AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS
AASHTO
THE VOICE OF TRANSPORTATION

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ERRATA

Dear Customer:

Due to errors found after the publication had been completed, AASHTO has reprinted the pages listed below and made the following errata changes to the *AASHTO LRFD Bridge Design Specifications,* Fifth Edition:

Page No(s).	Affected Article	Errata Change to
Front Matter		
p. iii/p. iv	Contents	Revise titles of Articles 10.7.3.8, 10.7.3.12, and 10.7.7
Section 10—Foundations		
p. 10-45/p. 10-46	Article 10.5.5.2.3	Table 10.5.5.2.3-1: Revise rows 4, 5, 7 Add table note Table 10.5.5.2.3-1 (cont'd.): Revise rows 2, 4, 5, 6 Delete Table 10.5.5.2.3-2
p. 10-47/p. 10-48	Article 10.5.5.2.3	Delete Table 10.5.5.2.3-3
	Article 10.5.5.2.4	Paragraph 2: Revise sentence 1 Paragraph 3: Revise sentence 1
	Article C10.5.5.2.4	Add new paragraph 2 Paragraph 3 (was paragraph 2): Delete sentences 1 and 2 Paragraph 3 (was paragraph 2): Revise sentences 3 and 4 Paragraph 3 (was paragraph 2): Delete rest of paragraph 3 Paragraph 4 (was paragraph 3): Revise sentences 1, 3, and 4 Old paragraph 5: Delete entirely, including Steps 1–3
p. 10-49/p. 10-50	Article C10.5.5.2.4	Paragraph 7 (was paragraph 9): Delete sentences 4, 6, and 8 Paragraph 7 (was paragraph 9): Revise sentence 7 Paragraph 7 (was paragraph 9): Add sentence to end of paragraph 7 Paragraph 8 (was paragraph 10): Revise sentence 1

Page No(s).	Affected Article	Errata Change to
p. 10-49/p. 10-50	Article 10.5.2.2.4	Table 10.5.2.2.4-1: Revise row 17
p. 10-81/p. 10-82	Article 10.7.1.1	Revise paragraph 1 (only one sentence) Revise bullets 2, 4, and 5
p. 10-83/10-84	Article C10.7.1.3	Revise paragraph 2 (only one sentence)
	Article 10.7.1.4	Revise Sentences 1 and 2 (only one paragraph)
	Article C10.7.1.4	Paragraph 1: Revise sentences 2 and 3 Revise paragraph 2 (only one sentence)
	Article 10.7.1.5	Revise bullets 1, 3, and 7
	Article 10.7.1.6.2	Revise paragraph 1 (only one sentence) Paragraph 3: Revise sentence 2 Revise paragraph 4 (only one sentence)
	Article C10.7.1.6.2	Paragraph 2: Revise sentence 2 Revise paragraph 4 (only one sentence)
p. 10-85/p. 10-86	Article C10.7.1.6.4	Revise sentences 1 and 2 (only one paragraph)
	Article C10.7.2.3.1	Revise paragraph 2 (only one sentence) Revise paragraph 3 (only one sentence)
	Article 10.7.2.3.1	Table 10.7.2.3.1-1: Revise caption
p. 10-87/p. 10-88	Article 10.7.2.4	Paragraph 2: Revise sentence 2
	Article C10.7.2.4	Paragraph 1: Revise sentence 1 Paragraph 2: Revise sentences 2 and 4 Paragraph 4: Revise sentence 1
p. 10-89/p. 10-90	Article 10.7.2.4	Table 10.7.2.4-1: Revise caption and row 2 Paragraph 7: Revise sentence 2
	Article C10.7.2.4	Paragraph 7: Revise sentence 1 Paragraph 8: Revise sentences 1 and 2
	Article 10.7.2.5	Paragraph 1: Revise sentences 1 and 3
	Article C10.7.2.5	Revise paragraph 1 (only one sentence) Paragraph 2: Revise sentences 1-4
	Article C10.7.2.6	Revise (only one paragraph, one sentence)
	Article 10.7.3.1	Revise bullet 2
	Article C10.7.3.1	Paragraph 1: Revise sentence 1
p. 10-91/p. 10-92	Article 10.7.3.1	Revise bullet 6
	Article C10.7.3.1	Paragraph 1: Revise sentence 3
	Article 10.7.3.2.1	Revise (only one paragraph, one sentence)
	Article C10.7.3.2.1	Paragraph 2: Add new sentence 2
	Article 10.7.3.2.2	Revise (only one paragraph, one sentence)
	Article 10.7.3.2.3	Revise sentences 2 and 4 (only one paragraph)

Page No(s).	Affected Article	Errata Change to
p. 10-91/p. 10-92	Article C10.7.3.2.3	Paragraph 2: Revise sentences 1–3 Revise bullets 1–4 Revise paragraph 3 (only one sentence)
	Article 10.7.3.3	Revise sentences 1–3
	Article C10.7.3.3	Paragraph 1: Revise sentence 1 and add new sentences 2 and 3 Paragraph 2: Add new sentence 1 and revise sentence 2 (was sentence 1) Paragraph 3: Revise sentence 2 and add new sentence 3 Paragraph 4: Revise sentence 1
p. 10-93/p. 10-94	Article C10.7.3.3	Paragraph 5: Add new sentence 3 Add new paragraph 7
	Article C10.7.3.4.1	Revise paragraph 1 (only one sentence) Paragraph 2: Revise sentence 1
	Article C10.7.3.4.2	Revise sentence 3 (only one paragraph)
	Article C10.7.3.4.3	Paragraph 3: Revise sentences 1 and 2 Revise paragraph 4 (only one sentence)
p. 10-95/p. 10-96	Article C10.7.3.4.3	Add new paragraphs 5 and 6
	Article C10.7.3.5	Paragraph 1: Revise sentence 1 Paragraph 2: Revise sentence 2 Add new paragraph 3
	Article 10.7.3.6	Paragraph 1: Revise sentence 1 Add new paragraph 2
	Article C10.7.3.6	Delete paragraph 1 Paragraph 2: Revise sentences 1–3 Paragraph 3: Revise sentence 3 Revise paragraph 5 (only one sentence)
	Article 10.7.3.7	Paragraph 1: Revise sentence 2 Revise paragraph 2 (only one sentence)
	Article C10.7.3.7	Revise paragraph 1 (only one sentence) Paragraph 2: Revise sentences 1 and 2 Where list: Revise items 1 and 5 Revise paragraph 5 (only one sentence)
p. 10-97/p. 10-98	Article 10.7.3.8	Revise article title
	Article 10.7.3.8.1	Paragraph 1: Revise sentences 1–4
	Article C10.7.3.8.1	Paragraph 1: Revise sentence 2 Add new paragraphs 2 and 3
	Article 10.7.3.8.2	Paragraph 1: Revise sentences 1 and 2 Revise paragraph 2 (only one sentence) Revise bullet 3
	Article C10.7.3.8.2	Paragraph 1: Revise sentences 1 and 3 Paragraph 1: Add new sentence 4 Paragraph 2: Revise sentences 1 and 4

Page No(s).	Affected Article	Errata Change to
p. 10-97/p. 10-98	Article C10.7.3.8.2	Revise paragraph 3 (only one sentence) Delete paragraph 4
	Article C10.7.3.8.3	Paragraph 1: Delete sentence 2
p. 10-99/p. 10-100	Article 10.7.3.8.3	Paragraph 1: Revise sentence 3
	Article C10.7.3.8.3	Paragraph 1: Revise sentence 2 (was sentence 3)
	Article 10.7.3.8.4	Paragraph 1: Revise sentences 1 and 2 Add new paragraph 2 Paragraph 3 (was 2): Revise sentence 1 Delete old paragraph 3
	Article C10.7.3.8.4	Paragraph 1: Revise sentence 1 Paragraph 1: Add new sentence 2 Paragraph 1: Revise sentences 3 and 4 (were sentences 2 and 3) Delete paragraphs 2 and 3 Add new paragraphs 2 and 3
	Article 10.7.3.8.5	Where list after Eq. 10.7.3.8.5-1: Revise items 1 and 2 Where list after Eq. 10.7.3.8.5-2: Revise item 2
	Article C10.7.3.8.5	Add new paragraph 1 Paragraph 2 (was paragraph 1): Revise sentences 2 and 3 Paragraph 3 (was paragraph 2): Revise sentences 1, 3, and 4 Paragraph 4 (was paragraph 3): Replace entire paragraph with new sentences 1 and 2 Delete old paragraph 4
p. 10-101/p. 10-102	Article C10.7.3.8.5	Paragraph 5: Revise sentence 1
	Article 10.7.3.8.6a	Paragraph 1: Revise sentence 2 Paragraph 2: Revise sentence 2
	Article C10.7.3.8.6a	Paragraph 1: Revise sentence 1
	Article 10.7.3.8.6b	Paragraph 1: Revise sentence 2
p. 10-103/p. 10-104	Article 10.7.3.8.6c	Paragraph 1: Revise sentence 1
	Article C10.7.3.8.6c	Paragraph 1: Revise sentence 2
	Article 10.7.3.8.6d	Paragraph 1: Revise sentence 1
p. 10-105/p. 10-106	Article 10.7.3.8.6e	Where list: Revise item
	Article C10.7.3.8.6f	Paragraph 1: Revise sentence 1
	Article 10.7.3.8.6f	Figure 10.7.3.8.6f-1: Revise caption
p. 10-107/p. 10-108	Article 10.7.3.8.6f	Figure 10.7.3.8.6f-2: Revise caption Figure 10.7.3.8.6f-3: Revise caption Figure 10.7.3.8.6f-4: Revise caption Figure 10.7.3.8.6f-5: Revise caption
p. 10-109/p. 10-110	Article 10.7.3.8.6f	Figure 10.7.3.8.6f-6: Revise caption Figure 10.7.3.8.6f-7: Revise caption

Page No(s).	Affected Article	Errata Change to
p. 10-109/p. 10-110	Article 10.7.3.8.6f	Figure 10.7.3.8.6f-8: Revise caption Figure 10.7.3.8.6f-9: Revise caption
	Article C10.7.3.8.6g	Paragraph 1: Revise sentence 2
p. 10-111/p. 10-112	Article 10.7.3.8.6g	Revise paragraph 3 (only one sentence) Where list after Eq. 10.7.3.8.6g-3: Revise item 1 Revise paragraph 5 (only one sentence)
	Article C10.7.3.8.6g	Revise bullet 2
p. 10-113/p. 10-114	Article 10.7.3.8.6g Article 10.7.3.9	Table 10.7.3.8.6g-2: Revise caption Revise paragraph 1 (only one sentence) Revise paragraph 2 (only one sentence) Revise paragraph 3 (only one sentence)
p. 10-115/p. 10-116	Article 10.7.3.9	Paragraph 5: Revise sentence 1
	Article 10.7.3.10	Revise paragraph 2 (only one sentence) Paragraph 3: Revise sentence 1 Paragraph 3: Add new sentence 3 Paragraph 4: Revise sentences 1 and 2
	Article C10.7.3.10	Paragraph 2: Revise sentences 1 and 2 Revise paragraph 2 (only one sentence) Add new paragraph 4
p. 10-117/p. 10-118	Article 10.7.3.11	Revise paragraph 3 (only one sentence) Revise paragraph 6 (only one sentence) Revise paragraph 7 (only one sentence)
	Article 10.7.3.12	Revise article title Paragraph 1: Revise sentences 1 and 2 Paragraph 2: Revise sentences 1 and 5
	Article C10.7.3.12	Paragraph 1: Revise sentence 1
p. 10-119/p. 10-120	Article C10.7.3.12	Paragraph 5: Revise sentence 1
	Article 10.7.3.13.1	Paragraph 1: Revise sentence 1
	Article C10.7.3.13.1	Paragraph 2: Revise sentences 1 and 3
	Article 10.7.3.13.4	Delete paragraph 3, bullets 1 and 2, Eqs. C10.7.3.13.4-1 and C10.7.3.13.4-2, and where list
	Article C10.7.3.13.4	Add new paragraph 1, bullets 1 and 2, Eqs. C10.7.3.13.4-1 and C10.7.3.13.4-2, and where list Revise paragraph 2 (was paragraph 1; only one sentence) Paragraph 3 (was paragraph 1): Revise sentence 1
	Article 10.7.4	Paragraph 2: Revise sentence 2
p. 10-121/p. 10-122	Article C10.7.5	Revise paragraph 13 (only one sentence)
	Article 10.7.6	Revise paragraph 1 (only one sentence) Revise bullets 3–6

