile group settlement. For this approach, an equivalent footing was placed at increasing pile penetration depths and resulting pile group settlements were estimated. Figure D-108 presents the equivalent footing concept with the appropriate stress distribution. The displayed group length dimension, Z , is 37 feet and is appropriate for only Group Configuration 3. The shallowest depth of an equivalent footing which satisfied vertical deformation requirements was thus determined. After calculating settlement, the location of the equivalent footing location was correlated to the required pile penetration depth. This depth was then used to establish the minimum required pile penetration depths to satisfy deformation requirements.

The following example calculation is performed for an equivalent footing located 35 feet below the bottom of footing. From Elev. 270.0 feet to Elev. 214.0 feet, settlement calculations for cohesive soil were performed for the silty clay layers. The stress increase from the superstructure and embankment loads was less than the preconsolidation stress at each layer. Therefore, Equation 7-57 was used to estimate recompression settlement for overconsolidated soil. It was furthermore assumed that compression of the underlying limestone bedrock is negligible.

The length of the pile group in Group Configuration 3 is 37 feet from exterior pile edge to exterior pile edge, and therefore this example is suitable for only this configuration. The same dimension for Group Configuration 1 and 2 is 41 feet. Similar to the stress increase calculations from embankment loading, the soil profile was again divided into 5 foot thick layers with the exception of the final layer which is 1 foot thick. The elevation shown in Table D-117 references the midpoint of each respective soil layer, while z is the depth below the equivalent footing to the midpoint of each respective soil layer. Tabulated values for this analysis are recorded in Table D-117 with a total settlement estimated of 1.21 inches.

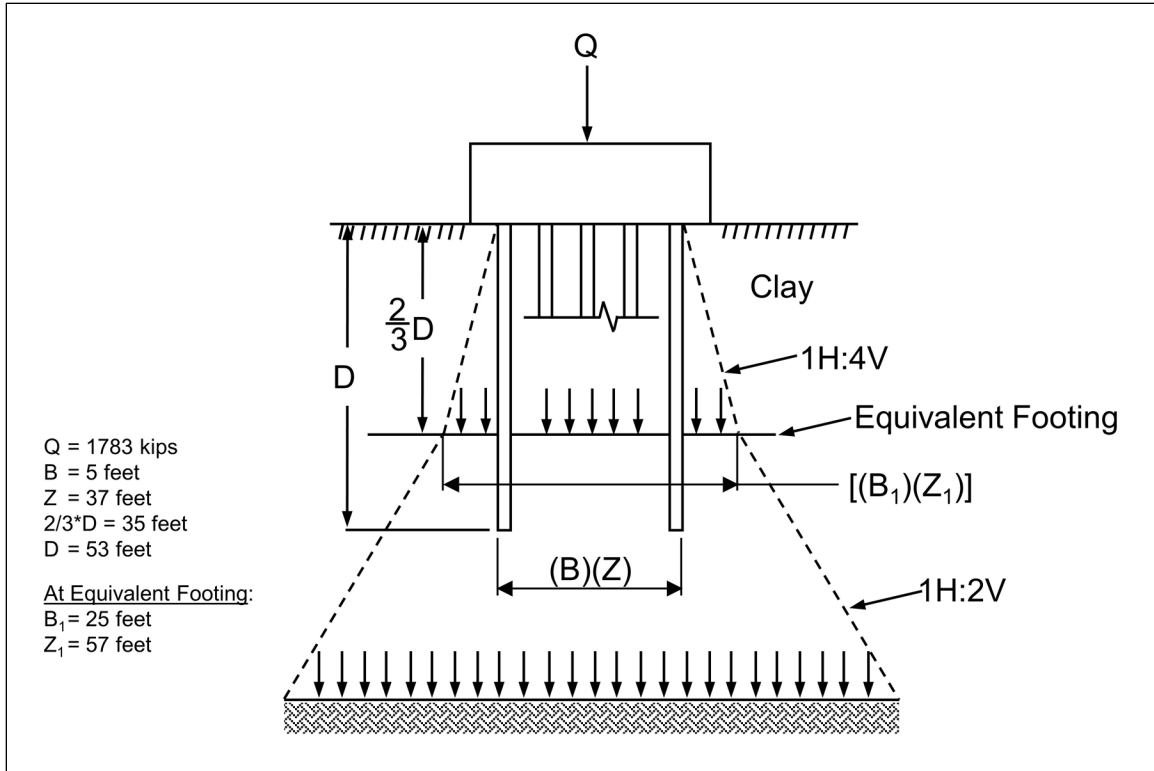


Figure D-108 Equivalent footing at 35 feet below the pile cap with respective stress distribution for conventional settlement analysis.

Considering only the unfactored permanent load, Q , calculate the vertical effective stress increase below the equivalent footing resulting from superstructure loads, $\Delta\sigma'_{v(ss)}$, at Elev. 267.5 feet.

- Q = unfactored permanent load, 1783 kips (Service I, without LL, Table D-105).
- B_1 = pile group width at equivalent footing, 25 feet.
- Z_1 = pile group length at equivalent footing, 57 feet (Group Configuration 3 only).
- z = depth below equivalent footing, 2.5 feet (Elev. 267.5 feet).

$$\Delta\sigma'_{v(ss)} = \frac{Q}{(B_1+z)+(Z_1+z)} \quad [\text{Eq. 7-55}]$$

$$\Delta\sigma'_{v(ss)} = \frac{(1783 \text{ kips})}{((25 \text{ feet})+(2.5 \text{ feet})) * ((57 \text{ feet})+(2.5 \text{ feet}))}$$

$$\Delta\sigma'_{v(ss)} = 1.09 \text{ ksf}$$

Including the vertical effective stress increase from the embankment and superstructure, calculate the total vertical effective stress increase below the equivalent footing at Elev. 267.5 feet.

- $\Delta\sigma'_{v(e)}$ = change in effective stress at depth z below the equivalent footing from embankment loading, 0.79 ksf (Table D-115).
 $\Delta\sigma'_{v(ss)}$ = change in effective stress at depth z below the equivalent footing from superstructure loading, 1.09 ksf.

$$\Delta\sigma'_{v(e+ss)} = \Delta\sigma'_{v(e)} + \Delta\sigma'_{v(ss)}$$

$$\Delta\sigma'_{v(e+ss)} = (0.79 \text{ ksf}) + (1.09 \text{ ksf})$$

$$\Delta\sigma'_{v(e+ss)} = 1.88 \text{ ksf}$$

Determine the preconsolidation stress from the initial vertical effective stress and overconsolidation ratio at depth z below the equivalent footing (Elev. 267.5 feet).

- σ'_{v0} = initial vertical effective stress at depth z below the equivalent footing, 2.24 ksf.
 OCR = overconsolidation ratio, 2.01 (Soil Layer 2, Table D-116).

$$\sigma'_p = \sigma'_{v0} * \text{OCR}$$

$$\sigma'_p = (2.24 \text{ ksf}) * (2.01)$$

$$\sigma'_p = 4.50 \text{ ksf}$$

Evaluate the stress increase at depth z below the equivalent footing (Elev. 267.5 feet) relative to the preconsolidation stress.

- σ'_{v0} = initial vertical effective stress below equivalent footing, 2.70 ksf.
 $\Delta\sigma'_{v(e+ss)}$ = change in effective stress below equivalent footing, 1.88 ksf.
 σ'_p = preconsolidation stress below equivalent footing, 4.50 ksf.

$$\sigma'_{v0} + \Delta\sigma'_{v(e+ss)} \leq \sigma'_p$$

$$(2.24 \text{ ksf}) + (1.88 \text{ ksf}) \leq (4.50 \text{ ksf})$$

Determine the stress increase by comparing σ'_{v0} and $\sigma'_{1(ss+e)}$.

$$\text{Stress Increase} = \frac{\sigma'_{1(ss+e)} - \sigma'_{v0}}{\sigma'_{v0}} * 100\% \quad [\text{Eq. D-5}]$$

$$\text{Stress Increase} = \frac{(4.12 \text{ ksf}) - (2.24 \text{ ksf})}{(2.24 \text{ ksf})} * 100\%$$

$$\text{Stress Increase} = 84\%$$

The stress increase is greater than or equal to 10%. Deformation for this depth increment should be estimated and included in the sum of all depth increments in which the stress increase is not less than 10%.

Estimate consolidation settlement of layer for overconsolidated soil using Equation 7-57 for ($\sigma'_{v0} + \Delta\sigma \leq \sigma'_p$).

- e_o = initial soil layer void ratio, 0.80 (Soil Layer 2, Table D-116).
- C_r = recompression index, 0.03 (Soil Layer 2, Table D-116).
- H_o = initial height of soil layer, 5 feet.
- σ'_{v0} = initial vertical effective stress at depth z below the equivalent footing, 2.24 ksf.
- $\Delta\sigma'_{v(e+ss)}$ = change in vertical effective stress at depth z below the equivalent footing, 1.88 ksf.

$$S_c = \frac{C_r}{1+e_o} H_o \log \left(\frac{\sigma'_{v0} + \Delta\sigma'_v}{\sigma'_{v0}} \right) \quad [\text{Eq. 7-57}]$$

$$S_c = \frac{(0.03)}{1+(0.80)} (5 \text{ feet}) * \log \left(\frac{(2.24 \text{ ksf}) + (1.88 \text{ ksf})}{(2.24 \text{ ksf})} \right)$$

$$S_c = 0.0221 \text{ feet} * \left(\frac{12 \text{ inches}}{1 \text{ foot}} \right) = 0.26 \text{ inches}$$

Table D-117 Calculation of Settlement using Equivalent Footing and Conventional Primary Consolidation Equations

EL (feet)	z (feet)	H _o (feet)	σ' _{vo} (ksf)	OCR	σ' _p (ksf)	B (feet)	Z (feet)	Δσ' _{v(ss)} (ksf)	Δσ' _{v(e+ss)} (ksf)	Stress Incr. (%)	S (in)
267.5	2.5	5	2.24	2.01	4.504812	27.5	59.5	1.09	1.88	84	0.26
262.5	7.5	5	2.54	1.71	4.3398945	32.5	64.5	0.85	1.64	65	0.17
257.5	12.5	5	2.87	1.71	4.902912	37.5	69.5	0.68	1.48	51	0.14
252.5	17.5	5	3.20	1.71	5.472342	42.5	74.5	0.56	1.35	42	0.12
247.5	22.5	5	3.53	1.71	6.041772	47.5	79.5	0.47	1.26	36	0.10
242.5	27.5	5	3.87	1.71	6.611202	52.5	84.5	0.40	1.18	31	0.09
237.5	32.5	5	4.20	1.71	7.180632	57.5	89.5	0.35	1.12	27	0.08
232.5	37.5	5	4.53	1.71	7.750062	62.5	94.5	0.30	1.07	24	0.07
227.5	42.5	5	4.87	1.71	8.319492	67.5	99.5	0.27	1.02	21	0.06
222.5	47.5	5	5.20	1.71	8.888922	72.5	104.5	0.24	0.98	19	0.06
217.5	52.5	5	5.53	1.71	9.458352	77.5	109.5	0.21	0.95	17	0.05
216.5	53.5	1	5.21	1.71	8.9038845	78.5	110.5	0.21	0.94	18	0.01
										Total:	1.21

The above analysis was performed for additional equivalent footing locations considering dimensions of the three trial group configurations. Table D-118 summarizes the analysis results for pile group settlement using the equivalent footing with conventional primary consolidation. As established by global project performance requirements, total vertical deformation should be limited to 1.5 inches at each substructure location. To account for the additional deformation from elastic pile compression, the equivalent footing should be located 35 feet below the bottom of pile cap (i.e., located at Elev. 270.0 feet). Using the equivalent footing to pile penetration depth relationship displayed in Figure D-108, the respective pile penetration depth, *D*, is 53 feet ((3/2) * 35 feet ≈ 53 feet) for an equivalent footing at a depth of 35 feet.

Table D-118 Summary of Pile Group Settlement Estimates Based on Equivalent Footing Depth and Conventionally Settlement Computations

Equivalent Footing Elevation (feet)	Equivalent Footing Depth (feet)	Pile Toe Elevation (feet)	Pile Toe Depth (feet)	Estimated Settlement Group Configuration 1 and 2 (inches)	Estimated Settlement Group Configuration 3 (inches)
280.0	25	267.0	38	5.63	6.22
275.0	30	260.0	45	1.46	2.85
270.0	35	252.0	53	1.20	1.21
265.0	40	245.0	60	0.97	0.99

Pile group settlement was also evaluated using the neutral plane method and Janbu tangent modulus approach as outlined in Section 7.3.5 of Chapter 7. To compare settlement estimates using this approach with the conventional consolidation theory results presented in Table D-118, the neutral plane was placed at the same elevation as the equivalent footing location that resulted in the estimated settlement of 1.21 inches (i.e., the neutral plane was placed at 35 feet). Figure D-109 illustrates the neutral plane location and stress distribution used with this approach. To highlight the differences between these two methods, the equivalent footing at 35 feet in Figure D-109 has plan dimension of 5 feet by 37 feet. However, the equivalent footing in Figure D-108 used with the conventional consolidation theory has an equivalent footing plan dimension of 25 feet by 57 feet. As a result of the greater contact stress, the neutral plane calculated settlements are significantly greater.