Page No(s).	Affected Article	Errata Change to
p. 10-123/p. 10-124	Article 10.7.6	Revise bullet 7 Paragraph 2: Revise sentence 1
	Article 10.7.7	Revise article title Revise paragraph 1 (only one sentence) Revise bullets 1–5
	Article 10.7.8	Where list after Eq. 10.7.8-1: Revise item 1
	Article C10.7.8	Paragraph 3: Revise sentences 1 and 4 Paragraph 4: Revise sentences 1–3
p. 10-125/p. 10-126	Article 10.7.8	Delete paragraph 2
	Article 10.7.9	Revise article title Paragraph 1: Revise sentences 1 and 2 Paragraph 2: Revise sentence 1 Paragraph 2: Delete sentence 2
	Article C10.7.9	Revise sentences 1 and 2 (only one paragraph)
P 10-159/p. 10-160	Article 10.10	Add Allen (2007) Revise Hannigan et al. (2006)

Please substitute the original pages of text with the enclosed pages, which will remain clearly distinguishable as errata pages once they have been inserted due to the large errata page header.

Please note that there are some unusual page breaks in the errata. This was intentional in order to expedite correction of Section 10 in the downloadable version of the book.

We apologize for any inconvenience this may have caused.

AASHTO Publications Staff

Regarding pile drivability analysis, the only source of load is from the pile driving hammer. Therefore, the load factors provided in Section 3 do not apply. In past practice, e.g., AASHTO (2002), no load factors were applied to the stresses imparted to the pile top by the pile hammer. Therefore, a load factor of 1.0 should be used for this type of analysis. Generally, either a wave equation analysis or dynamic testing, or both, are used to determine the stresses in the pile resulting from hammer impact forces. See Article 10.7.8 for the specific calculation of the pile structural resistance available for analysis of pile drivability. The structural resistance available during driving determined as specified in Article 10.7.8 considers the ability of the pile to handle the transient stresses resulting from hammer impact, considering variations in the materials, pile/hammer misalignment, and variations in the pile straightness and uniformity of the pile head impact surface.

Table 10.5.5.2.3-1—Resistance Factors for Driven Piles

Condition/Resistance Determination Method		Resistance Factor
Nominal Bearing Resistance of Single Pile—Dynamic Analysis and Static Load Test Methods, φ_{dyn}	Driving criteria established by successful static load test of at least one pile per site condition and dynamic testing* of at least two piles per site condition, but no less than 2% of the production piles	0.80
	Driving criteria established by successful static load test of at least one pile per site condition without dynamic testing	0.75
	Driving criteria established by dynamic testing* conducted on 100% of production piles	0.75
	Driving criteria established by dynamic testing,* quality control by dynamic testing* of at least two piles per site condition, but no less than 2% of the production piles	0.65
	Wave equation analysis, without pile dynamic measurements or load test but with field confirmation of hammer performance	0.50
	FHWA-modified Gates dynamic pile formula (End of Drive condition only)	0.40
	Engineering News (as defined in Article 10.7.3.8.5) dynamic pile formula (End of Drive condition only)	0.10

* Dynamic testing requires signal matching, and best estimates of nominal resistance are made from a restrike. Dynamic tests are calibrated to the static load test, when available.

Table 10.5.5.2.3-1—Resistance Factors for Driven Piles (continued)

Condition/Resistance Determination Method		Resistance Factor
Nominal Bearing Resistance of Single Pile—Static Analysis Methods, φ_{stat}	Side Resistance and End Bearing: Clay and Mixed Soils	
	α-method (Tomlinson, 1987; Skempton, 1951)	0.35
	β-method (Esrig & Kirby, 1979; Skempton, 1951)	0.25
	λ-method (Vijayvergiya & Focht, 1972; Skempton, 1951)	0.40
	Side Resistance and End Bearing: Sand	
	Nordlund/Thurman Method (Hannigan et al., 2005)	0.45
	SPT-method (Meyerhof)	0.30
	CPT-method (Schmertmann)	0.50
	End bearing in rock (Canadian Geotech. Society, 1985)	0.45
Block Failure, φ_{b1}	Clay	0.60
Uplift Resistance of Single Piles, φ_{up}	Nordlund Method	0.35
	α-method	0.25
	β-method	0.20
	λ-method	0.30
	SPT-method	0.25
	CPT-method	0.40
	Static load test	0.60
	Dynamic test with signal matching	0.50
Group Uplift Resistance, φ_{ug}	All soils	0.50
Lateral Geotechnical Resistance of Single Pile or Pile Group	All soils and rock	1.0
Structural Limit State	Steel piles	See the provisions of Article 6.5.4.2
	Concrete piles	See the provisions of Article 5.5.4.2.1
	Timber piles	See the provisions of Article 8.5.2.2 and 8.5.2.3
Pile Drivability Analysis, φ_{da}	Steel piles	See the provisions of Article 6.5.4.2
	Concrete piles	See the provisions of Article 5.5.4.2.1
	Timber piles	See the provisions of Article 8.5.2.2
	In all three Articles identified above, use φ identified as “resistance during pile driving”	

10.5.5.2.4—Drilled Shafts

Resistance factors shall be selected based on the method used for determining the nominal shaft resistance. When selecting a resistance factor for shafts in clays or other easily disturbed formations, local experience with the geologic formations and with typical shaft construction practices shall be considered.

Where the resistance factors provided in Table 10.5.5.2.4-1 are to be applied to a single shaft supporting a bridge pier, the resistance factor values in the Table should be reduced by 20 percent. Where the resistance factor is decreased in this manner, the η_R factor provided in Article 1.3.4 shall not be increased to address the lack of foundation redundancy.

The number of static load tests to be conducted to justify the resistance factors provided in Table 10.5.5.2.4-1 shall be based on the variability in the properties and geologic stratification of the site to which the test results are to be applied. A site, for the purpose of assessing variability, shall be defined in accordance with Article 10.5.5.2.3.

C10.5.5.2.4

The resistance factors in Table 10.5.5.2.4-1 were developed using either statistical analysis of shaft load tests combined with reliability theory (Paikowsky et al., 2004), fitting to allowable stress design (ASD), or both. Where the two approaches resulted in a significantly different resistance factor, engineering judgment was used to establish the final resistance factor, considering the quality and quantity of the available data used in the calibration. The available reliability theory calibrations were conducted for the Reese and O'Neill (1988) method, with the exception of shafts in intermediate geo-materials (IGMs), in which case the O'Neill and Reese (1999) method was used. In Article 10.8, the O'Neill and Reese (1999) method is recommended. See Allen (2005) for a more detailed explanation on the development of the resistance factors for shaft foundation design, and the implications of the differences in these two shaft design methods on the selection of resistance factors.

The information in the commentary to Article 10.5.5.2.3 regarding the number of load tests to conduct considering site variability applies to drilled shafts as well.

For single shafts, lower resistance factors are specified to address the lack of redundancy. See Article C10.5.5.2.3 regarding the use of η_R.

Where installation criteria are established based on one or more static load tests, the potential for site variability should be considered. The number of load tests required should be established based on the characterization of site subsurface conditions by the field and laboratory exploration and testing program. One or more static load tests should be performed per site to justify the resistance factor selection as discussed in Article C10.5.5.2.3, applied to drilled shafts installed within the site. See Article C10.5.5.2.3 for details on assessing site variability as applied to selection and use of load tests.

For the specific case of shafts in clay, the resistance factor recommended by Paikowsky et al. (2004) is much lower than the recommendation from Barker et al. (1991). Since the shaft design method for clay is nearly the same for both the 1988 and 1999 methods, a resistance factor that represents the average of the two resistance factor recommendations is provided in Table 10.5.5.2.4-1. This difference may point to the differences in local geologic formations and local construction practices, pointing to the importance of taking such issues into consideration when selecting resistance factors, especially for shafts in clay.

IGMs are materials that are transitional between soil and rock in terms of their strength and compressibility, such as residual soils, glacial tills, or very weak rock. See Article C10.8.2.2.3 for a more detailed definition of an IGM.

Since the mobilization of shaft base resistance is less certain than side resistance due to the greater deformation required to mobilize the base resistance, a lower resistance factor relative to the side resistance is provided for the base resistance in Table 10.5.5.2.4-1. O'Neill and Reese (1999) make further comment that the recommended resistance factor for tip resistance in sand is applicable for conditions of high quality control on the properties of drilling slurries and base cleanout procedures. If high quality control procedures are not used, the resistance factor for the O'Neill and Reese (1999) method for tip resistance in sand should be also be reduced. The amount of reduction should be based on engineering judgment.

Shaft compression load test data should be extrapolated to production shafts that are not load tested as specified in Article 10.8.3.5.6. There is no way to verify shaft resistance for the untested production shafts, other than through good construction inspection and visual observation of the soil or rock encountered in each shaft. Because of this, extrapolation of the shaft load test results to the untested production shafts may introduce some uncertainty. Statistical data are not available to quantify this at this time. Historically, resistance factors higher than 0.70, or their equivalent safety factor in previous practice, have not been used for shaft foundations. If the recommendations in Paikowsky, et al. (2004) are used to establish a resistance factor when shaft static load tests are conducted, in consideration of site variability, the resistance factors recommended by Paikowsky, et al. for this case should be reduced by 0.05, and should be less than or equal to 0.70 as specified in Table 10.5.5.2.4-1.

This issue of uncertainty in how the load test is applied to shafts not load tested is even more acute for shafts subjected to uplift load tests, as failure in uplift can be more abrupt than failure in compression. Hence, a resistance factor of 0.60 for the use of uplift load test results is recommended.

Table 10.5.5.2.4-1—Resistance Factors for Geotechnical Resistance of Drilled Shafts

Method/Soil/Condition			Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts, φ_{stat}	Side resistance in clay	α-method (O'Neill and Reese, 1999)	0.45
	Tip resistance in clay	Total Stress (O'Neill and Reese, 1999)	0.40
	Side resistance in sand	β-method (O'Neill and Reese, 1999)	0.55
	Tip resistance in sand	O'Neill and Reese (1999)	0.50
	Side resistance in IGMs	O'Neill and Reese (1999)	0.60
	Tip resistance in IGMs	O'Neill and Reese (1999)	0.55
	Side resistance in rock	Horvath and Kenney (1979) O'Neill and Reese (1999)	0.55
	Side resistance in rock	Carter and Kulhawy (1988)	0.50
	Tip resistance in rock	Canadian Geotechnical Society (1985) Pressuremeter Method (Canadian Geotechnical Society, 1985) O'Neill and Reese (1999)	0.50
Block Failure, φ_{b1}	Clay		0.55
Uplift Resistance of Single-Drilled Shafts, φ_{up}	Clay	α-method (O'Neill and Reese, 1999)	0.35
	Sand	β-method (O'Neill and Reese, 1999)	0.45
	Rock	Horvath and Kenney (1979) Carter and Kulhawy (1988)	0.40
Group Uplift Resistance, φ_{ug}	Sand and clay		0.45
Horizontal Geotechnical Resistance of Single Shaft or Shaft Group	All materials		1.0
Static Load Test (compression), φ_{load}	All Materials		0.70
Static Load Test (uplift), φ_{upload}	All Materials		0.60

10.5.5.2.5—Micropiles

Resistance factors shall be selected from Table 10.5.5.2.5-1 based on the method used for determining the nominal axial pile resistance. If the resistance factors provided in Table 10.5.5.2.5-1 are to be applied to piles in potentially creeping soils, highly plastic soils, weak rock, or other marginal ground type, the resistance factor values in the Table should be reduced by 20 percent to reflect greater design uncertainty.

C10.5.5.2.5

The resistance factors in Table 10.5.5.2.5-1 were calibrated by fitting to ASD procedures tempered with engineering judgment. The resistance factors in Table 10.5.5.2.5.-2 for structural resistance were calibrated by fitting to ASD procedures and are equal to or slightly more conservative than corresponding resistance factors from Section 5 of the AASHTO LRFD Specifications for reinforced concrete column design.

For footings that rest on clay, the sliding resistance may be taken as the lesser of:

- The cohesion of the clay, or
- Where footings are supported on at least 6.0 in. of compacted granular material, one-half the normal stress on the interface between the footing and soil, as shown in Figure 10.6.3.4-1 for retaining walls.

The following notation shall be taken to apply to Figure 10.6.3.4-1:

q_s = unit shear resistance, equal to S_u or 0.5 σ'_v, whichever is less

R_τ = nominal sliding resistance between soil and foundation (kips) expressed as the shaded area under the q_s diagram

S_u = undrained shear strength (ksf)

σ'_v = vertical effective stress (ksf)

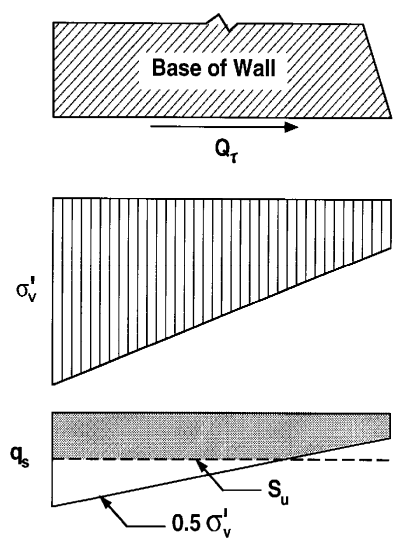


Figure 10.6.3.4-1—Procedure for Estimating Nominal Sliding Resistance for Walls on Clay

10.6.4—Extreme Event Limit State Design

10.6.4.1—General

Extreme limit state design checks for spread footings shall include, but not necessarily be limited to:

- Bearing resistance,
- Eccentric load limitations (overturning),
- Sliding, and
- Overall stability.

Resistance factors shall be as specified in Article 10.5.5.3.

10.6.4.2—Eccentric Load Limitations

For footings, whether on soil or on rock, the eccentricity of loading for extreme limit states shall not exceed the limits provided in Article 11.6.5.

If live loads act to reduce the eccentricity for the Extreme I limit state, γ_{EQ} shall be taken as 0.0.

10.6.5—Structural Design

The structural design of footings shall comply with the requirements given in Section 5.

For structural design of an eccentrically loaded foundation, a triangular or trapezoidal contact stress distribution based on factored loads shall be used for footings bearing on all soil and rock conditions.

For purposes of structural design, it is usually assumed that the bearing stress varies linearly across the bottom of the footing. This assumption results in the slightly conservative triangular or trapezoidal contact stress distribution.

10.7—DRIVEN PILES

10.7.1—General

10.7.1.1—Application

Driven piling should be considered in the following situations:

- When spread footings cannot be founded on rock, or on competent soils at a reasonable cost,
- At locations where soil conditions would normally permit the use of spread footings but the potential exists for scour, liquefaction or lateral spreading, in which case driven piles bearing on suitable materials below susceptible soils should be considered for use as a protection against these problems,
- Where right-of-way or other space limitations would not allow the use spread footings,
- Where existing soil, contaminated by hazardous materials, must be removed for the construction of spread footings, or
- Where an unacceptable amount of settlement of spread footings may occur.

10.7.1.2—Minimum Pile Spacing, Clearance, and Embedment into Cap

Center-to-center pile spacing should not be less than 30.0 in. or 2.5 pile diameters. The distance from the side of any pile to the nearest edge of the pile cap shall not be less than 9.0 in.

The tops of piles shall project at least 12.0 in. into the pile cap after all damaged material has been removed. If the pile is attached to the cap by embedded bars or strands, the pile shall extend no less than 6.0 in. into the cap.

Where a reinforced concrete beam is cast-in-place and used as a bent cap supported by piles, the concrete cover on the sides of the piles shall not be less than 6.0 in., plus an allowance for permissible pile misalignment. Where pile reinforcement is anchored in the cap satisfying the requirements of Article 5.13.4.1, the projection may be less than 6.0 in.

10.7.1.3—Piles through Embankment Fill

Piles to be driven through embankments should penetrate a minimum of 10 ft through original ground unless refusal on bedrock or competent bearing strata occurs at a lesser penetration.

Fill used for embankment construction should be a select material, which does not obstruct pile penetration to the required depth.

C10.7.1.3

If refusal occurs at a depth of less than 10 ft, other foundation types, e.g., footings or shafts, may be more effective.

To minimize the potential for obstruction of the piles, the maximum size of any rock particles in the fill should not exceed 6.0 in. Pre-drilling or spudding pile locations should be considered in situations where obstructions in the embankment fill cannot be avoided, particularly for displacement piles. Note that predrilling or spudding may reduce the pile side resistance and lateral resistance, depending on how the predrilling or spudding is conducted. The diameter of the predrilled or spudded hole, and the potential for caving of the hole before the pile is installed will need to be considered to assess the effect this will have on side and lateral resistance.

If compressible soils are located beneath the embankment, piles should be driven after embankment settlement is complete, if possible, to minimize or eliminate downdrag forces.

10.7.1.4—Batter Piles

When the lateral resistance of the soil surrounding the piles is inadequate to counteract the horizontal forces transmitted to the foundation, or when increased rigidity of the entire structure is required, batter piles should be considered for use. Where negative side resistance (downdrag) loads are expected, batter piles should be avoided. If batter piles are used in areas of significant seismic loading, the design of the pile foundation shall recognize the increased foundation stiffness that results.

C10.7.1.4

In some cases, it may be desirable to use batter piles. From a general viewpoint, batter piles provide a much stiffer resistance to lateral loads than would be possible with vertical piles. They can be very effective in resisting static lateral loads.

Due to increased foundation stiffness, batter piles may not be desirable in resisting lateral dynamic loads if the structure is located in an area where seismic loads are potentially high.

10.7.1.5—Pile Design Requirements

Pile design shall address the following issues as appropriate:

- Nominal bearing resistance to be specified in the contract, type of pile, and size of pile group required to provide adequate support, with consideration of how nominal bearing pile resistance will be determined in the field.
- Group interaction.
- Pile quantity estimation and estimated pile penetration required to meet nominal axial resistance and other design requirements.
- Minimum pile penetration necessary to satisfy the requirements caused by uplift, scour, downdrag, settlement, liquefaction, lateral loads, and seismic conditions.

C10.7.1.5

The driven pile design process is discussed in detail in Hannigan et al. (2006).

- Foundation deflection to meet the established movement and associated structure performance criteria.
- Pile foundation nominal structural resistance.
- Pile drivability to confirm that acceptable driving stresses and blow counts can be achieved at the nominal bearing resistance, and at the estimated resistance to reach the minimum tip elevation, if a minimum tip elevation is required, with an available driving system.
- Long-term durability of the pile in service, i.e. corrosion and deterioration.

10.7.1.6—Determination of Pile Loads

10.7.1.6.1—General

The loads and load factors to be used in pile foundation design shall be as specified in Section 3. Computational assumptions that shall be used in determining individual pile loads are described in Section 4.

C10.7.1.6.1

The specification and determination of top of cap loads is discussed in Section 3. The Engineer should select different levels of analysis, detail and accuracy as appropriate for the structure under consideration. Details are discussed in Section 4.

10.7.1.6.2—Downdrag

The provisions of Article 3.11.8 shall apply for determination of load due to negative side resistance.

Where piles are driven to end bearing on a dense stratum or rock and the design of the pile is structurally controlled, downdrag shall be considered at the strength and extreme limit states.

For friction piles that can experience settlement at the pile tip, downdrag shall be considered at the service, strength and extreme limit states. Estimate pile and pile group settlement according to Article 10.7.2.

The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive side and tip resistance below the lowest layer contributing to downdrag computed as specified in Article 3.11.8.

C10.7.1.6.2

Downdrag occurs when settlement of soils along the side of the piles results in downward movement of the soil relative to the pile. See commentary to Article C3.11.8.

In the case of friction piles with limited tip resistance, the downdrag load can exceed the geotechnical resistance of the pile, causing the pile to move downward enough to allow service limit state criteria for the structure to be exceeded. Where pile settlement is not limited by nominal bearing resistance below the downdrag zone, service limit state tolerances will govern the geotechnical design.

This design situation is not desirable and the preferred practice is to mitigate the downdrag induced foundation settlement through a properly designed surcharge and/or preloading program, or by extending the piles deeper for higher resistance.

Instrumented static load tests, dynamic tests with signal matching, or static analysis procedures in Article 10.7.3.8.6 may be used to estimate the available nominal resistance to withstand the downdrag plus structure loads.

10.7.1.6.3—Uplift Due to Expansive Soils

Piles penetrating expansive soil shall extend to a depth into moisture-stable soils sufficient to provide adequate anchorage to resist uplift. Sufficient clearance should be provided between the ground surface and underside of caps or beams connecting piles to preclude the application of uplift loads at the pile/cap connection due to swelling ground conditions.

C10.7.1.6.3

Evaluation of potential uplift loads on piles extending through expansive soils requires evaluation of the swell potential of the soil and the extent of the soil strata that may affect the pile. One reasonably reliable method for identifying swell potential is presented in Table 10.4.6.3-1. Alternatively, ASTM D4829 may be used to evaluate swell potential. The thickness of the potentially expansive stratum must be identified by:

- Examination of soil samples from borings for the presence of jointing, slickensiding, or a blocky structure and for changes in color, and
- Laboratory testing for determination of soil moisture content profiles.

10.7.1.6.4—Nearby Structures

Where pile foundations are placed adjacent to existing structures, the influence of the existing structure on the behavior of the foundation, and the effect of the new foundation on the existing structures, including vibration effects due to pile installation, shall be investigated.