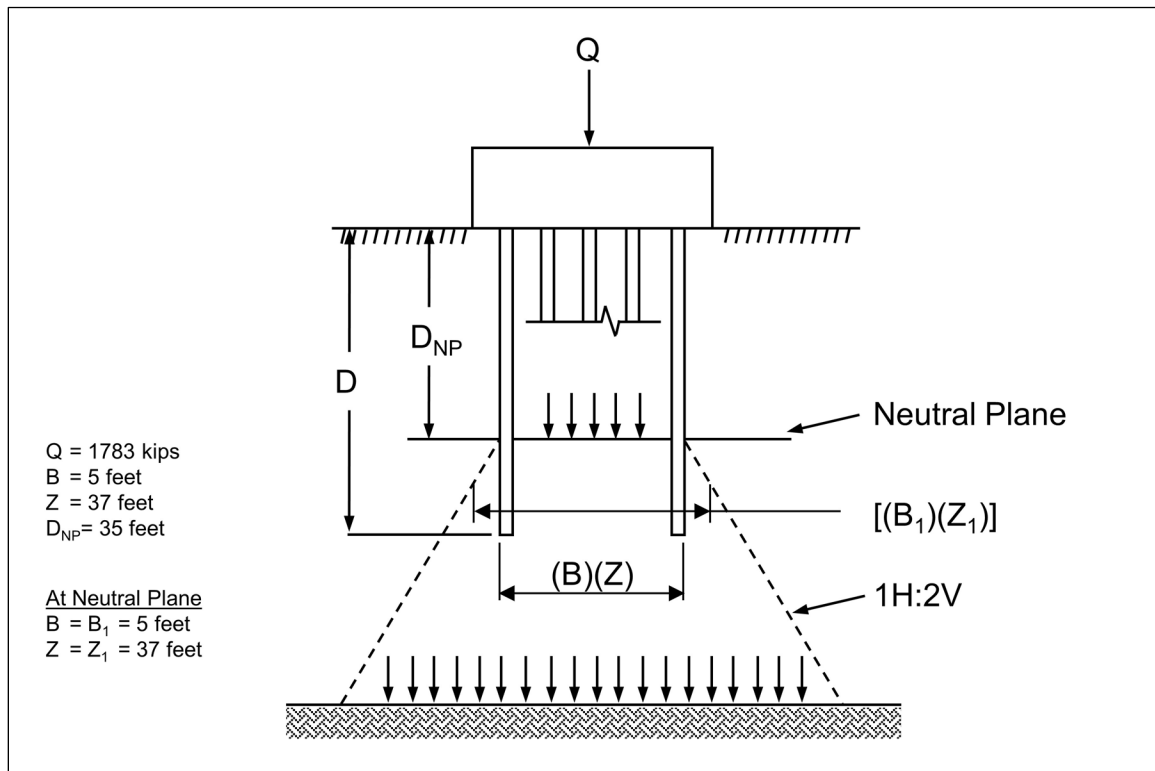


Figure D-109 Neutral plane at 35 feet below the pile cap and resulting stress distribution.

The following settlement calculation is performed for the neutral plane located 35 feet below the bottom of footing. From Elev. 270.0 feet to 214.0 feet, strain calculations for cohesive soil (stress exponent of $j=0$) were applied for Soil Layers 1 to 3. It was assumed that vertical deformation below the encountered bedrock at Elev. 214.0 feet is negligible. The length of the pile group in Group Configuration 3

is 37 feet from exterior pile edge to exterior pile edge, and therefore this example is suitable for only this configuration. The same dimension for Group Configuration 1 and 2 is 41 feet. Similar to the stress increase calculations from embankment loading, the soil profile was again divided into 5 foot thick layers with the exception of the final layer which is 1 foot thick. The elevation shown in Table D-119 references the midpoint of each respective soil layer, while z is the depth below the equivalent footing to the midpoint of each respective soil layer. Tabulated values for this analysis are recorded in Table D-119.

Considering only the unfactored permanent load, Q , calculate the vertical effective stress increase below the equivalent footing resulting from superstructure loads, $\Delta\sigma'_{v(ss)}$, at EL 267.5 feet.

- Q = unfactored permanent load, 1783 kips (Service I, without LL, D-105).
- B = pile group width, 5 feet.
- Z = pile group length, 37 feet (Group Configuration 3 only).
- z = depth below equivalent footing, 2.5 feet (Elev. 267.5 feet).

$$\Delta\sigma'_{v(ss)} = \frac{Q}{(B+z)+(Z+z)} \quad [\text{Eq. 7-55}]$$

$$\Delta\sigma'_{v(ss)} = \frac{(1783 \text{ kips})}{((5 \text{ feet})+(2.5 \text{ feet}))*((37 \text{ feet})+(2.5 \text{ feet}))}$$

$$\Delta\sigma'_{v(ss)} = 6.02 \text{ ksf}$$

Including the vertical effective stress increase from the embankment and superstructure, calculate the total vertical effective stress increase below the equivalent footing at EL 267.5 feet.

- $\Delta\sigma'_{v(e)}$ = change in effective stress at depth z below the equivalent footing from embankment loading, 0.79 ksf (Table D-115).
- $\Delta\sigma'_{v(ss)}$ = change in effective stress at depth z below the equivalent footing from superstructure loading, 6.02 ksf.

$$\Delta\sigma'_{v(e+ss)} = \Delta\sigma'_{v(e)} + \Delta\sigma'_{v(ss)}$$

$$\Delta\sigma'_{v(e+ss)} = (0.79 \text{ ksf}) + (6.02 \text{ ksf})$$

$$\Delta\sigma'_{(e+ss)} = 6.81 \text{ ksf}$$

Determine the preconsolidation stress from the initial vertical effective stress and overconsolidation ratio at EL 267.5 feet.

- σ'_{vo} = initial vertical effective stress at depth z below the equivalent footing, 2.24 ksf.
 OCR = overconsolidation ratio, 2.01 (Soil Layer 2, Table D-116).

$$\sigma'_p = \sigma'_{vo} * OCR$$

$$\sigma'_p = (2.24 \text{ ksf}) * (2.01)$$

$$\sigma'_p = 4.50 \text{ ksf}$$

Evaluate the stress increase at depth at EL 267.5 feet relative to the preconsolidation stress.

- σ'_{vo} = initial vertical effective stress at depth z below the equivalent footing, 2.24 ksf.
 $\Delta\sigma'_{v(e+ss)}$ = change in effective stress at depth z below the equivalent footing, 6.81 ksf.
 σ'_p = preconsolidation stress at depth z below the equivalent footing, 4.50 ksf.

$$\sigma'_{vo} + \Delta\sigma'_{v(e+ss)} \geq \sigma'_p$$

$$(2.24 \text{ ksf}) + (6.81 \text{ ksf}) \geq (4.50 \text{ ksf})$$

Determine the percent stress increase by comparing σ'_{vo} and $\sigma'_{1(e+ss)}$.

- σ'_{vo} = initial vertical effective stress at depth z below the equivalent footing, 2.24 ksf.
 $\sigma'_{1(e+ss)}$ = new vertical effective stress considering both superstructure and embankment loads, 2.24 ksf + 6.81 ksf = 9.05 ksf.

$$\text{Percent Stress Increase} = \frac{\sigma'_{1(e+ss)} - \sigma'_{vo}}{\sigma'_{vo}} * 100 \quad [\text{Eq. D-5}]$$

$$\text{Percent Stress Increase} = \frac{(9.05) - (2.24)}{2.24} * 100$$

$$\text{Percent Stress Increase} = 304\%$$

For cohesive soil at Elevation 272.5 feet, $z = 2.5$ ($j = 0$), the modulus number is determined by Equation 7-68.

- e_o = void ratio, 0.80 (Soil Layer 2, Table D-116).
 C_c = compression index, 0.30 (Soil Layer 2, Table D-116).

$$m_n = 2.30 \left[\frac{1+e_o}{C_c} \right] \quad [\text{Eq. 7-68}]$$

$$m_n = 2.30 \left[\frac{1+(0.80)}{(0.30)} \right]$$

$$m_n = 13.8$$

For cohesive soil at Elevation 272.5 feet, $z = 2.5$ ($j = 0$), the recompression modulus number is determined by Equation 7-69.

- e_o = initial soil layer void ratio, 0.80 (Soil Layer 2, Table D-116).
 C_c = recompression index, 0.03 (Soil Layer 2, Table D-116).

$$m_{nr} = 2.30 \left[\frac{1+e_o}{C_r} \right] \quad [\text{Eq. 7-69}]$$

$$m_{nr} = 2.30 \left[\frac{1+(0.80)}{(0.03)} \right]$$

$$m_{nr} = 138$$

For overconsolidated cohesive soils ($j = 0$) in which the new vertical effective stress exceeds the preconsolidation stress, determine strain in layer with Equation 7-65.

- σ'_p = preconsolidation stress, 4.50 ksf.
 σ'_{v0} = initial vertical effective stress at depth z below the equivalent footing, 2.24 ksf.
 $\sigma'_{1(e+ss)}$ = new vertical effective stress, 9.05 ksf.
 m_{nr} = recompression modulus, 138.
 m_n = compression modulus, 13.8.

$$\varepsilon = \frac{1}{m_{nr}} \ln \left[\left(\frac{\sigma'_p}{\sigma'_{v0}} \right) \right] + \frac{1}{m_n} \ln \left[\left(\frac{\sigma'_{1}}{\sigma'_p} \right) \right] \quad [\text{Eq. 7-65}]$$

$$\varepsilon = \frac{1}{(138)} \ln \left[\left(\frac{(4.50 \text{ ksf})}{(2.24 \text{ ksf})} \right) \right] + \frac{1}{(13.8)} \ln \left[\left(\frac{(9.05 \text{ ksf})}{(4.50 \text{ ksf})} \right) \right]$$

$$\varepsilon = 0.0557$$

Calculate the layer compression denoted, S , with the initial height of the layer, H_o .

$$S = \Delta\varepsilon * H_o$$

$$S = 0.0557 * (5 \text{ feet}) \left(\frac{12 \text{ inches}}{1 \text{ foot}} \right)$$

$$S = 3.34 \text{ inches}$$

For overconsolidated cohesive soils ($j = 0$) in which the new effective stress does not exceed the preconsolidation stress, such as at EL 252.5, determine strain in layer with Equation 7-66.

- σ'_p = preconsolidation pressure, 5.47 ksf (Table D-119).
- σ'_{vo} = initial vertical effective stress at depth z below the equivalent footing, 3.20 ksf (Table D-119).
- $\sigma'_{1(e+ss)}$ = new vertical effective stress, 5.45 ksf (Table D-119).
- m_{nr} = recompression modulus, 177.1 (Calculation not presented for Soil Layer 3).

$$\varepsilon = \frac{1}{m_{nr}} \ln \left[\left(\frac{\sigma'_{1(e+ss)}}{\sigma'_{vo}} \right) \right] \quad [\text{Eq. 7-66}]$$

$$\varepsilon = \frac{1}{(177.1)} \ln \left[\left(\frac{(5.35 \text{ ksf})}{(3.20 \text{ ksf})} \right) \right]$$

$$\varepsilon = 0.0030$$

Calculate the layer compression denoted, S , with the initial height of the layer, H_o .

$$S = \varepsilon * H_o$$

$$S = 0.0030 \text{ feet} * (5 \text{ feet}) \left(\frac{12 \text{ inches}}{1 \text{ foot}} \right)$$

$$S = 0.18 \text{ inches}$$

Table D-119 Settlement Estimate for Neutral Plane Method with
the Neutral Plane at EL 270.0 feet

EL (feet)	z (feet)	H _o (ksf)	σ' _{vo} (ksf)	OCR	σ' _p (ksf)	B (feet)	Z (feet)	Δσ' _{ss} (ksf)	σ' _{1e+ss} (ksf)	Stress Incr. (%)	ε	S (in)
267.5	2.5	5	2.24	2.01	4.50	7.5	39.5	6.02	9.05	304	0.0556	3.34
262.5	7.5	5	2.54	1.71	4.34	12.5	44.5	3.21	6.53	157	0.0261	1.57
257.5	12.5	5	2.87	1.71	4.90	17.5	49.5	2.06	5.72	99	0.0117	0.70
252.5	17.5	5	3.20	1.71	5.47	22.5	54.5	1.45	5.45	70	0.0030	0.18
247.5	22.5	5	3.53	1.71	6.04	27.5	59.5	1.09	5.41	53	0.0024	0.14
242.5	27.5	5	3.87	1.71	6.61	32.5	64.5	0.85	5.50	42	0.0020	0.12
237.5	32.5	5	4.20	1.71	7.18	37.5	69.5	0.68	5.66	35	0.0017	0.10
232.5	37.5	5	4.53	1.71	7.75	42.5	74.5	0.56	5.86	29	0.0015	0.09
227.5	42.5	5	4.87	1.71	8.32	47.5	79.5	0.47	6.10	25	0.0013	0.08
222.5	47.5	5	5.20	1.71	8.89	52.5	84.5	0.40	6.35	22	0.0011	0.07
217.5	52.5	5	5.53	1.71	9.46	57.5	89.5	0.35	6.61	20	0.0010	0.06
214.5	53.5	1	5.21	1.71	8.90	58.5	90.5	0.34	6.27	20	0.0011	0.01
											Total	6.46

For the same equivalent footing location of Elev. 270 feet, the above results of 6.46 inches of estimated settlement using the neutral plane approach is significantly larger than the 1.23 inches of settlement (Table D-118) based on consolidation theory. For this example calculation, it was decided that the settlement method yielding the greatest settlement magnitude would be used to establish the range of pile penetration depths that satisfy tolerable deformations. The neutral plane method illustrated above was performed for additional equivalent footing depths until tolerable settlements were estimated.

Table D-120 summarizes analysis results for pile group settlement estimated using the neutral plane method and Janbu Tangent modulus approach. Although not yet calculated, it is preliminarily estimated that elastic pile compression may be on the order of 0.5 inches, effectively limiting tolerable soil settlement to on the order of 1 inch. Therefore, to limit the total vertical deformation including elastic pile compression and settlement to less than 1.5 inches, analysis results in Table D-120 indicate that the neutral plane should be located at Elev. 215.0 feet, 90 feet below the bottom of pile cap.

Table D-120 Summary of Pile Group Settlement Estimates Based on Janbu Tangent Modulus with Neutral Plane Method For All Group Configurations

Neutral Plane Elevation (feet)	Neutral Plane Depth (feet)	Estimated Settlement Group Configuration 1 and 2 (inches)	Estimated Settlement Group Configuration 3 (inches)
235.0	70	1.85	2.13
230.0	75	1.50	1.77
225.0	80	1.23	1.41
220.0	85	1.07	1.25
215.0	90	0.28	0.35

It is assumed that the equivalent footing acts at the same location as the neutral plane. Accordingly, an analysis was then performed to determine the pile toe elevation necessary to locate the neutral plane at Elev. 215.0, thereby establishing the minimum required pile penetration depth to satisfy tolerable deformations.

The location of the neutral plane and the magnitude of drag force are evaluated following the procedure outlined in Section 7.3.6 of Chapter 7, using unfactored permanent loads and nominal geotechnical resistance. Because load factors for the Service limit state are 1.0, applicable loads at this limit state may be considered unfactored. The Service I, without LL limit state loads are therefore used for evaluation for the neutral plane. This example again utilizes the load for Group Configuration 3 ($Q = 127$ kips, Table D-108). Figure D-110 presents a graphical interpretation of the neutral plane for the HP 12x74 pile driven to 91 feet (seated on bedrock at Elev. 214.0).