C10.7.1.6.4

Vibration due to pile driving can cause settlement of existing foundations as well as structural damage to the adjacent facility, especially in loose cohesionless soils. The combination of taking measures to mitigate the vibration levels through use of nondisplacement piles, predrilling, proper hammer choice, etc., and a good vibration monitoring program should be considered.

10.7.2—Service Limit State Design

10.7.2.1—General

Service limit state design of driven pile foundations includes the evaluation of settlement due to static loads, and downdrag loads if present, overall stability, lateral squeeze, and lateral deformation. Overall stability of a pile supported foundation shall be evaluated where:

- The foundation is placed through an embankment,
- The pile foundation is located on, near or within a slope,
- The possibility of loss of foundation support through erosion or scour exists, or
- Bearing strata are significantly inclined.

Unbalanced lateral forces caused by lack of overall stability or lateral squeeze should be mitigated through stabilization measures, if possible.

C10.7.2.1

Lateral analysis of pile foundations is conducted to establish the load distribution between the superstructure and foundations for all limit states, and to estimate the deformation in the foundation that will occur due to those loads. This Article only addresses the evaluation of the lateral deformation of the foundation resulting from the distributed loads.

In general, it is not desirable to subject the pile foundation to unbalanced lateral loading caused by lack of overall stability or caused by lateral squeeze.

10.7.2.2—Tolerable Movements

The provisions of Article 10.5.2.1 shall apply.

C10.7.2.2

See Article C10.5.2.1.

10.7.2.3—Settlement

10.7.2.3.1—Equivalent Footing Analogy

For purposes of calculating the settlements of pile groups, loads should be assumed to act on an equivalent footing based on the depth of embedment of the piles into the layer that provides support as shown in Figures 10.7.2.3.1-1 and 10.7.2.3.1-2.

Pile group settlement shall be evaluated for pile foundations in cohesive soils, soils that include cohesive layers, and piles in loose granular soils. The load used in calculating settlement shall be the permanently applied load on the foundation.

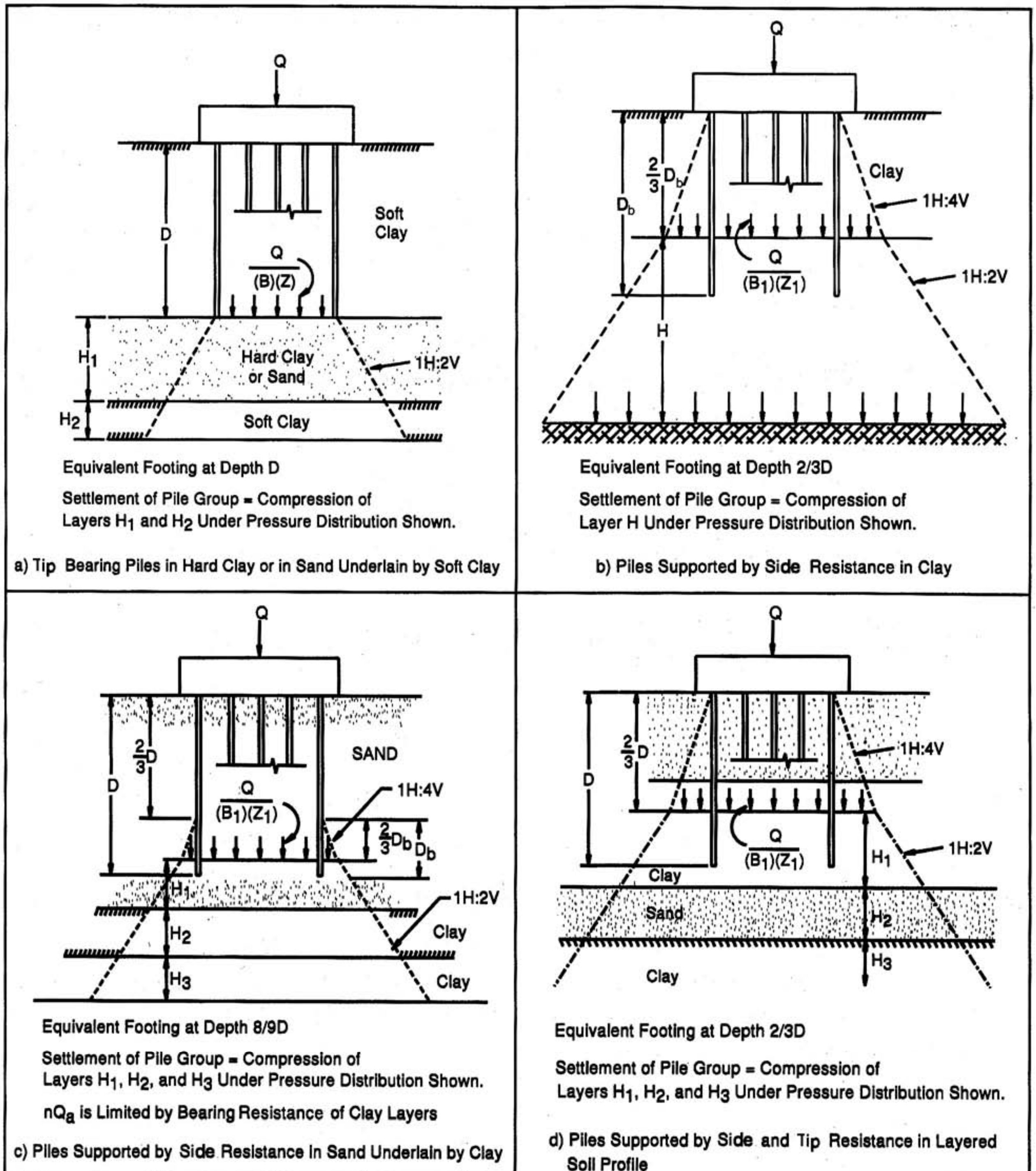
In applying the equivalent footing analogy for pile foundation, the reduction to equivalent dimensions B' and L' as used for spread footing design does not apply.

C10.7.2.3.1

Pile design should ensure that strength limit state considerations are satisfied before checking service limit state considerations.

For piles embedded adequately into dense granular soils such that the equivalent footing is located on or within the dense granular soil, and furthermore are not subjected to downdrag loads, a detailed assessment of the pile group settlement may be waived.

Methods for calculating settlement are discussed in Hannigan et al., (2006).



Notes:

(1) Plan area of perimeter of pile group = (B)(Z).

(2) Plan area $(B_1)(Z_1)$ = projection of area (B)(Z) at depth based on shown pressure distribution.

(3) For relatively rigid pile cap, pressure distribution is assumed to vary with depth as above.

(4) For flexible slab or group of small separate caps, compute pressures by elastic solutions.

Figure 10.7.2.3.1-1—Stress Distribution below Equivalent Footing for Pile Group after Hannigan et al. (2006)

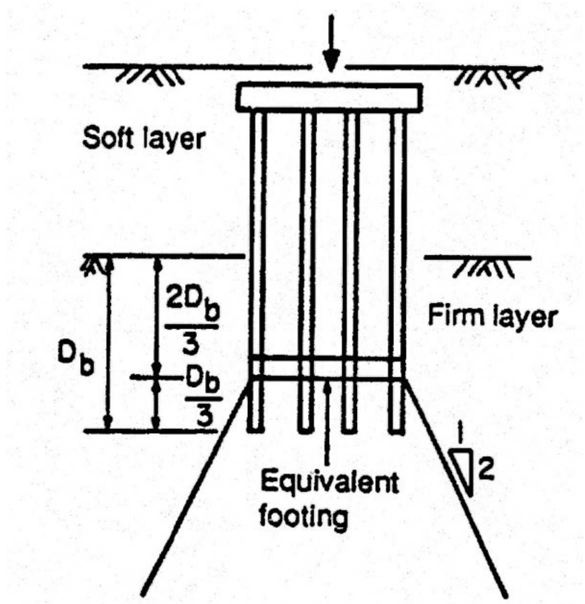


Figure 10.7.2.3.1-2—Location of Equivalent Footing (after Duncan and Buchignani, 1976)

10.7.2.3.2—Pile Groups in Cohesive Soil

Shallow foundation settlement estimation procedures shall be used to estimate the settlement of a pile group, using the equivalent footing location specified in Figure 10.7.2.3-1.1 or Figure 10.7.2.3.1-2.

The settlement of pile groups in cohesionless soils may be taken as:

Using *SPT*: $$\rho = \frac{qI\sqrt{B}}{N1_{60}} \quad (10.7.2.3.2\text{-}1)$$

Using *CPT*: $$\rho = \frac{qBI}{2q_c} \quad (10.7.2.3.2\text{-}2)$$

in which:

$$I = 1 - 0.125\frac{D'}{B} \geq 0.5 \quad (10.7.2.3.2\text{-}3)$$

where:

ρ = settlement of pile group (in.)

q = net foundation pressure applied at $2D_b/3$, as shown in Figure 10.7.2.3.1-1; this pressure is equal to the applied load at the top of the group divided by the area of the equivalent footing and does not include the weight of the piles or the soil between the piles (ksf)

B = width or smallest dimension of pile group (ft)

C10.7.2.3.2

The provisions are based upon the use of empirical correlations proposed by Meyerhof (1976). These are empirical correlations and the units of measure must match those specified for correct computations. This method may tend to over-predict settlements.

I = influence factor of the effective group embedment (dim)

D' = effective depth taken as $2D_b/3$ (ft)

D_b = depth of embedment of piles in layer that provides support, as specified in Figure 10.7.2.3.1-1 (ft)

$N1_{60}$ = *SPT* blow count corrected for both overburden and hammer efficiency effects (blows/ft) as specified in Article 10.4.6.2.4.

q_c = static cone tip resistance (ksf)

Alternatively, other methods for computing settlement in cohesionless soil, such as the Hough method as specified in Article 10.6.2.4.2 may also be used in connection with the equivalent footing approach.

The corrected *SPT* blow count or the static cone tip resistance should be averaged over a depth equal to the pile group width *B* below the equivalent footing. The *SPT* and *CPT* methods (Eqs. 10.7.2.3.2-1 and 10.7.2.3.2-2) shall only be considered applicable to the distributions shown in Figure 10.7.2.3.1-1b and Figure 10.7.2.3.1-2.

10.7.2.4—Horizontal Pile Foundation Movement

Horizontal movement induced by lateral loads shall be evaluated. The provisions of Article 10.5.2.1 shall apply regarding horizontal movement criteria.

The horizontal movement of pile foundations shall be estimated using procedures that consider soil-structure interaction. Tolerable horizontal movements of piles shall be established on the basis of confirming compatible movements of structural components, e.g., pile to column connections, for the loading condition under consideration.

The effects of the lateral resistance provided by an embedded cap may be considered in the evaluation of horizontal movement.

The orientation of nonsymmetrical pile cross-sections shall be considered when computing the pile lateral stiffness.

Lateral resistance of single piles may be determined by static load test. If a static lateral load test is to be performed, it shall follow the procedures specified in ASTM D3966.

The effects of group interaction shall be taken into account when evaluating pile group horizontal movement. When the *P-y* method of analysis is used, the values of *P* shall be multiplied by *P*-multiplier values, P_m, to account for group effects. The values of P_m provided in Table 10.7.2.4-1 should be used.

C10.7.2.4

Pile foundations are subjected to lateral loads due to wind, traffic loads, bridge curvature, vessel or traffic impact and earthquake. Batter piles are sometimes used but they are somewhat more expensive than vertical piles, and vertical piles are more effective against dynamic loads.

Methods of analysis that use manual computation were developed by Broms (1964a and 1964b). They are discussed in detail by Hannigan et al. (2006). Reese developed analysis methods that model the horizontal soil resistance using *P-y* curves. This analysis has been well developed and software is available for analyzing single piles and pile groups (Reese, 1986; Williams et al., 2003; and Hannigan et al., 2006).

Deep foundation horizontal movement at the foundation design stage may be analyzed using computer applications that consider soil-structure interaction. Application formulations are available that consider the total structure including pile cap, pier and superstructure (Williams et al., 2003).

If a lateral static load test is used to assess the site specific lateral resistance of a pile, information on the methods of analysis and interpretation of lateral load tests presented in the *Handbook on Design of Piles and Drilled Shafts Under Lateral Load*, Reese (1984) and *Static Testing of Deep Foundations*, Kyfor et al. (1992) should be used.

Table 10.7.2.4-1—Pile P-Multipliers, P_m, for Multiple Row Shading (averaged from Hannigan et al., 2006)

Pile *CTC* spacing (in the direction of loading)	*P*-Multipliers, P_m		
	Row 1	Row 2	Row 3 and higher
3*B*	0.8	0.4	0.3
5*B*	1.0	0.85	0.7

Loading direction and spacing shall be taken as defined in Figure 10.7.2.4-1. If the loading direction for a single row of piles is perpendicular to the row (bottom detail in the Figure), a group reduction factor of less than 1.0 should only be used if the pile spacing is 5*B* or less, i.e., a P_m of 0.8 for a spacing of 3*B*, as shown in Figure 10.7.2.4-1.

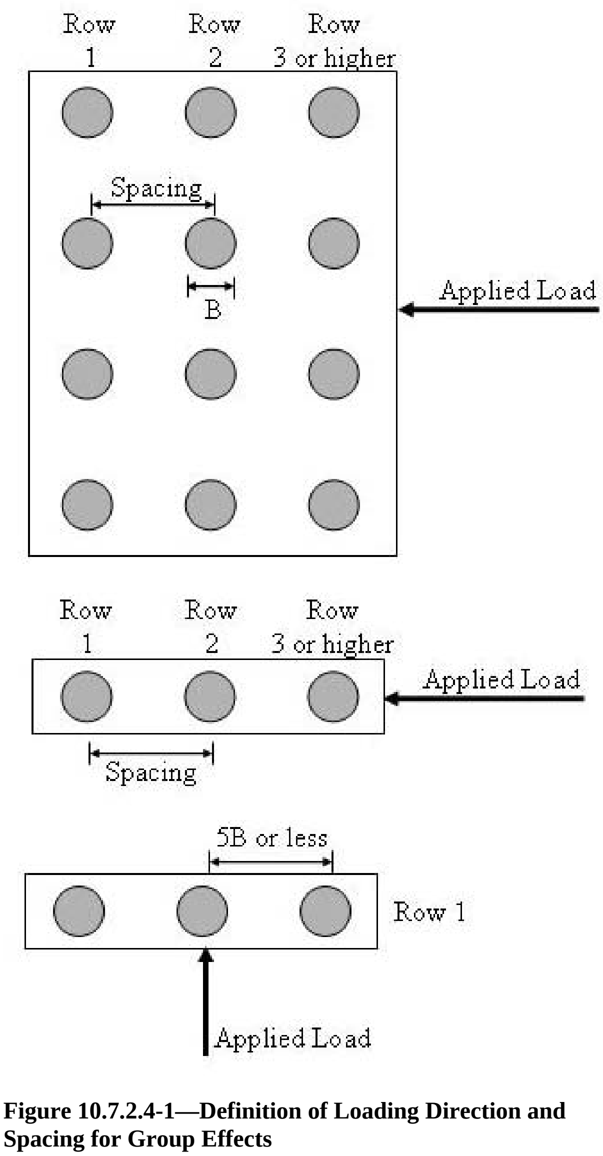


Figure 10.7.2.4-1—Definition of Loading Direction and Spacing for Group Effects

Since many piles are installed in groups, the horizontal resistance of the group has been studied and it has been found that multiple rows of piles will have less resistance than the sum of the single pile resistance. The front piles "shade" rows that are further back.

The P-multipliers, P_m, in Table 10.7.2.4-1 are a function of the center-to-center (*CTC*) spacing of piles in the group in the direction of loading expressed in multiples of the pile diameter, *B*. The values of P_m in Table 10.7.2.4-1 were developed for vertical piles only.

Lateral load tests have been performed on pile groups, and multipliers have been determined that can be used in the analysis for the various rows. Those multipliers have been found to depend on the pile spacing and the row number in the direction of loading. To establish values of P_m for other pile spacing values, interpolation between values should be conducted.

The multipliers are a topic of current research and may change in the future. Values from recent research have been tabulated by Hannigan et al. (2006).

Note that these *P-y* methods generally apply to foundation elements that have some ability to bend and deflect. For large diameter, relatively short foundation elements, e.g., drilled shafts or relatively short stiff piles, the foundation element rotates rather than bends, in which case strain wedge theory (Norris, 1986; Ashour et al., 1998) may be more applicable. When strain wedge theory is used to assess the lateral load response of groups of short, large diameter piles or shaft groups, group effects should be addressed through evaluation of the overlap between shear zones formed due to the passive wedge that develops in front of each shaft in the group as lateral deflection increases. Note that P_m in Table 10.7.2.4-1 is not applicable if strain wedge theory is used.

Batter piles provide a much stiffer lateral response than vertical piles when loaded in the direction of the batter.

10.7.2.5—Settlement Due to Downdrag

The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive side and tip resistance below the lowest layer contributing to the downdrag. In general, the available factored geotechnical resistance should be greater than the factored loads applied to the pile, including the downdrag, at the service limit state. In the case where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag, e.g., piles supported by side resistance, to fully resist the downdrag, the structure should be designed to tolerate the full amount of settlement resulting from the downdrag and the other applied loads.

If adequate geotechnical resistance is available to resist the downdrag plus structure loads in the service limit state, the amount of deformation needed to fully mobilize the geotechnical resistance should be estimated, and the structure designed to tolerate the anticipated movement.

C10.7.2.5

The static analysis procedures in Article 10.7.3.8.6 may be used to estimate the available pile nominal resistance to withstand the downdrag plus structure loads.

Nominal resistance may also be estimated using a dynamic method, e.g., dynamic measurements with signal matching analysis, wave equation, pile driving formula, etc., per Article 10.7.3.8, provided the side resistance within the zone contributing to downdrag is subtracted from the nominal bearing resistance determined from the dynamic method during pile installation. The side resistance within the zone contributing to downdrag may be estimated using the static analysis methods specified in Article 10.7.3.8.6, from signal matching analysis, or from instrumented pile load test results. Note that the static analysis methods may have bias that, on average, over or under predicts the side resistance. The bias of the method selected to estimate the side resistance within the downdrag zone should be taken into account as described in Article 10.7.3.3.

For the establishment of settlement tolerance limits, see Article 10.5.2.1.

10.7.2.6—Lateral Squeeze

Bridge abutments supported on pile foundations driven through soft soils that are subject to unbalanced embankment fill loading shall be evaluated for lateral squeeze.

C10.7.2.6

Guidance on evaluating the potential for lateral squeeze and potential mitigation methods are included in Hannigan et al., (2006).

10.7.3—Strength Limit State Design

10.7.3.1—General

For strength limit state design, the following shall be determined:

- Loads and performance requirements;
- Pile type, dimensions, and nominal bearing resistance;
- Size and configuration of the pile group to provide adequate foundation support;
- Estimated pile length to be used in the construction contract documents to provide a basis for bidding;
- A minimum pile penetration, if required, for the particular site conditions and loading, determined based on the maximum (deepest) depth needed to meet all of the applicable requirements identified in Article 10.7.6;

C10.7.3.1

A minimum pile penetration should only be specified if needed to ensure that uplift, lateral stability, depth to resist downdrag, depth to satisfy scour concerns, and depth for structural lateral resistance are met for the strength limit state, in addition to similar requirements

- The maximum driving resistance expected in order to reach the minimum pile penetration required, if applicable, including any soil/pile side resistance that will not contribute to the long-term nominal bearing resistance of the pile, e.g., soil contributing to downdrag, or soil that will be removed by scour;

- The drivability of the selected pile to achieve the required nominal axial resistance or minimum penetration with acceptable driving stresses at a satisfactory blow count per unit length of penetration; and

- The nominal structural resistance of the pile and/or pile group.

for the service and extreme event limit states. See Article 10.7.6 for additional details. Assuming static load tests, dynamic methods, e.g., dynamic test with signal matching, wave equation, pile formulae, etc., are used during pile installation to establish when the nominal bearing resistance has been met, a minimum pile penetration should not be used to ensure that the required nominal pile bearing, i.e., compression, resistance is obtained.

A nominal resistance measured during driving exceeding the compressive nominal resistance required by the contract may be needed in order to reach a minimum pile penetration specified in the contract.

The drivability analysis is performed to establish whether a hammer and driving system will likely install the pile in a satisfactory manner.

10.7.3.2—Point Bearing Piles on Rock

10.7.3.2.1—General

As applied to pile compressive resistance, this Article shall be considered applicable to soft rock, hard rock, and very strong soils such as very dense glacial tills that will provide high nominal bearing resistance in compression with little penetration.

C10.7.3.2.1

If pile penetration into rock is expected to be minimal, the prediction of the required pile length will usually be based on the depth to rock.

A definition of hard rock that relates to measurable rock characteristics has not been widely accepted. Local or regional experience with driving piles to rock provides the most reliable definition.

In general, it is not practical to drive piles into rock to obtain significant uplift or lateral resistance. The ability to obtain sufficient uplift resistance will depend on the softness of the rock formation. Local experience should also be considered. If significant lateral or uplift foundation resistance is required, drilled shaft foundations should be considered. If it is still desired to use piles, a pile drivability study should be performed to verify the feasibility of obtaining the desired penetration into rock.

10.7.3.2.2—Piles Driven to Soft Rock

Soft rock that can be penetrated by pile driving shall be treated in the same manner as soil for the purpose of design for bearing resistance, in accordance with Article 10.7.3.8.

C10.7.3.2.2

Steel piles driven into soft rock may not require tip protection.

10.7.3.2.3—Piles Driven to Hard Rock

The nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. The nominal bearing resistance shall not exceed the values obtained from Article 6.9.4.1 with the resistance factors specified in Article 6.5.4.2 and Article 6.15 for severe driving conditions. A pile-driving acceptance criteria shall be developed that will prevent pile damage. Dynamic pile measurements should be used to monitor for pile damage.

C10.7.3.2.3

Care should be exercised in driving piles to hard rock to avoid tip damage. The tips of steel piles driven to hard rock should be protected by high strength, cast steel tip protection.

If the rock surface is reasonably flat, installation with pile tip protection should be considered. In the case of sloping rock, or when battered piles are driven to rock, greater difficulty can arise and the use of tip protection with teeth should be considered. The designer should perform a wave equation analysis to check anticipated stresses, and also consider the following to minimize the risk of pile damage during installation:

- Use a relatively small hammer. If a hammer with adjustable stroke or energy setting is used, it should be operated with a small stroke to seat the pile. The nominal axial resistance can then be proven with a few larger hammer blows.
- A large hammer should not be used if it cannot be adjusted to a low stroke. It may be impossible to detect possible toe damage if a large hammer with large stroke is used.
- For any hammer size, specify a limited number of hammer blows after the pile tip reaches the rock, and stop immediately. An example of a limiting criteria is five blows per one half inch.
- Extensive dynamic testing can be used to verify bearing resistance on a large percentage of the piles. This approach could be used to justify larger design nominal resistances.