First, the sustained load plus the cumulative shaft resistance versus depth is plotted. Next, the mobilized toe resistance minus the cumulative shaft resistance versus pile penetration depth is plotted. The exact percentage of toe mobilization is unknown at this stage of design, and therefore multiple toe mobilization curves should be evaluated to determine the neutral plane location. The 0% toe mobilization curve is the most conservative location to evaluate pile settlement since it locates the neutral plane at the highest elevation. The 100% toe mobilization curve should be used to check the pile section's structural strength since it results in the greatest axial force in the pile. The structural strength check is performed in Block 17.

At the South Abutment, it is expected that piles will be supported partially by toe resistance on bedrock, and therefore the 0% toe mobilization curve presents an unreasonable baseline to evaluate settlement in this case. Some toe resistance is likely mobilized, and to remain consistent with the other substructure locations, the 50 % toe mobilization curve is again used to evaluate settlement.

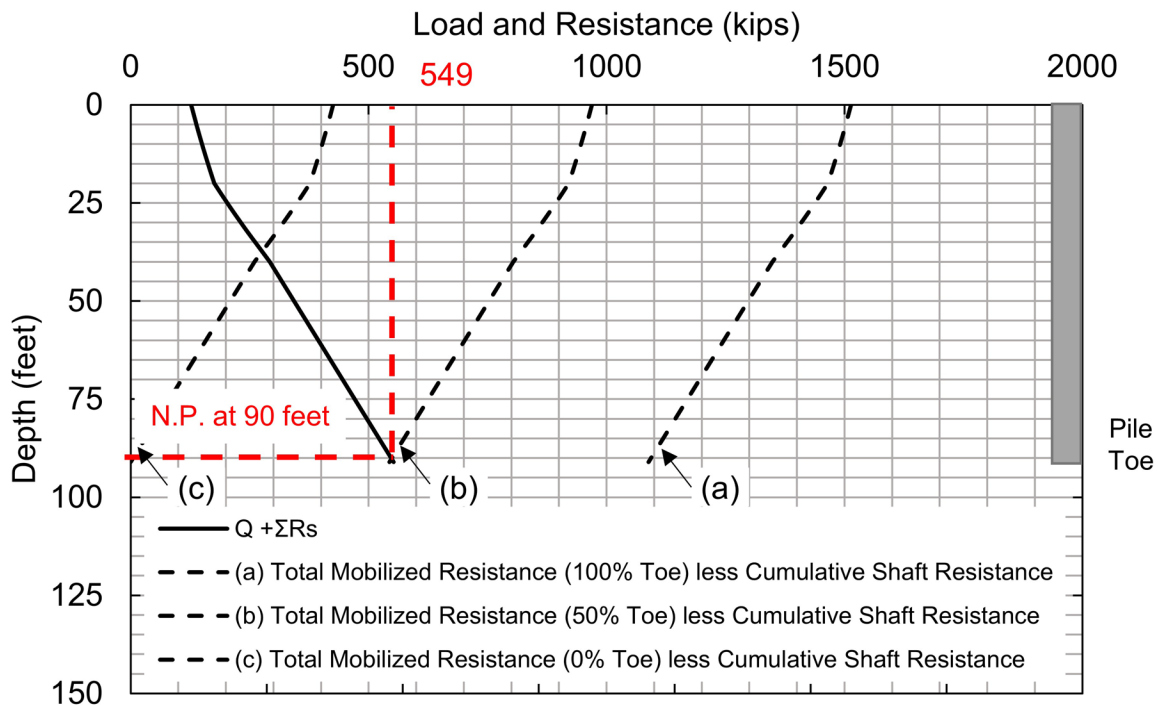


Figure D-110 Neutral plane location considering 50% toe mobilization for HP 12x74 at the South Abutment.

Assuming 50 % toe mobilization, the neutral plane is located 90 feet below the bottom footing with a resulting maximum load in the pile of 549 kips. Since the pile toe is seated on bedrock at EL 214.0, vertical deformation will be limited to elastic shortening of the pile and elastic compression of the bedrock which is expected to be negligible. The minimum pile penetration required to satisfy tolerable deformations is therefore approximately 91 feet resulting in the piles being driven to bedrock to control vertical deformations.

This procedure was performed for the remaining candidate pile sections and trial group configurations. , the above analysis was also performed to determine the pile penetration depth required to locate the neutral plane below a depth of 90 feet (EL 215.0 ft). The results of these analyses indicate that all the candidate pile sections and trial group configurations require the pile to be driven to bedrock (i.e., at an approximate depth of 91 feet.

As previously noted, a comparison of the required pile penetration depth determined from both settlement estimation methods yielded significantly different results. This is a direct result of the equivalent footing dimensions and stress distribution for each respective method. Previously, Table D-118 summarized the minimum pile penetration depth based on conventional settlement estimates for piles supported by shaft resistance in clay. Table D-121 presents the minimum pile penetration depth for all pile sections and group configurations based on settlement estimation using the neutral plane method with Janbu Tangent modulus. In the latter settlement approach, for all candidate pile sections and group configurations, the piles must be driven to bedrock at approximately 91 feet.

Experience should be used to establish a penetration depth with either settlement estimation approach. For this design example, the established minimum pile penetration depth to satisfy tolerable vertical deformations will be based solely on the settlement results using the neutral plane and Janbu tangent modulus. Table D-121 presents the established pile penetration depths to satisfy tolerable deformations at the South Abutment.

Table D-121 Established Minimum Required Pile Penetration Depths to Satisfy Tolerable Deformations at the South Abutment

Group Configuration	All Candidate Pile Sections (feet)
1, 2 and 3	Bedrock (\approx 91)

Elastic shortening of the pile should be considered along with settlement. For elastic compression, the load per pile from the Service I, without live load limit state is applied at the pile head. As shown in Table D-108, this load is 127 kips.

Note that the drag force from negative shaft resistance increases the axial compression force in the pile. Negative shaft resistance above the neutral plane acts to increase axial compression force in the pile, whereas below the neutral plane, positive shaft resistance reduces the axial compression force in the pile. This effect must be accounted for in the elastic compression calculation. Accordingly, the unfactored axial load used to compute elastic compression, Q , changes for each pile segment length, increasing equal to the unfactored permanent load plus the shaft resistance down to the neutral plane. In this example, the unfactored axial load is equal to the resistance distribution from 100% toe mobilization. The highest drag force magnitude results from this curve and represents the worst case.

Equation 7-48 is used to illustrate this example for the first 12 inch increment of the HP 12x74 pile section. The average shaft resistance and average load for each respective depth interval is used to estimate the elastic compression. For each 12 inch segment, the elastic modulus remains constant, and was evaluated as 29,000 ksi. The pile cross sectional area likewise remains constant as 21.8 in². Remaining calculations were performed using a spreadsheet; Table D-122 summarizes the elastic compression with depth.

Determine the unfactored axial load, Q , in segment.

- Q_d = unfactored permanent load, 127.0 kips.
- R_s^- = average (negative) shaft resistance, 1.1 kips.

$$Q = Q_d + R_s^- = 127.0 \text{ kips} + 1.1 \text{ kips}$$

$$Q = 128.1 \text{ kips}$$

Calculate elastic compression of segment with unfactored axial load from combined unfactored permanent load and negative shaft resistance.

- L = segment length, 12 inches.
- A = cross sectional area of pile material, 21.8 in².
- E = elastic modulus of pile, 29,000 ksi.

$$\Delta = \frac{QL}{AE} \quad [\text{Eq. 7-48}]$$

$$\Delta = \frac{(128.1 \text{ kips}) * (12 \text{ inches})}{(21.8 \text{ in}^2) * (29,000 \text{ ksi})}$$

$$\Delta = 0.00243 \text{ inches}$$

Table D-122 Elastic Compression Calculation

Depth Below Pile Head (feet)	Average Shaft Resistance (kips)	Average Unfactored Axial Load (kips)	Δ (inches)
0	0.0	127.0	0.00000
0-1	1.1	128.1	0.00243
1-2	3.3	130.3	0.00247
2-3	5.5	132.5	0.00252
3-4	7.7	134.7	0.00256
4-5	10.0	137.0	0.00260
5-6	12.2	139.2	0.00264
6-7	14.5	141.5	0.00269
7-8	16.9	143.9	0.00273
8-9	19.2	146.2	0.00278
9-10	21.6	148.6	0.00282
10-11	24.1	151.1	0.00287
11-12	26.5	153.5	0.00291
12-13	29.0	156.0	0.00296
13-14	31.5	158.5	0.00301
14-15	34.1	161.1	0.00306
15-16	36.7	163.7	0.00311
16-17	39.3	166.3	0.00316
17-18	41.9	168.9	0.00321
18-19	44.6	171.6	0.00326
19-20	47.3	174.3	0.00331
20-21	51.3	178.3	0.00339
21-22	56.7	183.7	0.00349
22-23	62.1	189.1	0.00359
23-24	67.6	194.6	0.00369
24-25	73.2	200.2	0.00380
25-26	78.7	205.7	0.00391
26-27	84.3	211.3	0.00401
27-28	90.0	217.0	0.00412
28-29	95.7	222.7	0.00423
29-30	101.6	228.6	0.00434
30-31	107.4	234.4	0.00445
31-32	113.3	240.3	0.00456
32-33	119.2	246.2	0.00467
33-34	125.3	252.3	0.00479
34-35	131.5	258.5	0.00491
35-36	137.7	264.7	0.00502
36-37	143.8	270.8	0.00514
37-38	149.9	276.9	0.00526

Table D-122 Elastic Compression Calculation (continued)

Depth Below Pile Head (feet)	Average Shaft Resistance (kips)	Average Unfactored Axial Load (kips)	Δ (inches)
38-39	156.0	283.0	0.00537
39-40	162.2	289.2	0.00549
40-41	167.8	294.8	0.00560
41-42	172.9	299.9	0.00569
42-43	178.0	305.0	0.00579
43-44	183.1	310.1	0.00589
44-45	188.2	315.2	0.00598
45-46	193.3	320.3	0.00608
46-47	198.4	325.4	0.00618
47-48	203.5	330.5	0.00627
48-49	208.6	335.6	0.00637
49-50	213.7	340.7	0.00647
50-51	218.8	345.8	0.00656
51-52	223.9	350.9	0.00666
52-53	229.0	356.0	0.00676
53-54	234.1	361.1	0.00685
54-55	239.3	366.3	0.00695
55-56	244.4	371.4	0.00705
56-57	249.4	376.4	0.00715
57-58	254.5	381.5	0.00724
58-59	259.6	386.6	0.00734
59-60	264.8	391.8	0.00744
60-61	269.9	396.9	0.00753
61-62	275.0	402.0	0.00763
62-63	280.1	407.1	0.00773
63-64	285.2	412.2	0.00782
64-65	290.3	417.3	0.00792
65-66	295.4	422.4	0.00802
66-67	300.5	427.5	0.00811
67-68	305.6	432.6	0.00821
68-69	310.7	437.7	0.00831
69-70	315.8	442.8	0.00840
70-71	320.9	447.9	0.00850
71-72	326.0	453.0	0.00860
72-73	331.1	458.1	0.00869
73-74	336.2	463.2	0.00879
74-75	341.3	468.3	0.00889
75-76	346.4	473.4	0.00899

Table D-122 Elastic Compression Calculation (continued)

Depth Below Pile Head (feet)	Average Shaft Resistance (kips)	Average Unfactored Axial Load (kips)	Δ (inches)
76-77	351.5	478.5	0.009082
77-78	356.6	483.6	0.009179
78-79	361.7	488.7	0.009276
79-80	366.8	493.8	0.009373
80-81	371.9	498.9	0.009470
81-82	377.0	504.0	0.009567
82-83	382.1	509.1	0.009664
83-84	387.2	514.2	0.009760
84-85	392.3	519.3	0.009858
85-86	397.4	524.4	0.009955
86-87	402.5	529.5	0.010051
87-88	407.6	534.6	0.010148
88-89	412.7	539.7	0.010245
89-90	417.9	544.9	0.010342
90-91	423.0	550.0	0.010439
			0.55

For the pile head load of 127 kips (Group Configuration 3 loads), estimated elastic compression of the HP 12x74 pile section driven to 91 feet is 0.55 inches. As the piles are driven to bedrock, it is assumed no settlement will occur, and therefore it is estimated that total vertical deformation at the South Abutment is 0.55 inches.

D.40 Block 16: South Abutment – Check pile drivability to maximum pile penetration depth requirements established in Blocks 12 through 15

Preliminary pile drivability was previously evaluated for the 5 candidate pile sections in Block 10. The plots of nominal resistance, blow count and compression stress versus depth in Figure D-99 should now be reviewed considering the established minimum pile penetration depths. A candidate pile section must be capable of being driven to the penetration depth necessary to achieve the nominal geotechnical resistance in axial compression and tension, and to a penetration depth necessary to satisfy lateral load demands as well as axial and lateral deformation requirements. Minimum pile penetration depths have been previously established in Blocks 13, 14 and 15 and are presented in Tables D-111, D-114 and D-126.

Although a minimum pile penetration depth is not typically established for nominal geotechnical resistance in axial compression, the pile should also be capable of being driven reasonably close to the estimated pile penetration depth where the nominal resistance is expected to develop. If the pile cannot be driven to the required depth within driving stress limits and at reasonable blow counts, a larger pile hammer, a pile section with greater impedance, or pile installation aids such as predrilling or jetting may be required to satisfy or improve drivability. Alternatively, substructure design modifications should be considered.

For the candidate HP 12x74 pile section in Group Configuration 3, the pile penetration depth for axial compression loading was estimated at 73 feet (Table D-110), the minimum penetration depth for axial tension loading was 18 feet (Table D-111), the minimum penetration depth for lateral loading was 25 feet (Table D-114) and the estimated minimum penetration depth to satisfy vertical deformation limits was 91 feet (Table D-126). Accordingly, candidate pile section must have sufficient drivability to the maximum of these depths, 91 feet, where driving is anticipated to terminate on bedrock.

A review of the preliminary drivability in Figure D-99 indicates that the HP 12x74 pile section can be driven to bedrock at approximately 91 feet. In addition, from the preliminary drivability evaluation (with soil and hammer model assumptions described in Block 10), it is estimated that the blow count will not exceed 120 blows per foot or 10 blows per inch before this penetration depth. Compression driving stresses are estimated to remain below driving-stress limits. Therefore, it is concluded that pile drivability to the estimated pile penetration depth is acceptable.

D.41 Block 17: South Abutment – Determine the Neutral Plane Location and Resulting Drag Force. Check Structural Strength Limit State for Pile Penetration Depth From Block 16

Previously in Block 15, the neutral plane was evaluated as part of the settlement calculations. The neutral plane and resulting drag force are now evaluated to check the structural strength limit state for candidate pile sections. Section 7.3.6 provides guidance for evaluating the neutral plane location and the magnitude of the drag force. Load using the Service I, without LL limit state are considered for the applied pile head load. This example again utilizes the load for Group Configuration 3 ($Q = 127$ kips, Table D-108).

At 100 percent toe mobilization, the neutral plane is at its lowest potential location, and thus the highest drag force magnitude results. Accordingly, this is the toe

mobilization curve that should be used to check the pile section's structural strength. Figure D-111 presents a graphical interpretation of the neutral plane for the HP12x74 pile section driven to the estimated pile penetration depth of 91 feet (seated on bedrock). In this case the neutral plane is located 91 feet below pile head with a resulting maximum axial compression force in the pile of 552 kips.

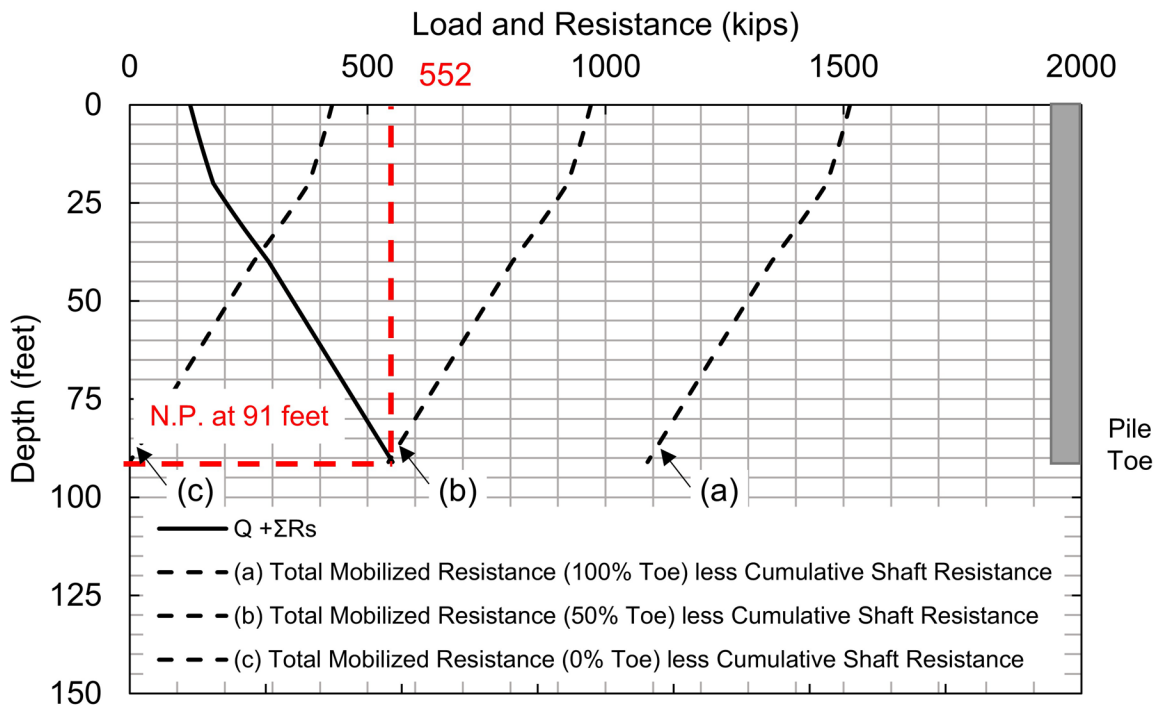


Figure D-111 Neutral plane location considering 100 percent toe mobilization for HP 12x74 at the South Abutment.

The resulting unfactored drag force, DF , is the difference between the maximum unfactored axial compression force in the pile, Q_{max} , minus the unfactored permanent load (Q). In this case, the drag force is evaluated for 100 percent toe mobilization.

$$DF = Q_{max} - Q$$

$$DF = (552 \text{ kips}) - (127 \text{ kips}) = 425 \text{ kips}$$

Following this calculation, the structural resistance was evaluated with Equation 7-70. As discussed in Section 7.3.6 of Chapter 7, a load factor of 1.25 is applied to the permanent load while a load factor of 1.1 is applied to the drag force. When driving through dense soil to rock, it may be generally appropriate to assume that the pile toe may be subject to damage during driving. Therefore, a structural resistance

factor for axial compression, ϕ_c , of 0.5 is applied to the nominal structural resistance of 1088 kips (Table D-94). For this assumption, the factored structural resistance, P_r , for the HP 12x74 section is 544 kips.

$$P_u = 1.25 (Q) + \gamma_p (DF) < P_r \quad [\text{Eq. 7-70}]$$

$$P_u = 1.25 (127 \text{ kips}) + 1.1(425 \text{ kips}) = 626 \text{ kips}$$

$$626 \text{ kips} > 544 \text{ kips}$$

In this case, the factored structural resistance is less than the factored loads, and therefore the pile section is unacceptable.

It may be appropriate to reconsider the structural resistance factor used above based upon the driving conditions. The pile is driven through silty clay for approximately 91 feet, and the probability of damage in this zone is very low. In addition, a review of the boring logs (Figure D-2) indicates the transition from silty clay to bedrock is relatively abrupt, and there will be little difficulty seating the pile on bedrock. If an appropriately sized hammer (or appropriate energy setting) is selected, dynamic testing is performed to monitor and control driving stresses, and driving criteria is established such that the risk of pile toe damage is low, the use of a higher structural resistance factor may be warranted. Consideration could also be given to dynamically testing 100% of the piles as a further means of identifying and mitigating any damaged piles. AASHTO (2014) design specifications allow a structural resistance factor for axial compression of 0.6 for H-pile sections with good driving conditions where damage is unlikely.

Proceeding with the drag force analysis, if a structural resistance factor for axial compression, ϕ_c , of 0.6 is applied to the nominal structural resistance of 1088 kips (Table D-94), the factored structural resistance, P_r , for the HP 12x74 section is 652 kips. Equation 7-68 is used to evaluate the structural resistance.

$$P_u = 1.25 (Q) + \gamma_p (DF) < P_r \quad [\text{Eq. 7-70}]$$

$$P_u = 1.25 (127 \text{ kips}) + 1.1(425 \text{ kips}) = 626 \text{ kips}$$

$$626 \text{ kips} < 652 \text{ kips}$$

In this case, the factored structural resistance is greater than the factored load, and therefore the pile section is acceptable.

It may be beneficial to review the ratio of factored load to nominal structural resistance or, P_u / P_n . The following evaluation serves to back calculate the minimum required structural resistance factor, $\phi_{c(\min)}$, for the section to be acceptable considering the factored load.

$$\phi_{c(\min)} = \frac{P_u}{P_n}$$

$$\phi_{c(\min)} = \frac{(626 \text{ kips})}{(1088 \text{ kips})}$$

$$\phi_{c(\min)} = 0.58$$

The neutral plane and drag force analysis was also performed for each candidate pile section using 100 percent toe mobilization. Factored structural resistance was subsequently evaluated considering the factored loads for group configurations shown in Table D-108. The pile penetration depth utilized for the structural resistance check was the required minimum penetration depth presented in Table D-126 (91 feet for all piles and group configurations at the South Abutment). It was again assumed that the probability of pile damage is reduced using the means discussed above. Therefore, a structural resistance factor in axial compression, ϕ_c , of 0.6 was applied to the nominal structural resistance for each candidate pile section. Table D-123 presents the ratio of the factored load to nominal structural resistance, at the neutral plane (also the pile toe), for all the candidate piles and group configurations.

Table D-123 Ratio of Factored Load to Nominal Structural Resistance in Axial Compression, $\phi_{c(\min)}$, at the Neutral Plane

Group Configuration	HP 10x42 $\phi_{c(\min)}$	HP 12x53 $\phi_{c(\min)}$	HP 12x74 $\phi_{c(\min)}$	HP 14x89 $\phi_{c(\min)}$	HP 14x117 $\phi_{c(\min)}$
1	0.99**	0.90**	0.64**	0.59	0.46
2	0.93**	0.84**	0.60**	0.56	0.43
3	0.88**	0.81**	0.58	0.54	0.41

The neutral plane is located at the pile toe for all sections and group configurations.

Note: ** - Section not acceptable considering ϕ_c equal or greater than 0.60.

Based on results of the drag force analysis, a candidate pile section may be eliminated from consideration if the factored loads are higher than the factored structural resistance. Table D-124 summarizes results of the drag force analysis.

The HP 12x74 pile section is acceptable in Group Configuration 3, while the HP 14x89 and HP 14x117 pile sections are acceptable in all three group configurations.

Table D-124 Does Candidate Pile Section Meet Structural Resistance Requirements Considering Drag Force at Minimum Pile Penetration Depth?

Group Configuration	HP 10x42	HP 12x53	HP 12x74	HP 14x89	HP 14x117
1	No	No	No	Yes	Yes
2	No	No	No	Yes	Yes
3	No	No	Yes	Yes	Yes

D.42 Decision 18: Does Estimated Total Settlement and Differential Settlement Between Adjacent Substructure Locations Satisfy Requirements and Angular Distortion Limits?

All substructures have now been preliminarily designed and the estimated vertical deformations computed for each location.

The vertical deformation limits and construction point concept detailed in Section 7.3 of Chapter 7 was used to first calculate tolerable differential settlement based upon angular distortion. Using Equation D-5, the angular distortion between substructure supports is limited to 0.004 radians, and for a 100 ft span on a multispan bridge, this equates to 4.8 inches of tolerable differential settlement.

Determine tolerable differential settlement, S_d , between substructure supports.

L_s = span length, 100 feet.

A = angular distortion limit, 0.004 radians (Table 7-18).

$$S_d = A * L_s \quad \text{[Eq. D-7]}$$

$$S_d = (0.004) * (100 \text{ feet}) * \left(\frac{12 \text{ inches}}{1 \text{ foot}}\right)$$

$$S_d = 4.8 \text{ inches}$$

Although this differential settlement is tolerable for angular distortion, project performance requirements limited the settlement at each substructure location to a maximum of 1.5 inches, the differential settlement between adjacent substructure locations limited to 1.0 inch, and a maximum angular distortion of 0.0008 radians. These requirements were established for rideability, drainage, and attached utility damage considerations. The estimated substructure performance is summarized in Table D-125.

Table D-125 Summary of Foundation Total Settlement, Differential Settlement, and Angular Distortion for HP 12x74 Pile Section

Substructure Location	Settlement Method	Total Settlement (inches)	Differential Settlement (inches)	Span Length (feet)	Angular Distortion
N. Abutment	N. Plane	1.17			
			0.18	100	0.0002
Pier 2	N. Plane	0.99			
			0.44	100	0.0004
S. Abutment	N. Plane	0.55			

As illustrated in Table D-125, the total settlement, differential settlement, and angular distortion requirements are all satisfied.

D.43 Block 19: South Abutment – Evaluate Economics of Candidate Piles, Preliminary Group Configurations, and Other Factors

The design process has served to compare strength and service limits for several candidate pile types within trial group configurations. Some candidate pile types have not met all of the strength, service, or drivability requirements. It is useful to quickly review the suitable and unsuitable pile types and group configurations and then assess the cost of the viable foundation solutions.

The number of initial candidate pile types has been reduced through evaluations of strength or drivability requirements while remaining candidate pile types are now subject to an economic analysis. Foundation cost and related economics are discussed in Section 3.4 of Chapter 3. Final selection of a pile type and group configuration is therefore based not only upon limit state requirements, but considers the associated cost.

Table D-126 summarizes the established minimum pile penetration depth based on analysis results from Blocks 12 through 15. For all candidate pile sections and group configurations at the South Abutment, the established minimum pile penetration depth was based on meeting tolerable vertical deformations.

Several candidate sections did not meet structural resistance requirements for axial loading, lateral loading or both. The candidate pile sections and/or group configurations not meeting design requirements are identified with an asterisk in Table D-126. For all three group configurations, the HP 10x42 and HP 12x52 pile sections did not meet all structural resistance requirements. The HP 12x74 section was also structurally unacceptable for Group Configurations 1 and 2. These candidate pile sections and group configuration were eliminated for the final design.

Table D-126 Established Minimum Pile Penetration Depth at the South Abutment

Group Configuration	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
1	91*	91*	91*	91*	91
2	91*	91*	91*	91	91
3	91*	91*	91	91	91

*Did not meet structural resistance requirement

Table D-127 presents the estimated minimum penetration depth for each candidate section to meet the factored geotechnical resistance requirements at the Strength I limit state. The larger pile sections require less pile penetration depth than the smaller pile sections to provide the same geotechnical resistance. In addition, the factored load per pile decreases from Group Configuration 1 to Group Configuration 3. However, from the analyses in Blocks 13 through 15, the established minimum penetration depth to preclude unacceptable vertical deformation requires all piles to be driven deeper than the depth needed solely for their factored geotechnical resistance. The minimum penetration depth requirement results in the additional geotechnical resistance gained by further pile embedment to be essentially wasted and therefore uneconomical.

Table D-127 Estimated Minimum Penetration Depth for Factored Geotechnical Resistance at Strength I Limit State

Group Configuration	HP 10x42 (feet)	HP 12x53 (feet)	HP 12x74 (feet)	HP 14x89 (feet)	HP 14x117 (feet)
1	91*	91*	91*	90*	88**
2	91*	90*	86*	74**	72**
3	91*	74*	73**	63**	62**

Note: * -Did not meet structural resistance requirement

Note: ** -Must be driven deeper to established minimum penetration depth (min. 91 feet).

Reference should be made to Block 19 of the North Abutment for a more detailed discussion of individual pile cost. Figure D-112 again presents the estimated individual pile cost versus pile penetration depth.