If such measures are taken, and successful local experience is available, it may be acceptable to not conduct the dynamic pile measurements.

10.7.3.3—Pile Length Estimates for Contract Documents

Subsurface geotechnical information combined with static analysis methods (Article 10.7.3.8.6), preconstruction probe pile programs (Article 10.7.9), and/or pile load tests (Article 10.7.3.8.2) shall be used to estimate the depth of penetration required to achieve the desired nominal bearing resistance to establish contract pile quantities. If static analysis methods are used, potential bias in the method selected should be considered when estimating the penetration depth required to achieve the desired nominal bearing resistance. Local pile driving experience shall also be considered when making pile quantity estimates. If the depth of penetration required to obtain the desired nominal bearing, i.e., compressive, resistance is less than the depth required to meet the provisions of Article 10.7.6, the minimum penetration required per Article 10.7.6 should be used as the basis for estimating contract pile quantities.

C10.7.3.3

The estimated pile length necessary to provide the required nominal resistance is determined using a static analysis, local pile driving experience, knowledge of the site subsurface conditions, and/or results from a static pile load test program. The required pile length is often defined by the presence of an obvious bearing layer. Local pile driving experience with such a bearing layer should be strongly considered when developing pile quantity estimates.

In variable soils, a program of probe piles across the site is often used to determine variable pile order lengths. Probe piles are particularly useful when driving concrete piles. The pile penetration depth (i.e., length) used to estimate quantities for the contract should also consider requirements to satisfy other design considerations, including service and extreme event limit states, as well as minimum pile penetration requirements for lateral stability, uplift, downdrag, scour, group settlement, etc.

One solution to the problem of predicting pile length is the use of a preliminary test program at the site. Such a program can range from a very simple operation of driving a few piles to evaluate drivability, to an extensive program where different pile types are driven and static load and dynamic testing is performed. For large projects, such test programs may be very cost effective.

In lieu of local pile driving experience, if a static analysis method is used to estimate the pile length required to achieve the desired nominal resistance for establishment of contract pile quantities, to theoretically account for method bias, the factored resistance used to determine the number of piles required in the pile group may be conservativley equated to the factored resistance estimated using the static analysis method as follows:

$$\varphi_{dyn} \text{ x } R_n = \varphi_{stat} \text{ x } R_{nstat} \quad \text{(C10.7.3.3-1)}$$

where:

φ_{dyn} = the resistance factor for the dynamic method used to verify pile bearing resistance during driving specified in Table 10.5.5.2.3-1

R_n = the nominal pile bearing resistance (kips)

φ_{stat} = the resistance factor for the static analysis method used to estimate the pile penetration depth required to achieve the desired bearing resistance specified in Table 10.5.5.2.3-1

R_{nstat} = the predicted nominal resistance from the static analysis method used to estimate the penetration depth required (kips)

Using Eq. C10.7.3.3-1 and solving for R_{nstat}, use the static analysis method to determine the penetration depth required to obtain R_{nstat}.

The resistance factor for the static analysis method inherently accounts for the bias and uncertainty in the static analysis method. However, local experience may dictate that the penetration depth estimated using this approach be adjusted to reflect that experience. Where piles are driven to a well defined firm bearing stratum, the location of the top of bearing stratum will dictate the pile length needed, and Eq. C10.7.3.3-1 is likely not applicable.

Note that R_n is considered to be nominal bearing resistance of the pile needed to resist the applied loads, and is used as the basis for determining the resistance to be achieved during pile driving, R_{ndr} (see Articles 10.7.6 and 10.7.7). R_{nstat} is only used in the static analysis method to estimate the pile penetration depth required.

Note that while there is a theoretical basis to this suggested approach, it can produce apparently erroneous results if attempting to use extremes in static analysis and dynamic methods, e.g., using static load test results and then using the Engineering News formula to control pile driving, or using a very inaccurate static analysis method in combination with dynamic testing and signal matching. Part of the problem is that the available resistance factors have been established in consideration of the risk and consequences of pile foundation failure rather than the risk and consequences of underrunning or overrunning pile quantities. Therefore, the approach provided in Eq. C10.7.3.3-1 should be used cautiously, especially when the difference between the resistance factors for method used to estimate pile penetration depth versus the one used for obtaining the required nominal axial resistance is large.

10.7.3.4—Nominal Axial Resistance Change after Pile Driving

10.7.3.4.1—General

Consideration should be given to the potential for change in the nominal axial pile resistance after the end of pile driving. The effect of soil relaxation or setup should be considered in the determination of nominal axial pile resistance for soils that are likely to be subject to these phenomena.

C10.7.3.4.1

Soil relaxation is not a common phenomenon but more serious than setup since it represents a reduction in the reliability of the foundation.

Soil setup is a common phenomenon that can provide the opportunity for using larger nominal resistances at no increase in cost. However, it is necessary that the resistance gain be adequately proven. This is usually accomplished by restrike testing with dynamic measurements (Komurka, et. al, 2003).

10.7.3.4.2—Relaxation

If relaxation is possible in the soils at the site the pile shall be tested in re-strike after a sufficient time has elapsed for relaxation to develop.

C10.7.3.4.2

Relaxation is a reduction in axial pile resistance. While relaxation typically occurs at the pile tip, it can also occur along the sides of the pile (Morgano and White, 2004). It can occur in dense sands or sandy silts and in some shales. Relaxation in the sands and silts will usually develop fairly quickly after the end of driving (perhaps in only a few minutes or hours) as a result of the return of the reduced pore pressure induced by dilation of the dense sands during driving. In some shales, relaxation occurs during the driving of adjacent piles and that will be immediate. There are other shales where the pile penetrates the shale and relaxation requires perhaps as much as two weeks to develop. In some cases, the amount of relaxation can be large.

10.7.3.4.3—Setup

Setup in the nominal axial resistance may be used to support the applied load. Where increase in resistance due to setup is utilized, the existence of setup shall be verified after a specified length of time by re-striking the pile.

C10.7.3.4.3

Setup is an increase in the nominal axial resistance that develops over time predominantly along the pile shaft. Pore pressures increase during pile driving due to a reduction of the soil volume, reducing the effective stress and the shear strength. Setup may occur rapidly in cohesionless soils and more slowly in finer grained soils as excess pore water pressures dissipate. In some clays, setup may continue to develop over a period of weeks and even months, and in large pile groups it can develop even more slowly.

Setup, sometimes called "pile freeze," can be used to carry applied load, providing the opportunity for using larger pile nominal axial resistances, if it can be proven. Signal matching analysis of dynamic pile measurements made at the end of driving and later in re-strike can be an effective tool in evaluating and quantifying setup. (Komurka et al., 2003; Bogard and Matlock, 1990).

If a wave equation or dynamic formula is used to determine the nominal pile bearing resistance on re-strike, care should be used as these approaches require accurate blow count measurement which is inherently difficult at the beginning of redrive (BOR). Furthermore, the resistance factors provided in Table 10.5.5.2.3-1 for driving formulas were developed for end of driving conditions and empirically have been developed based on the assumption that soil setup will occur. See Article C10.5.5.2.3 for additional discussion on this issue.

Higher degrees of confidence for the assessment of setup effects are provided by dynamic measurements of pile driving with signal matching analyses or static load tests after a sufficient wait time following pile installation.

The restrike time and frequency should be based on the time dependent strength change characteristics of the soil. The following restrike durations are recommended:

Soil Type	Time Delay until Restrike
Clean Sands	1 day
Silty Sands	2 days
Sandy Silts	3-5 days
Silts and Clays	7-14 days*
Shales	7 days

* Longer times are sometimes required.

Specifying a restrike time for friction piles in fine grained soils which is too short may result in pile length overruns.

10.7.3.5—Groundwater Effects and Buoyancy

Nominal axial resistance shall be determined using the groundwater level consistent with that used to calculate the effective stress along the pile sides and tip. The effect of hydrostatic pressure shall be considered in the design.

C10.7.3.5

Unless the pile is bearing on rock, the bearing resistance is primarily dependent on the effective surcharge that is directly influenced by the groundwater level. For drained loading conditions, the vertical effective stress is related to the groundwater level and thus it affects pile axial resistance. Lateral resistance may also be affected.

Buoyant forces may also act on a hollow pile or unfilled casing if it is sealed so that water does not enter the pile. During pile installation, this may affect the driving resistance (blow count) observed, especially in very soft soils.

For design purposes, anticipated changes in the groundwater level during construction and over the life of the structure should be considered with regard to its effect on pile resistance and constructability.

10.7.3.6—Scour

The effect of scour shall be considered in determining the minimum pile embedment and the required nominal driving resistance, R_{ndr}. The pile foundation shall be designed so that the pile penetration after the design scour event satisfies the required nominal axial and lateral resistance.

The resistance factors shall be those used in the design without scour. The side resistance of the material lost due to scour should be determined using a static analysis and it should not be factored, but consideration should be given to the bias of the static analysis method used to predict resistance. Method bias is discussed in Article 10.7.3.3.

The pile foundation shall be designed to resist debris loads occurring during the flood event in addition to the loads applied from the structure.

C10.7.3.6

The piles will need to be driven to the required nominal bearing resistance plus the side resistance that will be lost due to scour. The nominal resistance of the remaining soil is determined through field verification. The pile is driven to the required nominal bearing resistance plus the magnitude of the side resistance lost as a result of scour, considering the prediction method bias.

Another approach that may be used takes advantage of dynamic measurements. In this case, the static analysis method is used to determine an estimated length. During the driving of test piles, the side resistance component of the bearing resistance of pile in the scourable material may be determined by a signal matching analysis of the restrike dynamic measurements obtained when the pile tip is below the scour elevation. The material below the scour elevation must provide the required nominal resistance after scour occurs.

In some cases, the flooding stream will carry debris that will induce horizontal loads on the piles.

Additional information regarding pile design for scour is provided in Hannigan et al. (2006).

10.7.3.7—Downdrag

The foundation should be designed so that the available factored geotechnical resistance is greater than the factored loads applied to the pile, including the downdrag, at the strength limit state. The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive side and tip resistance below the lowest layer contributing to the downdrag. The pile foundation shall be designed to structurally resist the downdrag plus structure loads.

In the instance where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag, e.g., piles supported by side resistance, to fully resist the downdrag, or if it is anticipated that significant deformation will be required to mobilize the geotechnical resistance needed to resist the factored loads including the downdrag load, the structure should be designed to tolerate the settlement resulting from the downdrag and the other applied loads as specified in Article 10.7.2.5.

C10.7.3.7

The static analysis procedures in Article 10.7.3.8.6 may be used to estimate the available pile nominal resistance to withstand the downdrag plus structure loads.

Nominal resistance may also be estimated using an instrumented static load test or dynamic testing during restrike with signal matching, provided the side resistance within the zone contributing to downdrag is subtracted from the resistance determined from the static load or dynamic test. The side resistance within the zone contributing to downdrag may be estimated using the static analysis methods specified in Article 10.7.3.8.6, from restrike signal matching analysis, or from instrumented static pile load test results. Note that the static analysis method may have a bias, on average over or under predicting the side resistance. The bias of the method selected to estimate the skin friction should be taken into account as described in Article C10.7.3.3.