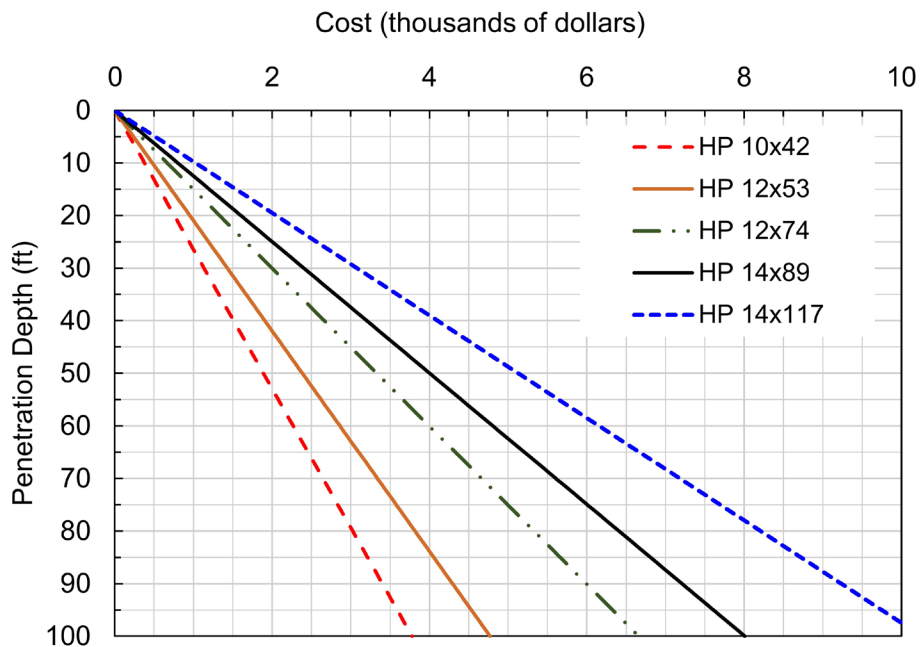


Figure D-112 Pile cost versus penetration depth.

For the South Abutment, Table D-128 shows the price per pile when considering the established minimum pile penetration depth. For example, for the HP 12x74 pile section in Group Configuration 3, the individual cost is determined by multiplying the cost per foot by the penetration depth. At the minimum pile penetration depth of 91 feet, the cost per pile is \$6,061. To note, there will be additional pile length embedded in the cap, however for pile cost estimation it can be considered negligible. Furthermore, Table D-129 shows the pile group cost reflecting the cost per pile and number of piles in the group.

Table D-128 Cost per Pile at Established Minimum Penetration Depth for Piles Meeting Structural Requirements

Group Configuration	HP 12x74	HP 14x89	HP 14x117
1	-	-	\$9,337
2	-	\$7,289	9,337
3	\$6,061	7,289	9,337

Table D-129 Pile Group Cost at Established Minimum Penetration Depth for Piles Meeting Structural Requirements

Group Configuration	Required Number of Piles	HP 12x74	HP 14x89	HP 14x117
1	18	-	-	\$168,059
2	22	-	\$160,360	205,405
3	26	\$157,576	189,517	242,752

Based upon this comparison, the HP 14x117 pile section proves to be least economical. Based upon pile group cost, the HP 14x89 pile section in Group Configuration 1 appears to be the most economical followed by the HP 12x74 pile section in Group Configuration 3. However, before selecting the lowest cost option considering solely the results in Table D-129, the cost of the pile cap should also be estimated and factored into the foundation cost, as discussed below.

Section 8.9 of Chapter 8 outlines a procedure to estimate the total pile cap thickness. Equation 8-80 is used to estimate this value along using the factored load per pile at the Strength limit state as previously presented in Table D-108.

A sample calculation is shown for the HP 12x74 pile in Group Configuration 3. Table D-130 summarizes the estimated cap thickness for each pile section and pile group permutation calculated using this procedure.

Estimate the total pile cap thickness.

P_{ui} = maximum single pile factored axial load, $Q = 238$ kips (Group Configuration 3, Table D-108).

$$t_{cap} = \frac{P_{ui}}{12} + 30 \quad [\text{Eq. 8-80}]$$

$$t_{cap} = \frac{(238 \text{ kips})}{12} + 30$$

$$t_{cap} = 50 \text{ inches}$$

Table D-130 Estimated Total Pile Cap Thickness

Group Configuration	HP 12x74 (inches)	HP 14x89 (inches)	HP 14x117 (inches)
1	-	-	59
2	-	53	53
3	50	50	50

Next, the volume of reinforced concrete required to construct the pile cap is determined from the estimated total pile cap thickness. The total pile cap width and length values were previously provided in Table D-106. The resulting volume of reinforced concrete for each pile section and pile group permutation is presented in Table D-131. Note that pile cap volume is shown in cubic yards (CY).

Table D-131 Estimated Volume of Reinforced Concrete in Pile Cap

Group Configuration	HP 12x74 (CY)	HP 14x89 (CY)	HP 14x117 (CY)
1	-	-	50.0
2	-	45.5	45.5
3	38.5	38.5	38.5

To estimate the pile cap cost, past pricing information is generally the best guide; however, similar to estimating the pile cost, due to fluctuations in the market price of material and other factors, pile cap costs are subject to change. The cost of the reinforced concrete pile cap, furnished and constructed, is estimated to be \$500 /CY. Using this estimated value, the volumes presented in Table D-131 were used to estimate the cost of the various reinforced concrete pile caps. The pile cap cost for each candidate section and group configuration is shown in Table D-132.

Table D-132 Estimated Cost of Reinforced Concrete Pile Cap

Group Configuration	HP 12x74	HP 14x89	HP 14x117
1	-	-	\$25,010
2	-	\$22,772	22,772
3	\$19,245	19,245	19,245

By adding the cost of the pile cap and piles for each permutation, the estimated total foundation cost is determined as presented in Table D-133. Additional construction costs should be considered and included, if applicable, such as excavations support, dewatering, and utility relocation or other environmental impacts. However, an exhaustive analysis of the other considerations is not presented in this example. Considering both the pile and the pile cap costs, the HP 12x74 section in the Group Configuration 3 is the most economical solution at the South Abutment followed by the HP 14x89 section in Group Configuration 2. Contractor equipment considerations, potential economies from use of only one pile section across the project, and other factors should be weighed in deciding between the HP 12x74 and HP 14 x 89 section use at the three substructure locations.

Table D-133 Estimated Foundation Cost Including Piles and Pile Cap at South Abutment

Group Configuration	HP 12x74	HP 14x89	HP 14x117
1	-	-	\$193,069
2	-	\$183,132	228,177
3	\$176,821	208,762	261,997

While the cost of pile over/underrun at other substructure locations was evaluated to account for inaccuracies in the assumed soil strength properties and estimated soil resistance, this is not the case at the South Abutment. Piles must be driven to bedrock (approximately 91 feet) at the South Abutment to satisfy a tolerable deformation, which effectively eliminates risk of length over/overrun differences between any candidate pile sections and a further economic assessment is therefore not needed.

D.44 Decision 20: Is the Preliminary Design of All Substructure Foundations Complete?

Yes, the preliminary foundation design has now been completed at all three substructure locations; North Abutment, Pier 2, and South Abutment.

D.45 Block 21: Refine Structural Modeling and Determine Loads at Foundation Top and Lateral Earth Pressure Loads on Abutments

The structure and foundation response to the loading cases is now further refined based on the structural and geotechnical analyses completed in Block 9 through

Block 17. The structural model is reanalyzed using the results from these preliminary substructure analyses to better define the structure loads at the foundation top, and lateral earth pressure loads on the abutments.

D.46 Decision 22: Did Loads Significantly Change, and Require Reevaluation of the Foundation Design?

No. The refined structural modeling determined that the structural loads provided in Block 11 are suitable for the final design.

D.47 Block 23: For Dynamic Elastic Analyses, Reevaluate Foundation Stiffnesses Using Unfactored Loads in Structural Model to get New Foundation Loads

Dynamic elastic analyses are not required since the structure is not subject to seismic events.

D.48 Decision 24: Did Loads Significantly Change, and Require Reevaluation of the Foundation Design?

Since dynamic analyses are not required, re-evaluation of the foundation design is not necessary.

D.49 Decision 25: Does the Design Meet All Limit State Requirements?

All strength, service, and extreme limit state requirements have been satisfied by one or more candidate piles and associated group configurations as demonstrated in Blocks 9 through 18.

To complete the final design, a candidate pile section and associated pile group configuration must be selected at each substructure location. Table D-134 summarizes the total foundation cost for HP 12x74, HP 14x89 and HP 14x117 at each respective substructure location. If one pile section is used at all substructure locations for the bridge, the lowest cost foundation selection is the HP 12x74 at \$367,043 followed by the HP 14x89 at \$369,674. The most economical solution is to use HP 12x74 H-piles at both abutments and HP 14x89 H-piles at Pier 2. This solution has an estimated foundation cost of \$357,434.

Table D-134 Estimated Total Foundation Cost at All Substructures Locations on the Southbound Bridge

Location	Most Economical HP 12x74 Cost	Associated HP 12x74 Group Configuration	Most Economical HP 14x89 Cost	Associated HP 14x89 Group Configuration	Most Economical HP 14x117 Cost	Associated HP 14x117 Group Configuration
North Abutment	\$96,192	1	\$102,121	1	\$122,145	1
Pier 2	94,030	5	84,421	4	104,671	4
South Abutment	176,821	3	183,132	2	193,069	1
Total	367,043	-	369,674	-	419,885	-

Although the results of the above economic assessment suggests the final design use different H-pile sections at different substructure locations, the \$9,609 savings in the estimated foundation cost is likely insufficient to address potential increased costs from varying installation equipment or accessories as well as pile quantity discounts. Therefore, the final design will be based on the HP 12x74 section for the entire structure.

D.50 Block 26: Design Pile Caps and Abutments

Preliminary design of pile caps is discussed in Section 8.9. Preliminary pile cap design was performed for this design example in Block 13. Final pile cap and abutment design is now performed by the structural engineer and is not detailed herein.

D.51 Block 27: Finalize Plans and Specifications Including Pile Quantities, Minimum Pile Penetration Requirements from Blocks 13 through 15, and Required Nominal Resistances

Plans and specifications are finalized based upon the analyses and results from the previous design blocks. A summary of the final design is presented in Table D-135. At Pier 2, Group Configuration 6 was selected over Group Configuration 5 to reduce the risk of pile damage from driving piles through the dense gravel with cobbles to the bedrock. This decision increases the estimated final foundation cost from \$367,043 to \$373,492. This modest increase is justified by the replacement and redesign costs anticipated to accommodate a damaged pile.

Table D-135 Final Design Foundation Summary and Associated Penetration Depths for the Southbound Bridge

Location	Group Configuration	Estimated Penetration Depth for Axial Compression (feet)	Minimum Penetration Depth for Axial Tension (feet)	Minimum Penetration Depth for Lateral Deformation (feet)	Minimum Penetration Depth for Vertical Deformation (feet)	Pile Section Drivability Limit (feet)
North Abutment	1	45	28	25	60*	62
Pier 2	6	40	0	20	56*	60
South Abutment	3	73	18	35	91*	91

Note: * - Penetration depth controls the foundation design.

Plans and specifications are developed detailing the factored load, the nominal resistance determination method, the required nominal driving resistance, and associated estimated or minimum pile penetration depth estimates for strength or deformation requirements. As decided earlier in the design, dynamic testing with signal matching of at least 2% of the piles per location will be used for the resistance determination method in the field.

D.51.1 Required Nominal Resistance at the North Abutment

At the North Abutment, the required factored load per pile at the Strength I limit state is 323 kips (Table D-14 of Block 10). Therefore the minimum required factored resistance in axial compression is 323 kips. Using the AASHTO resistance factor for nominal resistance determination by dynamic testing, $\phi_{dyn} = 0.65$, the nominal resistance per pile is calculated with Equation 7-3.

R_r = required factored resistance per pile, 323 kips (Table D-14).

ϕ_{dyn} = resistance factor (based on the static analysis method).

$$R_r \leq \phi_{dyn} R_n \quad [\text{Eq. 7-3}]$$

Rearranging terms for R_n :

$$R_n \geq \frac{R_r}{\phi_{dyn}} \quad [\text{Eq. 7-3}]$$

$$R_n \geq \frac{(323 \text{ kips})}{(0.65)}$$

$$R_n \geq 497 \text{ kips}$$

The nominal resistance, R_n , must therefore be greater than 497 kips per pile and this value is included on plan documents. As noted in Table D-6 and Figure D-9, for the HP 12x74 section in Group Configuration 1, the estimated depth to achieve this nominal resistance is 45 feet below the bottom of pile cap.

At the North Abutment, a limited amount of soil setup may also be considered in establishing the driving criteria. Figure D-21 illustrates the estimated soil setup at a depth of 45 feet is 35 kips. However, for tolerable deformations, a minimum pile penetration depth of 60 feet was specified. No additional setup occurs in the soils below 45 feet so the estimated setup at 60 feet is also 35 kips. Hence, production piles achieving a nominal driving resistance, R_{ndr} , of 462 kips should attain the required nominal resistance of 497 kips during restrike. Based on the geomaterials, a restrike test interval of 1 day is specified. The required nominal driving resistance, R_{ndr} , of 462 kips, the nominal resistance, R_n , of 497 kips, the minimum penetration depth, and restrike interval are included on plan documents.

D.51.2 Required Nominal Resistance at Pier 2

At Pier 2, the required factored load per pile at the Strength V limit state is 257 kips (Table D-61 of Block 10). Therefore the minimum required factored resistance in axial compression is 257 kips. Using the AASHTO recommended resistance factor $\phi_{dyn}=0.65$, the nominal resistance per pile to be determined from field determination is calculated with Equation 7-3.

R_r = required factored resistance per pile, 257 kips (Table D-61).
 ϕ_{dyn} = resistance factor (based on the static analysis method).

$$R_r \leq \phi_{dyn} R_n \quad [\text{Eq. 7-3}]$$

Rearranging terms for R_n :

$$R_n \geq \frac{R_r}{\phi_{dyn}} \quad [\text{Eq. 7-3}]$$

$$R_n \geq \frac{(257 \text{ kips})}{(0.65)}$$

$$R_n \geq 395 \text{ kips}$$

The nominal resistance, R_n , must therefore be greater than 395 kips per pile (this value is included on plan documents). As noted in Table D-63 and Figure D-41, for

the HP 12x74 section in Group Configuration 6, the minimum estimated depth to achieve this nominal resistance is 40 feet from the bottom of pile cap.