Pile design for downdrag is illustrated in Figure C10.7.3.7-1.

where:

R_{Sdd} = side resistance which must be overcome during driving through downdrag zone (kips)

$Q_p = \Sigma\gamma_i Q_i$ = factored load per pile, excluding downdrag load (kips)

DD = downdrag load per pile (kips)

$D_{est.}$ = estimated pile length needed to obtain desired nominal resistance per pile (ft)

φ_{dyn} = resistance factor, assuming that a dynamic method is used to estimate nominal pile resistance during installation of the pile (if a static analysis method is used instead, use φ_{stat})

γ_p = load factor for downdrag

The summation of the factored loads ($\Sigma\gamma_i Q_i$) should be less than or equal to the factored resistance ($\varphi_{dyn} R_n$). Therefore, the nominal resistance R_n should be greater than or equal to the sum of the factored loads divided by the resistance factor φ_{dyn}. The nominal bearing resistance (kips) of the pile needed to resist the factored loads, including downdrag, is therefore taken as:

$$R_n = \frac{\left(\Sigma\gamma_i Q_i\right)}{\varphi_{dyn}} + \frac{\gamma_p DD}{\varphi_{dyn}} \quad \text{(C10.7.3.7-1)}$$

The total nominal driving resistance, R_{ndr} (kips), needed to obtain R_n, accounting for the side resistance that must be overcome during pile driving that does not contribute to the nominal resistance of the pile, is taken as:

$$R_{ndr} = R_{Sdd} + R_n \quad \text{(C10.7.3.7-2)}$$

where:

R_{ndr} = nominal pile driving resistance required (kips)

Note that R_{Sdd} remains unfactored in this analysis to determine R_{ndr}.

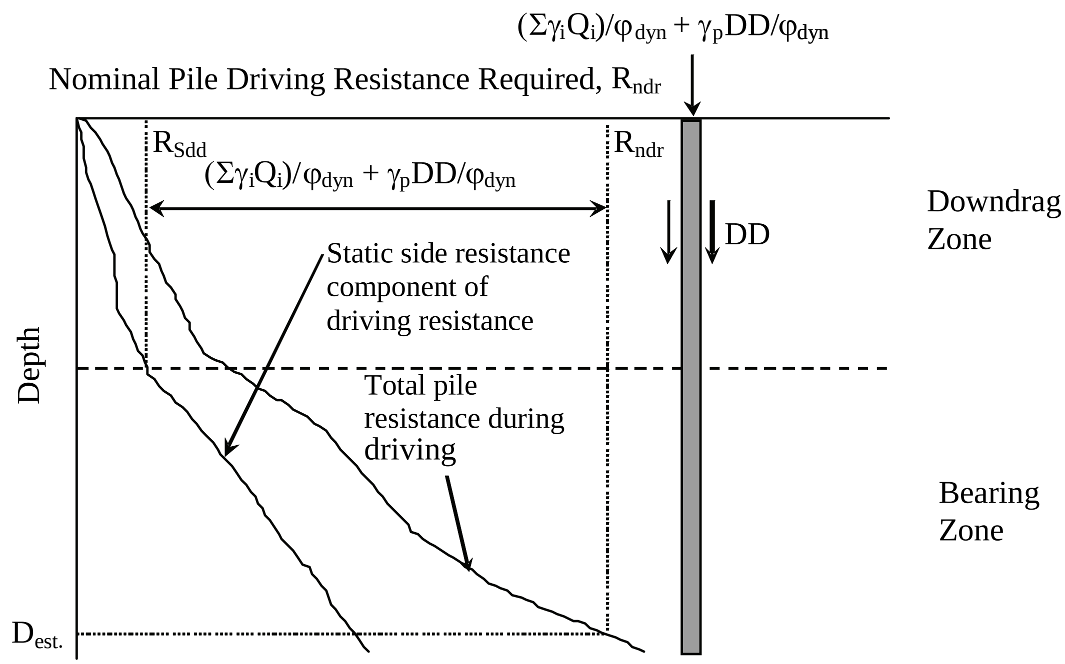


Figure C10.7.3.7-1—Design of Pile Foundations for Downdrag

10.7.3.8—Determination of Nominal Bearing Resistance for Piles

10.7.3.8.1—General

Nominal pile bearing resistance should be field verified during pile installation using static load tests, dynamic tests, wave equation analysis, or dynamic formula. The resistance factor selected for design shall be based on the method used to verify pile bearing resistance as specified in Article 10.5.5.2.3. The production piles shall be driven to the minimum blow count determined from the static load test, dynamic test, wave equation, or dynamic formula and, if required, to a minimum penetration needed for uplift, scour, lateral resistance, or other requirements as specified in Article 10.7.6. If it is determined that static load testing is not feasible and dynamic methods are unsuitable for field verification of nominal bearing resistance, the piles shall be driven to the tip elevation determined from the static analysis, and to meet other limit states as required in Article 10.7.6.

C10.7.3.8.1

This Article addresses the determination of the nominal bearing (compression) resistance needed to meet strength limit state requirements, using factored loads and factored resistance values. From this design step, the number of piles and pile nominal resistance needed to resist the factored loads applied to the foundation are determined. Both the loads and resistance values are factored as specified in Articles 3.4.1 and 10.5.5.2.3, respectively, for this determination.

In most cases, the nominal resistance of production piles should be controlled by driving to a required blow count. In a few cases, usually piles driven into cohesive soils with little or no toe resistance and very long wait times to achieve the full pile resistance increase due to soil setup, piles may be driven to depth. However, even in those cases, a pile may be selected for testing after a sufficient waiting period, using either a static load test or a dynamic test.

In cases where the project is small and the time to achieve soil setup is large compared with the production time to install all of the piles, no field testing for the verification of nominal resistance may be acceptable.

10.7.3.8.2—Static Load Test

If a static pile load test is used to determine the pile nominal axial resistance, the test shall not be performed less than 5 days after the test pile was driven unless approved by the Engineer. The load test shall follow the procedures specified in ASTM D1143, and the loading procedure should follow the Quick Load Test Procedure.

Unless specified otherwise by the Engineer, the nominal bearing resistance shall be determined from the test data as follows:

- For piles 24 in. or less in diameter (length of side for square piles), the Davisson Method;
- For piles larger than 36 in. in diameter (length of side for square piles), at a pile top movement, s_f (in.), as determined from Eq. 10.7.3.8.2-1; and
- For piles greater than 24 in. but less than 36 in. in diameter, criteria to determine the nominal bearing resistance that is linearly interpolated between the criteria determined at diameters of 24 and 36 in.

$$s_f = \frac{QL}{12\,AE} + \frac{B}{2.5} \qquad (10.7.3.8.2\text{-}1)$$

where:

Q = test load (kips)

L = pile length (ft)

A = pile cross-sectional area (ft^2)

E = pile modulus (ksi)

B = pile diameter (length of side for square piles) (ft)

Driving criteria should be established in consideration of the static load test results.

C10.7.3.8.2

The Quick Load Test Procedure is preferred because it avoids problems that frequently arise when performing a static load test that cannot be completed within an eight-hour period. Tests that extend over a longer period are difficult to perform due to the limited number of experienced personnel that are usually available. The Quick Load Test has proven to be easily performed in the field and the results usually are satisfactory. Static load tests should be conducted to failure whenever possible and practical to extract the maximum information, particularly when correlating with dynamic tests or static analysis methods. However, if the formation in which the pile is installed may be subject to significant creep settlement, alternative procedures provided in ASTM D1143 should be considered.

The Davisson Method to determine nominal bearing resistance evaluation is performed by constructing a line on the static load test curve that is parallel to the elastic compression line of the pile. The elastic compression line is calculated by assuming equal compressive forces are applied to the pile ends. The elastic compression line is offset by a specified amount of displacement. The Davisson Method is illustrated in Figure C10.7.3.8.2-1 and described in more detail in Hannigan et al. (2006).

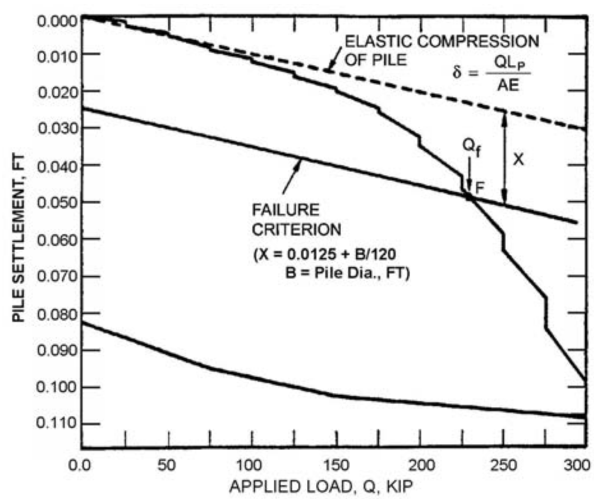


Figure C10.7.3.8.2-1—Alternate Method Load Test Interpretation (Cheney and Chassie, 2000, modified after Davisson, 1972)

For piles with large cross-sections, i.e., diameters greater than 24 in., the Davisson Method will under predict the nominal pile bearing resistance.

Development of driving criteria in consideration of static load test results is described in Hannigan, et al. (2006).

10.7.3.8.3—Dynamic Testing

Dynamic testing shall be performed according to the procedures given in ASTM D4945. If possible, the dynamic test should be performed as a restrike test if the Engineer anticipates significant time dependent strength

C10.7.3.8.3

The dynamic test may be used to establish the driving criteria at the beginning of production driving. A signal matching analysis (Rausche et al., 1972) of the dynamic test data should always be used change. The nominal pile bearing resistance shall be determined by a signal matching analysis of the dynamic pile test data if the dynamic test is used to establish the driving criteria.

to determine bearing resistance if a static load test is not performed. See Hannigan et al. (2006) for a description of and procedures to conduct a signal matching analysis. Re-strike testing should be performed if setup or relaxation is anticipated.

For example, note that it may not be possible to adjust the dynamic measurements with signal matching analysis to match the static load test results if the driving resistance at the time the dynamic measurement is taken is too large, i.e., the pile set per hammer blow is too small. In this case, adequate hammer energy is not reaching the pile tip to assess end bearing and produce an accurate match, though in such cases, the prediction will usually be very conservative. In general, a tip movement (pile set) of 0.10 to 0.15 in. is needed to provide an accurate signal matching analysis. See Hannigan, et al. (2006) for additional guidance on this issue.

In cases where a significant amount of soil setup occurs and the set at the beginning of redrive (BOR) is less than 0.10 inch per blow, a more accurate nominal resistance may be obtained by combining the end bearing determined using the signal matching analysis obtained for the end of driving (EOD) with the signal matching analysis for the shaft resistance at the beginning of redrive.

Dynamic testing and interpretation of the test data should only be performed by certified, experienced testers.

10.7.3.8.4—Wave Equation Analysis

If a wave equation analysis is to be used to establish the driving criteria, it shall be performed based on the hammer and pile driving system to be used for pile installation.

If a wave equation analysis is used for the determination of the nominal bearing resistance, then the driving criterion (blow count) may be the value taken either at the end of driving (EOD) or at the beginning of redrive (BOR). The latter should be used where the soils exhibit significant strength changes (setup or relaxation) with time. When restrike (i.e., BOR) blow counts are taken, the hammer shall be warmed up prior to restrike testing and the blow count shall be taken as accurately as possible for the first inch of restrike.