At Pier 2, more soil setup is anticipated due to the cohesive layer. Table D-52 and Table D-56 indicate the estimated soil setup at a depth of 40 feet is 71 kips. However, for tolerable deformations, a minimum pile penetration depth of 56 feet was specified. No additional setup occurs in the soils below 40 feet so the estimated setup at 60 feet is also 71 kips. Hence, production piles achieving a nominal driving resistance, R_{ndr} , of 324 kips should attain the required nominal resistance of 395 kips during restrike. Based on the geomaterials and project size, a restrike test interval of 3 days is specified. The required nominal driving resistance, R_{ndr} , of 324 kips, the nominal resistance, R_n , of 395 kips, the minimum penetration depth, and the restrike interval are included on plan documents.

D.51.3 Required Nominal Resistance at the South Abutment

At the South Abutment, the required factored load per pile at the Strength I limit state is 238 kips (Table D-108 of Block 10). Therefore the minimum required factored resistance in axial compression is 238 kips. Using the AASHTO recommended resistance factor $\phi_{dyn}=0.65$, the nominal resistance per pile to be determined from field determination is calculated with Equation 7-3.

R_r = required factored resistance per pile, 238 kips (Table D-108).
 ϕ_{dyn} = resistance factor (based on the static analysis method).

$$R_r \leq \phi_{dyn} R_n \quad [\text{Eq. 7-3}]$$

Rearranging terms for R_n :

$$R_n \geq \frac{R_r}{\phi_{dyn}} \quad [\text{Eq. 7-3}]$$

$$R_n \geq \frac{(238 \text{ kips})}{(0.65)}$$

$$R_n \geq 366 \text{ kips}$$

The nominal resistance, R_n , must therefore be greater than 366 kips per pile (this value is included on plan documents). As noted in Table D-100 and Figure D-85, for the HP 12x74 section in Group Configuration 3, the minimum estimated depth to achieve this nominal resistance is 73 feet from the bottom of pile cap (Table D-109).

At the South Abutment, a significant amount of soil setup is anticipated. The nominal resistances in Table D-100 and Table D-103 indicate the estimated soil setup at a depth of 73 feet is 167 kips. However, for tolerable deformations, a minimum pile penetration depth of 91 feet was specified. This depth corresponds to the interface between the silty clay and bedrock layer. The estimated setup in the soil profile to 91 feet is 210 kips. Hence, production piles achieving a nominal driving resistance, R_{ndr} , of 156 kips should attain the required nominal resistance of 366 kips with time. In clay soils, restrikes are often performed 1 week after initial driving. However, to satisfy the tolerable deformation requirements the piles will be driven to hard rock, the piles will have more than sufficient nominal resistance without setup. Hence, waiting 1 week to conduct restrike tests is not realistic or needed. Based on the geomaterials and project size, a restrike test interval of 1 day is specified. The required nominal driving resistance, R_{ndr} , of 156 kips, the nominal resistance, R_n , of 366 kips, the minimum penetration depth, and the restrike interval are included on plan documents.

D.52 Block 28: Perform Evaluation of Contractor's Proposed Equipment

Project specifications require the Contractor to provide documentation for the pile driving hammer to be used. The drive system submittal form shown in Figure 15-50 of Chapter 15 is typically completed by the Contractor, and provided to the Engineer. A preconstruction wave equation is then performed considering the contractor's specific hammer, selected pile section and respective soil model.

The Contractor has submitted an ICE I-36 V2 single acting diesel hammer for hammer approval. Figure D-113 presents a copy of the hammer submittal form.

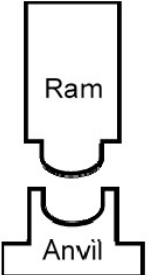


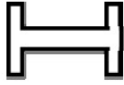



Contractor No.: <u>105589</u>		Structure Name and/or No.: <u>Freedom Bridge</u>	
Project: <u>Sunrise Expressway</u>		Pile Driving Contractor or Subcontractor: <u>CJG Construction</u>	
County: <u>Sussex</u>		(Piles driven by)	
	Hammer	Manufacturer: <u>ICE</u>	Model No.: <u>I-36v2</u>
		Hammer type: <u>Single Acting Diesel</u>	Serial No.: <u>16456</u>
		Manufacturer Maximum Rated Energy: <u>93.74</u> (kip-ft)	
		Stroke at Maximum Rated Energy: <u>11.81</u> (ft)	
		Range in Operating Energy: <u>67.7</u> to <u>93.74</u> (kip-ft)	
		Range in Operating Stroke: <u>8.5</u> to <u>11.81</u> (ft)	
		Ram Weight: <u>7.94</u> (kips)	
		Modifications: <u>N/A</u>	
		<hr/>	
	Striker Plate	Weight: <u>N/A</u> (kips)	Diameter: _____ (in)
		Thickness: _____ (in)	
		<hr/>	
	Hammer Cushion	Material#1	Material#2 (for composite cushion)
		Name: <u>Nylon</u>	Name: _____
		Area: <u>490.87</u> (in ²)	Area: _____ (in ²)
		Thickness/Plate: <u>4</u> (in)	Thickness/Plate: _____ (in)
		No. of Plates: <u>1</u>	No. of Plates: _____
		Total Thickness of Hammer Cushion: <u>4</u>	
		<hr/>	
	Helmet	Weight: <u>3.338</u> (kips)	
	Insert (If Any)	Weight: <u>N/A</u> (kips)	
		Total Weight of Helmet and Insert: <u>3.338</u> (kips)	
		<hr/>	
	Pile Cushion	Material: <u>N/A</u>	
		Area: _____ (in ²)	Thickness/Sheet: _____ (in ²)
		No. of Sheets: _____	
		Total Thickness of Pile Cushion: _____	
		Maximum Thickness Accommodated by Helmet: _____ (in)	
		<hr/>	
	Pile	Pile Type: <u>HP 12x74</u>	
		Wall Thickness: <u>0.61</u> (in)	Taper: <u>N/A</u>
		Cross Sectional Area: <u>21.8</u> (in ²)	Weight/ft: <u>N/A</u>
		Ordered Length: <u>90</u> (ft)	
		Factored Resistance: <u>N/A</u> (kips)	
		Nominal Resistance: <u>varies by substructure</u> (kips)	
		Description of Splice: <u>N/A</u>	
		<hr/>	
		Driving Shoe/Closure Plate Description: <u>N/A</u>	
		<hr/>	
		Submitted By: <u>Christian J. Logan</u>	Date: <u>11/8/2012</u>
		Telephone No.: _____	Email: _____

Figure D-113 Contractor's ICE I-36 V2 hammer submittal form.

D.52.1 Wave Equation for the North Abutment

At the North Abutment, the design stage drivability file was modified to evaluate the Contractor's proposed driving system. The drivability results, presented in Figure D-114, show that the ICE I-36 V2 appears suitable to drive the HP 12x74 pile section to the required minimum penetration depth of approximately 60 feet, within specified blow count range of 30 to 120 blows per foot, and with driving stresses below the maximum driving stress limit of 45 ksi.

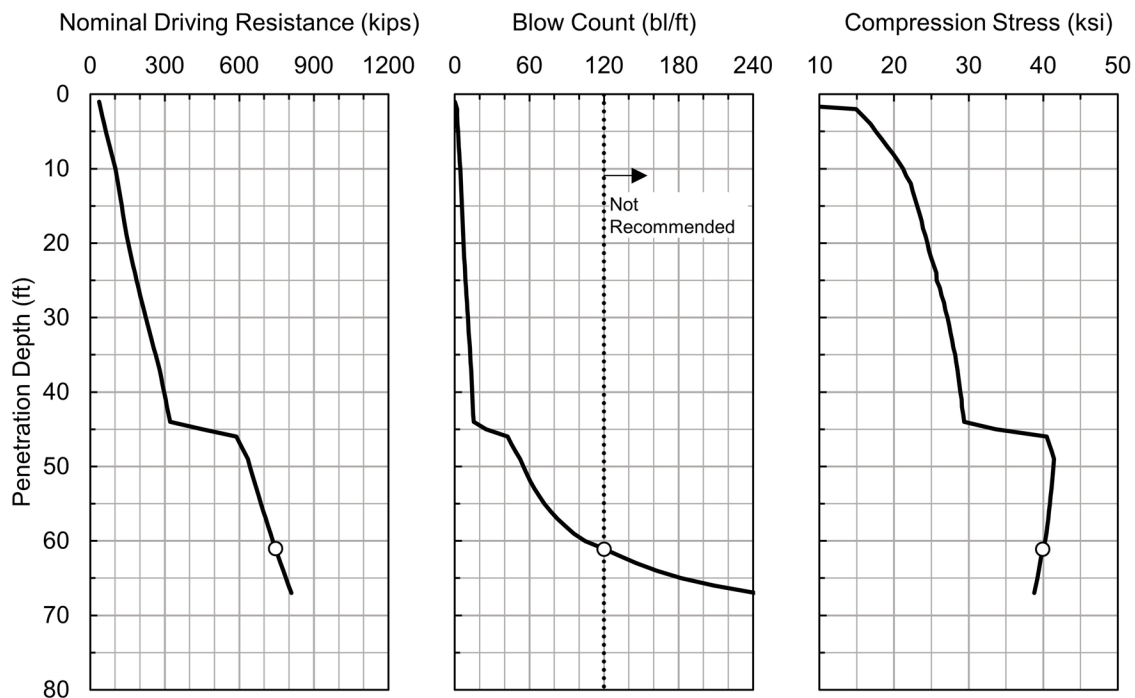


Figure D-114 Preliminary drivability with Contractor's hammer at the North Abutment.

A wave equation bearing graph was also developed for the proposed driving system and HP 12x74 pile section at the required minimum pile penetration depth of 60 feet. The plot of compression stress and tension stress versus blow count is shown as Figure D-115. Driving stresses are less than the 45 ksi specification limit. Figure D-116 presents the bearing graph of the nominal resistance versus blow count, and the calculated hammer stroke versus blow count. In this, it is important to note that the required nominal driving resistance of 462 kips as well as the nominal resistance of 497 kips are well within the blow count range of 30 to 120 blows per foot.

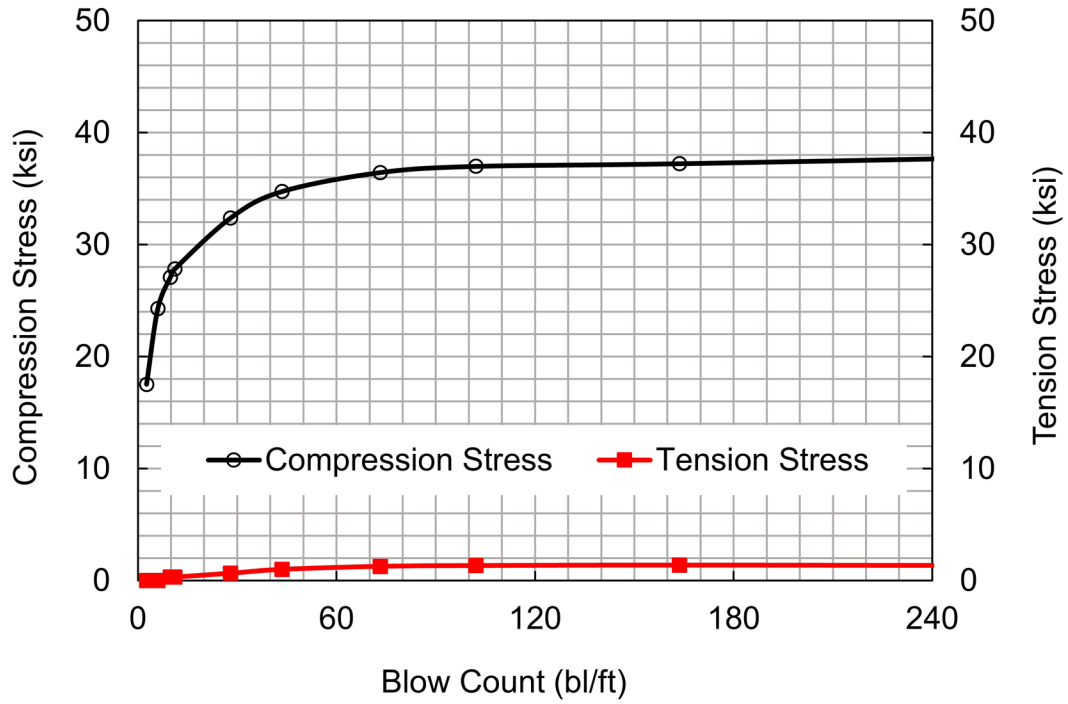


Figure D-115 Compression and tension stress versus blow count.

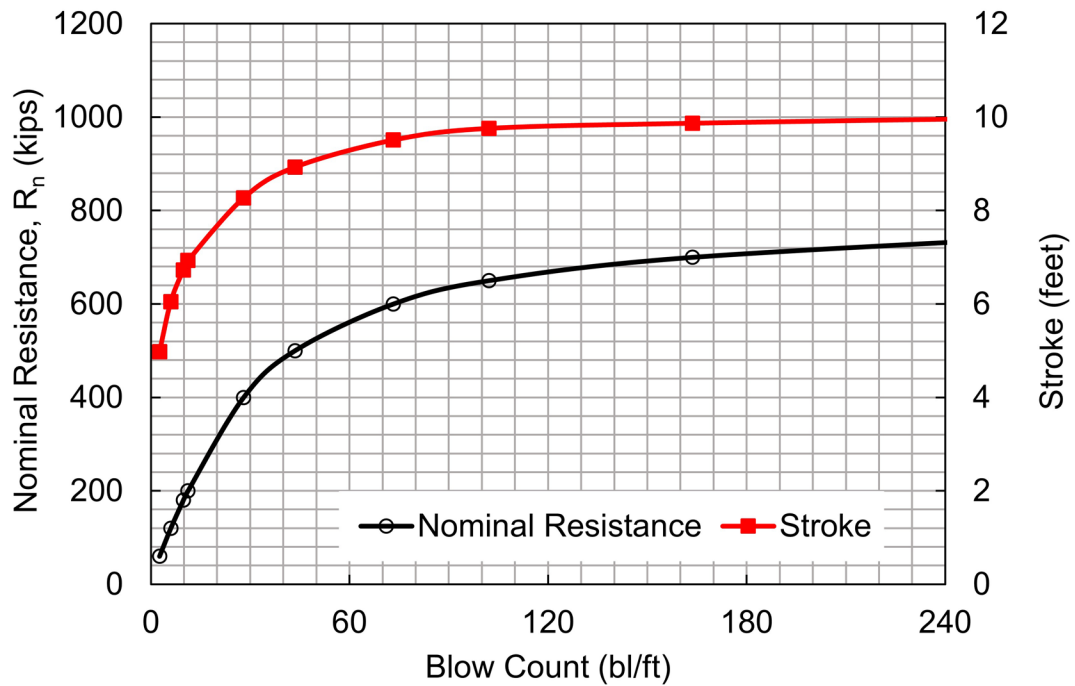


Figure D-116 Nominal resistance and stroke versus blow count.

Figure D-117 provides a preliminary Inspectors Chart of hammer stroke versus blow count for the ICE I-36 V2 and a nominal driving resistance, R_{ndr} , of 462 kips. This plot provide pile inspector with an additional tool to determine the necessary blow count at the observed hammer stroke. For example, the 462 kip nominal driving resistance is achieved at 62 blows per foot with a hammer stroke of 6 feet and at a blow count of 28 blows per foot at a hammer stroke of 11 feet.

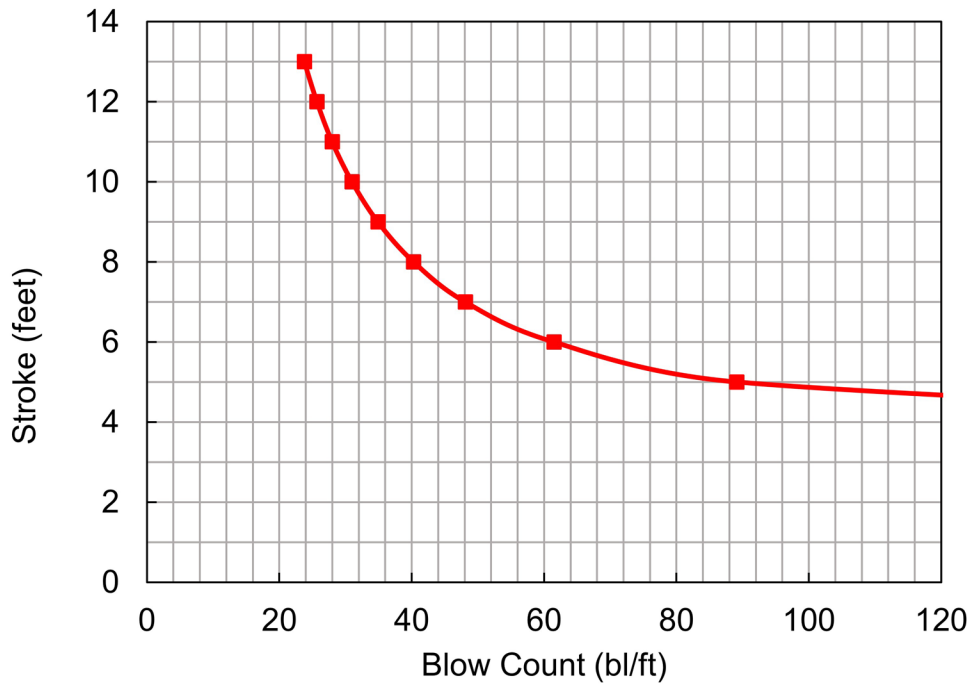


Figure D-117 Inspectors Chart for ICE I-36 V2 at the North Abutment.

D.52.2 Wave Equation for Pier 2

At the Pier 2, the design stage drivability file was again modified to evaluate the Contractor's proposed driving system. The drivability results presented in Figure D-118 show that the ICE I-36 V2 appears suitable to drive the HP 12x74 pile section to the required minimum penetration depth of approximately 56 feet, within specified blow count range of 30 to 120 blows per foot, and with driving stresses below the maximum driving stress limit of 45 ksi.

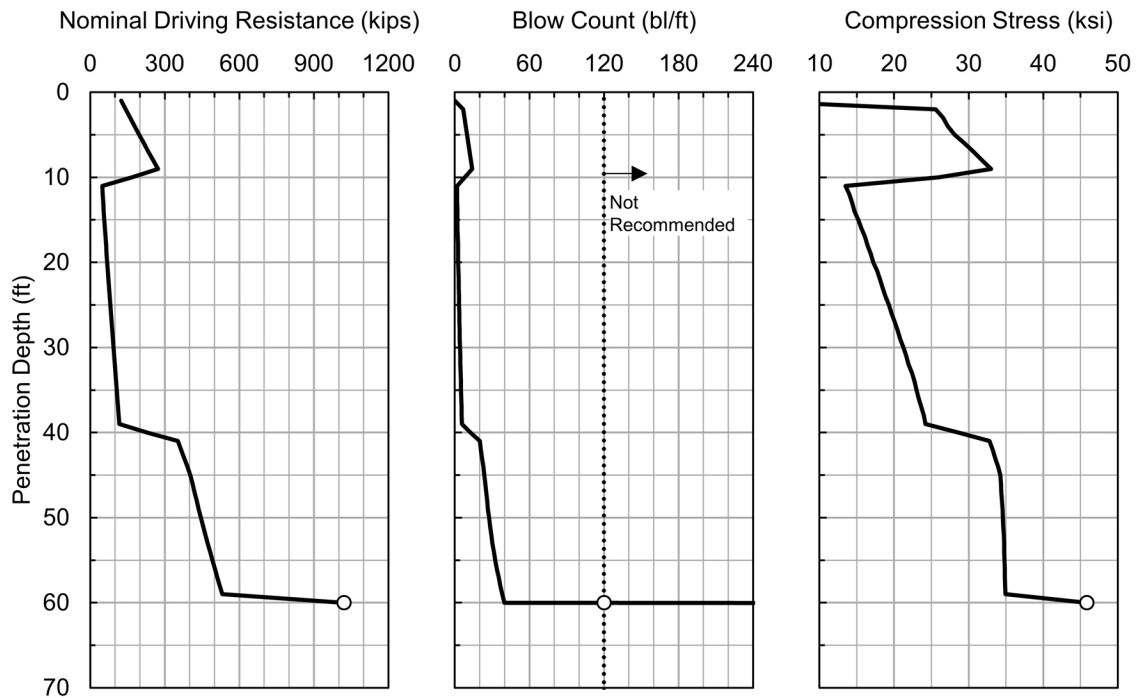


Figure D-118 Drivability with Contractor's hammer at Pier 2.

A wave equation bearing graph was also developed for the proposed driving system and HP 12x74 pile section at the required minimum pile penetration depth of 56 feet. The plot of compression stress and tension stress versus blow count is shown as Figure D-119. Driving stresses are less than the 45 ksi specification limit. Figure D-120 presents the bearing graph of the nominal resistance versus blow count, and the calculated hammer stroke versus blow count. In this, it is important to note that the required nominal driving resistance of 324 kips as well as the nominal resistance of 395 kips are within slightly less than acceptable blow count range of 30 blows per foot. However, the analysis was performed using the maximum fuel setting. If it is desired to have the blow count above 30 blows per foot at the nominal driving resistance, a reduced fuel setting could be used. Also, the nominal driving

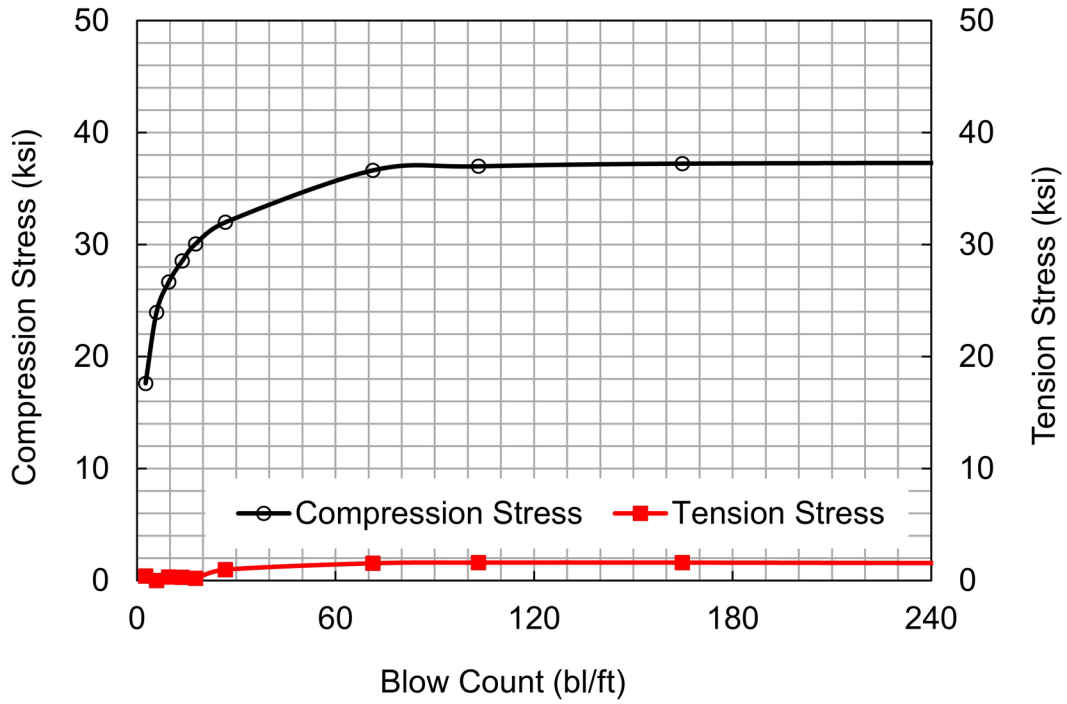


Figure D-119 Compression and tension stress versus blow count.

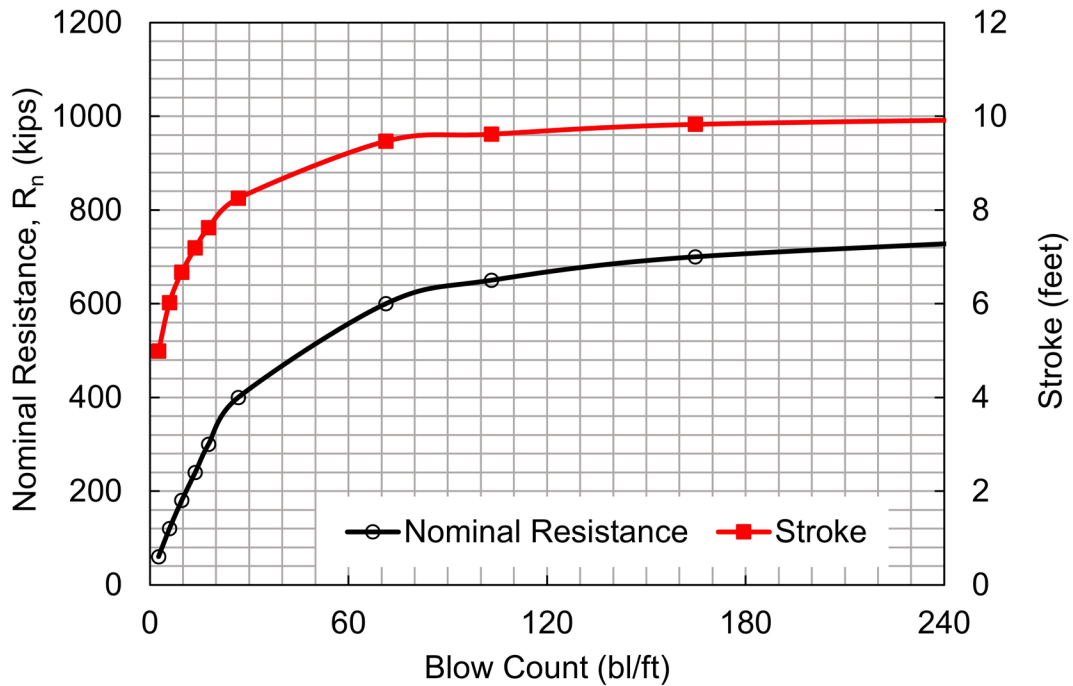


Figure D-120 Nominal resistance and stroke versus blow count.

resistance is expected to be on the order of 500 kips at the minimum pile penetration depth where the blow count is above 30 blows per foot.

Figure D-121 provides a preliminary Inspectors Chart of hammer stroke versus blow count for the ICE I-36 V2 and a nominal driving resistance, R_{ndr} , of 324 kips. This plot provide pile inspector with an additional tool to determine the necessary blow count at the observed hammer stroke. For example, the 324 kip nominal driving resistance is achieved at 37 blows per foot with a hammer stroke of 5 feet and at a blow count of 20 blows per foot at a hammer stroke of 9 feet.

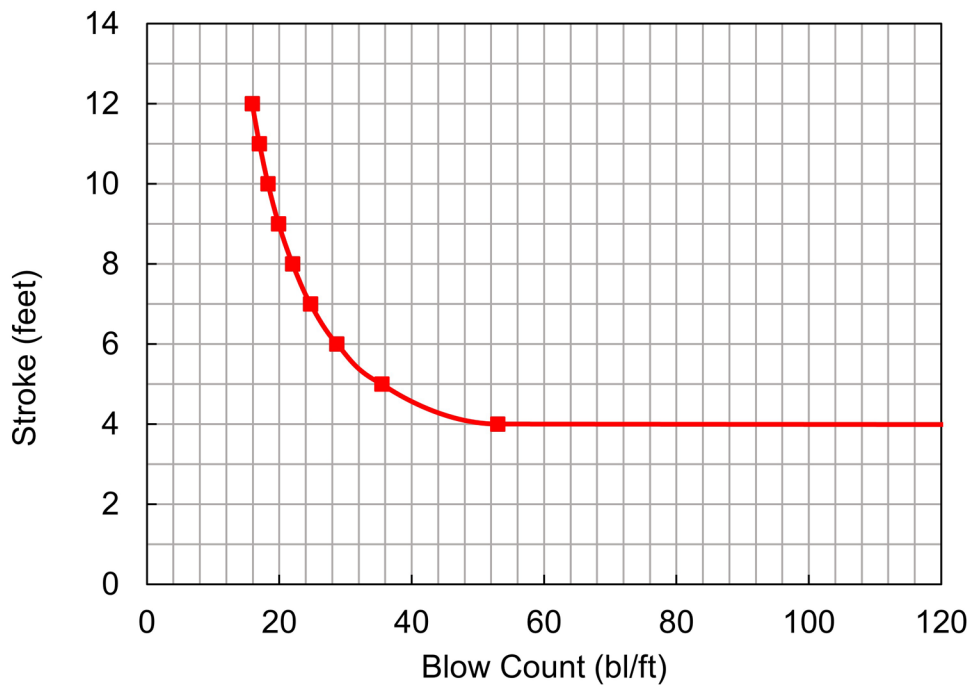


Figure D-121 Inspectors Chart for ICE I-36 V2 at Pier 2.

D.52.3 Wave Equation for the South Abutment

At the South Abutment, the design stage drivability file was revisited and modified to evaluate the Contractor's proposed driving system. The drivability results, presented in Figure D-122 show that the ICE I-36 V2 appears suitable to drive the HP 12x74 pile section to the required minimum penetration depth of approximately 91 feet, within specified blow count range of 30 to 120 blows per foot, and with driving stresses below the maximum driving stress limit of 45 ksi.

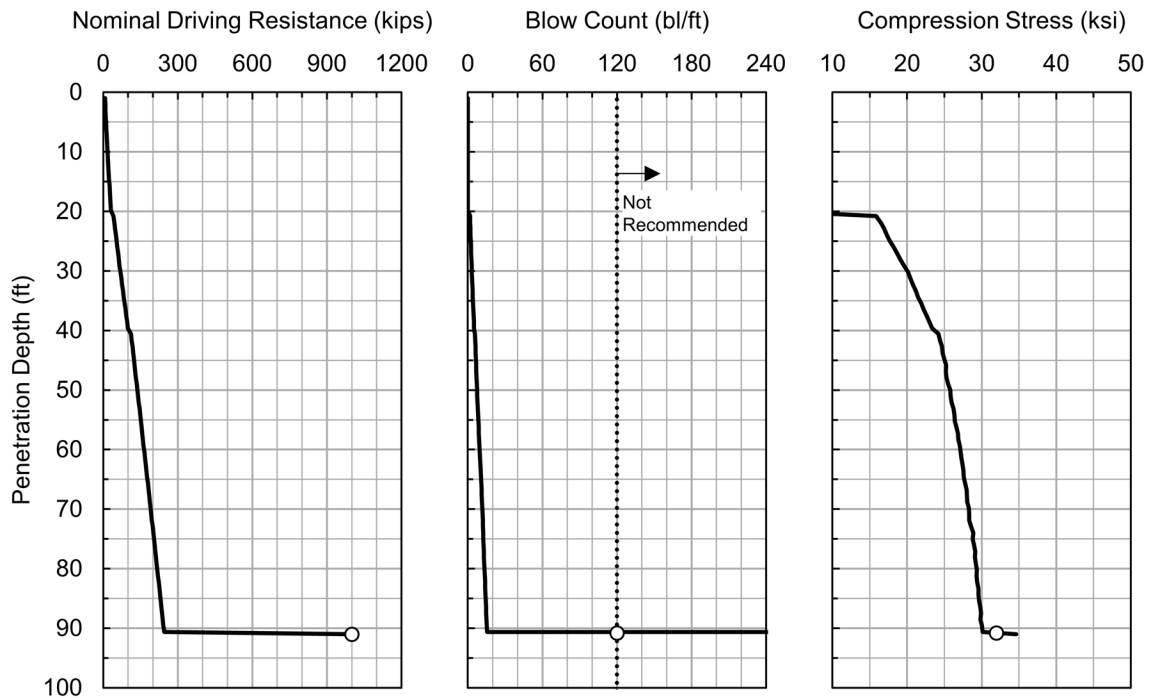


Figure D-122 Drivability with Contractor's hammer at the South Abutment.

A wave equation bearing graph was also developed for the proposed driving system and HP 12x74 pile section at the required minimum pile penetration depth of 91 feet. The plot of compression stress and tension stress versus blow count is shown as Figure D-123. Driving stresses are less than the 45 ksi specification limit. Figure D-124 presents the bearing graph of the nominal resistance versus blow count, and the calculated hammer stroke versus blow count. In this, it is important to note that the required nominal driving resistance of 156 kips as well as the nominal resistance of 366 kips are below the recommended blow count range of 30 to 120 blows per foot. However, the analysis was performed using the maximum fuel setting. If it is desired to have the blow count above 30 blows per foot at the nominal driving resistance, a reduced fuel setting could be used. It should also be noted that the

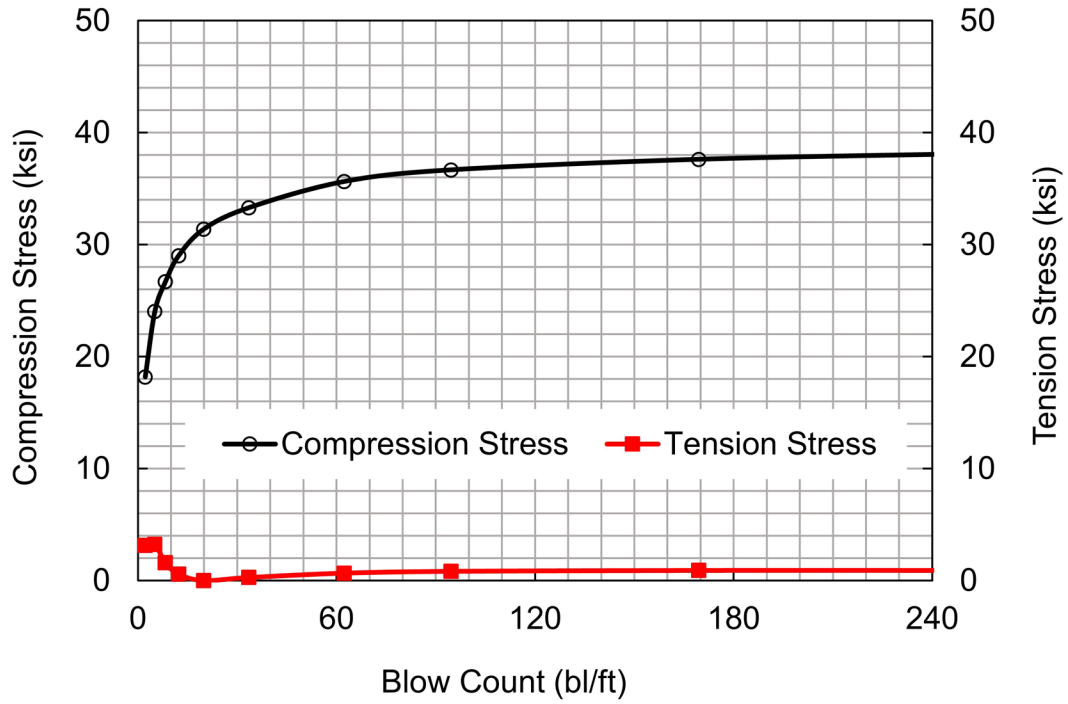


Figure D-123 Compression and tension stress versus blow count.

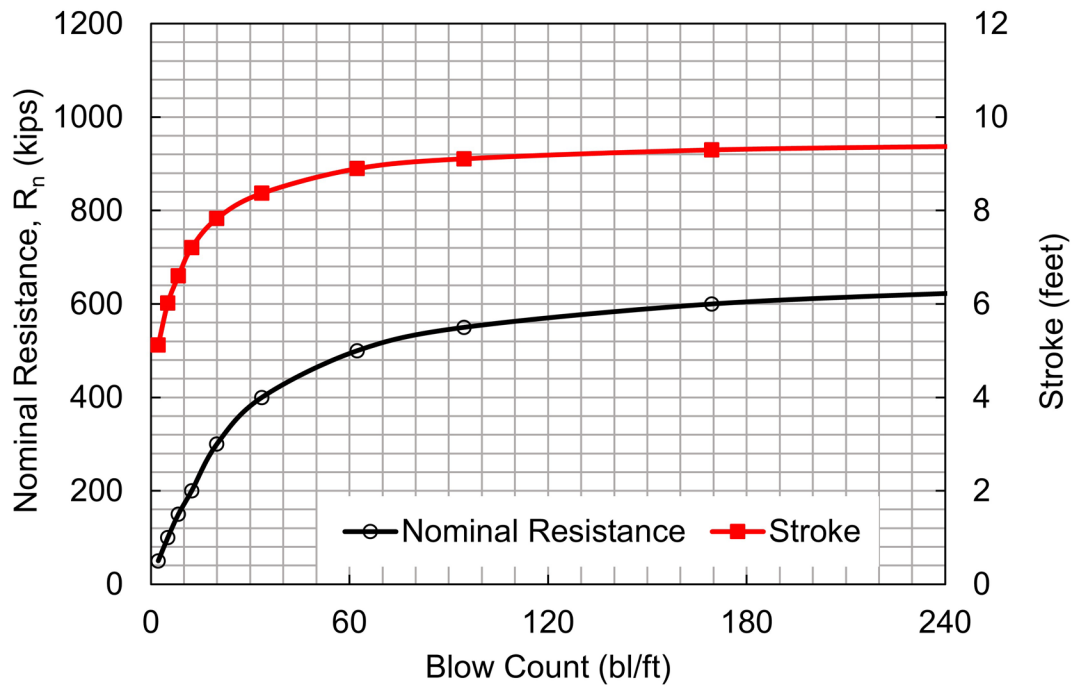


Figure D-124 Nominal resistance and stroke versus blow count.

nominal driving resistance will quickly transition from approximately 240 to 250 kips above rock to well in excess of 900 kips on rock. Hence, hammer operation at a reduced fuel setting is preferred to better control driving stresses.

Figure D-125 is a preliminary Inspectors Chart for the IC I-36 V2. For the nominal driving resistance, R_{ndr} , of 156 kips, Figure D-125 plots the stroke and respective blow count determined from the wave equation analysis. This chart provides the inspector with an additional tool to compare the relative stroke and penetration resistance for a hammer varies in performance. For example, 156 kips of nominal resistance is achieved at 8 blows/ft with a hammer stroke of 6 feet and also at 11 blows/ft with a hammer stroke of 4 feet.

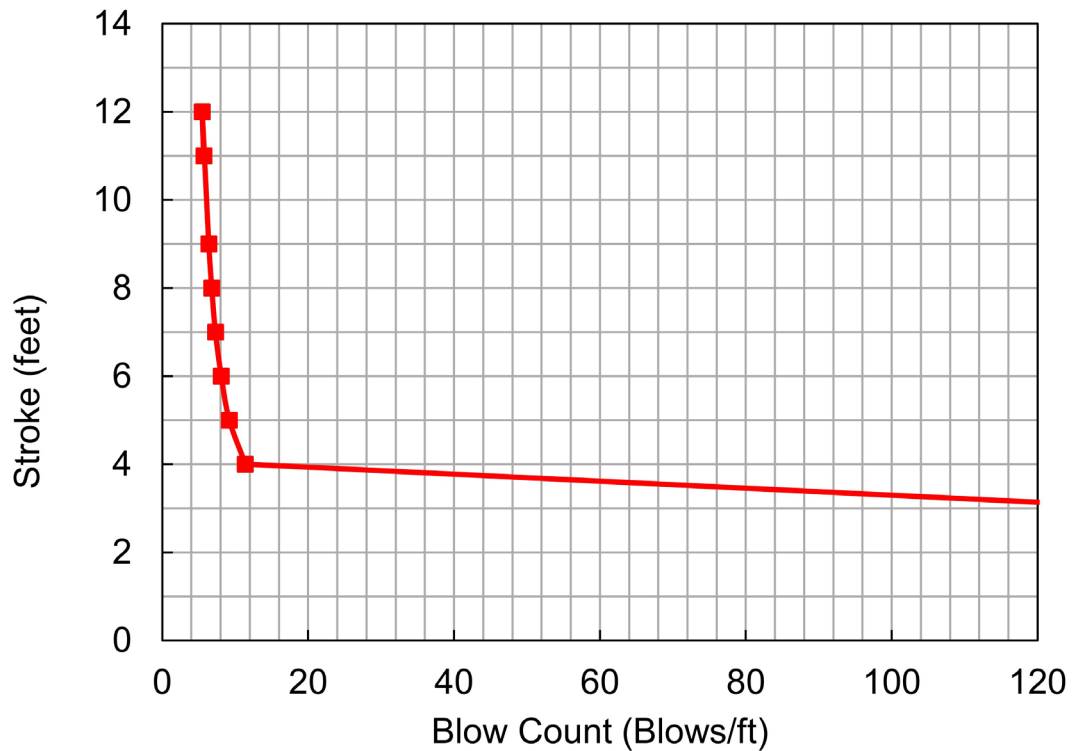


Figure D-125 Inspectors Chart for ICE I-36 V2 at the South Abutment.

D.53 Block 29: Set Preliminary Driving Criteria, Drive Test Pile(s) and Assess Constructability

The required nominal driving resistance at each substructure location was determined in Block 27. These nominal driving resistances consider the selected resistance determination method as well as changes in the long term geotechnical resistance due to soil setup, scour, or construction activities. The wave equation analyses performed for each substructure location in Block 28 will be used to set the preliminary driving criteria.

Two test piles are driven at production pile locations for each substructure. Dynamic monitoring is performed on these two piles during initial driving and again during restrike after an appropriate waiting period. Signal matching analysis is also performed on the initial driving and restrike dynamic test results to determine the magnitude of soil setup. The dynamic testing and signal matching results substantiate the constructability of the design..

D.54 Block 30: Adjust Driving Criteria or Design

Refined wave equation analyses are now performed based on the dynamic test results. The refined wave equation analyses establish the production pile installation criteria at each substructure location. The required nominal driving resistance considers the magnitude of soil setup substantiated by the test pile monitoring during initial driving and restrike. The test pile results confirmed that design modifications are not required.

D.55 Block 31: Drive Production Piles with Construction Monitoring, Resolve any Pile Installation Problems

Production piles are installed using the final driving criteria. Pile installation is documented in accordance with established quality control procedures.

D.56 Block 32: Perform Post-Construction Evaluation and Refinement For Future Designs

After completion of the foundation construction, the design is reviewed and evaluated for its ability to satisfy the design requirements cost effectively.