If the wave equation is used to assess the potential for pile damage, driving stresses shall not exceed the values obtained in Article 10.7.8, using the resistance factors specified or referred to in Table 10.5.5.2.3-1. Furthermore, the blow count needed to obtain the maximum driving resistance anticipated shall be less than the maximum value established based on the provisions in Article 10.7.8.

C10.7.3.8.4

Note that without dynamic test results with signal matching analysis and/or pile load test data (see Articles 10.7.3.8.2 and 10.7.3.8.3), some judgment is required to use the wave equation to predict the pile bearing resistance. Unless experience in similar soils exists, the recommendations of the software provider should be used for dynamic resistance input. Key soil input values that affect the predicted nominal resistance include the soil damping and quake values, the skin friction distribution, e.g., such as could be obtained from a static pile bearing analysis, and the anticipated amount of soil setup or relaxation. The actual hammer performance is a variable that can only be accurately assessed through dynamic measurements, though field observations such as hammer stroke or measured ram velocity can and should be used to improve the accuracy of the wave equation prediction.

In general, improved prediction accuracy of nominal bearing resistance is obtained when targeting the driving criteria at BOR conditions, if soil setup or relaxation is anticipated. Using the wave equation to predict nominal bearing resistance from EOD blow counts requires that an accurate estimate of the time-dependent changes in bearing resistance due to soil setup or relaxation be made. This is generally difficult to do unless site-specific, longer-term measurements of bearing resistance from static load tests or dynamic measurements with signal matching are available. Hence, driving criteria based on BOR measurements are recommended when using the wave equation for driving criteria development.

A wave equation analysis should also be used to evaluate pile drivability during design.

10.7.3.8.5—Dynamic Formula

If a dynamic formula is used to establish the driving criterion, the FHWA Gates Formula (Eq. 10.7.3.8.5-1) should be used. The nominal pile resistance as measured during driving using this method shall be taken as:

$$R_{ndr} = 1.75\sqrt{E_d}\,\log_{10}(10N_b) - 100 \quad (10.7.3.8.5\text{-}1)$$

where:

R_{ndr} = nominal pile driving resistance measured during pile driving (kips)

E_d = developed hammer energy. This is the kinetic energy in the ram at impact for a given blow. If ram velocity is not measured, it may be assumed equal to the potential energy of the ram at the height of the stroke, taken as the ram weight times the actual stroke (ft-lb)

N_b = Number of hammer blows for 1.0 in. of pile permanent set (blows/in.)

The Engineering News formula, modified to predict a nominal bearing resistance, may be used. The nominal pile resistance using this method shall be taken as:

$$R_{ndr} = \frac{12E_d}{(s+0.1)} \quad (10.7.3.8.5\text{-}2)$$

where:

R_{ndr} = nominal pile resistance measured during driving (kips)

E_d = developed hammer energy. This is the kinetic energy in the ram at impact for a given blow. If ram velocity is not measured, it may be assumed equal to the potential energy of the ram at the height of the stroke, taken as the ram weight times the stroke (ft-kips)

s = pile permanent set, (in.)

If a dynamic formula other than those provided herein is used, it shall be calibrated based on measured load test results to obtain an appropriate resistance factor, consistent with Article C10.5.5.2.

If a drivability analysis is not conducted, for steel piles, design stresses shall be limited as specified in Article 6.15.2.

C10.7.3.8.5

It is preferred to use more accurate methods such as wave equation or dynamic testing with signal matching to establish driving criteria (i.e., blow count). However, driving formulas have been in use for many years. Therefore, driving formulas are provided as an option for the development of driving criteria.

Two dynamic formulas are provided here for the Engineer. If a dynamic formula is used for either determination of the nominal resistance or the driving criterion, the FHWA Modified Gates formula is preferred over the Engineering News formula. It is discussed further in the Design and Construction of Driven Pile Foundations (Hannigan et al., 2006). Note that the units in the FHWA Gates formula are not consistent. The specified units in Eq. 10.7.3.8.5-1 must be used.

The Engineering News formula in its traditional form contains a factor of safety of 6.0. For LRFD applications, to produce a nominal resistance, the factor of safety has been removed. As is true of the FHWA Gates formula, the units specified in Eq. 10.7.3.8.5-2 must be used for the Engineering News formula. See Allen (2005, 2007) for additional discussion on the development of the Engineering News formula and its modification to produce a nominal resistance.

Driving formula should only be used to determine end of driving blow count criteria. These driving formula are empirically based on pile load test results, and therefore inherently include some degree of soil setup or relaxation (see Allen, 2007).

Dynamic formulas should not be used when the required nominal resistance exceeds 600 kips.

As the required nominal bearing resistance increases, the reliability of dynamic formulas tends to decrease. The FHWA Gates formula tends to underpredict pile nominal resistance at higher resistances. The Engineering News formula tends to become unconservative as the nominal pile resistance increases. If other driving formulas are used, the limitation on the maximum driving resistance to be used should be based upon the limits for which the data is considered reliable, and any tendency of the formula to over or under predict pile nominal resistance.

10.7.3.8.6—Static Analysis

10.7.3.8.6a—General

Where a static analysis prediction method is used to determine pile installation criteria, i.e., for bearing resistance, the nominal pile resistance shall be factored at the strength limit state using the resistance factors in Table 10.5.5.2.3-1 associated with the method used to compute the nominal bearing resistance of the pile. The factored nominal bearing resistance of piles, R_R, may be taken as:

$$R_R = \varphi R_n \tag{10.7.3.8.6a-1}$$

or:

$$R_R = \varphi R_n = \varphi_{stat} R_p + \varphi_{stat} R_s \tag{10.7.3.8.6a-2}$$

in which:

$$R_p = q_p A_p \tag{10.7.3.8.6a-3}$$

$$R_s = q_s A_s \tag{10.7.3.8.6a-4}$$

where:

φ_{stat} = resistance factor for the bearing resistance of a single pile specified in Article 10.5.5.2.3

R_p = pile tip resistance (kips)

R_s = pile side resistance (kips)

q_p = unit tip resistance of pile (ksf)

q_s = unit side resistance of pile (ksf)

A_s = surface area of pile side (ft^2)

A_p = area of pile tip (ft^2)

Both total stress and effective stress methods may be used, provided the appropriate soil strength parameters are available. The resistance factors for the side resistance and tip resistance, estimated using these methods, shall be as specified in Table 10.5.5.2.3-1. The limitations of each method as described in Article C10.5.5.2.3 should be applied in the use of these static analysis methods.

C10.7.3.8.6a

While the most common use of static analysis methods is solely for estimating pile quantities, a static analysis may be used to establish pile installation criteria if dynamic methods are determined to be unsuitable for field verification of nominal bearing resistance. This is applicable on projects where pile quantities are relatively small, pile loads are relatively low, and/or where the setup time is long so that re-strike testing would require an impractical wait-period by the Contractor on the site, e.g., soft silts or clays where a large amount of setup is anticipated.

For use of static analysis methods for contract pile quantity estimation, see Article 10.7.3.3.

10.7.3.8.6b—α-Method

The α-method, based on total stress, may be used to relate the adhesion between the pile and clay to the undrained strength of the clay. For this method, the nominal unit side resistance, in ksf, shall be taken as:

$$q_s = \alpha S_u \qquad (10.7.3.8.6b\text{-}1)$$

where:

S_u = undrained shear strength (ksf)

α = adhesion factor applied to S_u (dim)

The adhesion factor for this method, α, shall be assumed to vary with the value of the undrained strength, S_u, as shown in Figure 10.7.3.8.6b-1.

C10.7.3.8.6b

The α-method has been used for many years and gives reasonable results for both displacement and nondisplacement piles in clay.

In general, this method assumes that a mean value of S_u will be used. It may not always be possible to establish a mean value, as in many cases, data are too limited to reliably establish the mean value. The Engineer should apply engineering judgment and local experience as needed to establish an appropriate value for design (see Article C10.4.6).

For H-piles, the perimeter or “box” area should generally be used to compute the surface area of the pile side.

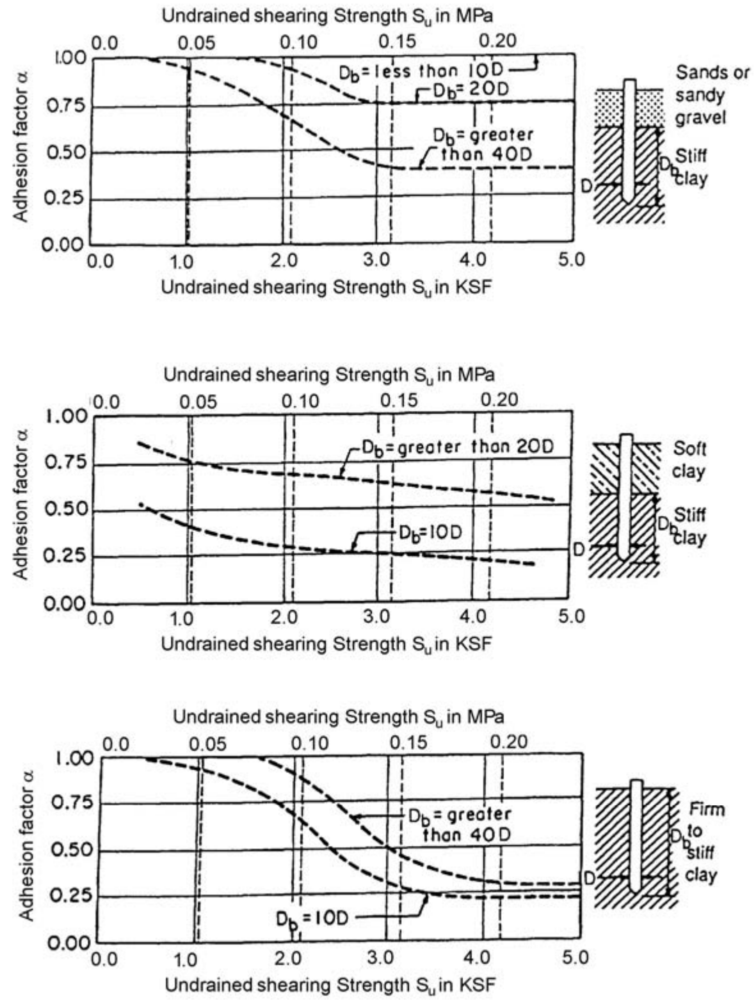


Figure 10.7.3.8.6b-1—Design Curves for Adhesion Factors for Piles Driven into Clay Soils after Tomlinson (1980)

10.7.3.8.6c—β-Method

The β-method, based on effective stress, may be used for predicting side resistance of prismatic piles. The nominal unit skin friction for this method, in ksf, shall be related to the effective stresses in the ground as:

$$q_s = \beta\sigma'_v \quad (10.7.3.8.6c\text{-}1)$$

where:

σ'_v = vertical effective stress (ksf)

β = a factor taken from Figure 10.7.3.8.6c-1

C10.7.3.8.6c

The β-method has been found to work best for piles in normally consolidated and lightly overconsolidated clays. The method tends to overestimate side resistance of piles in heavily overconsolidated soils. Esrig and Kirby (1979) suggested that for heavily overconsolidated clays, the value of β should not exceed two.

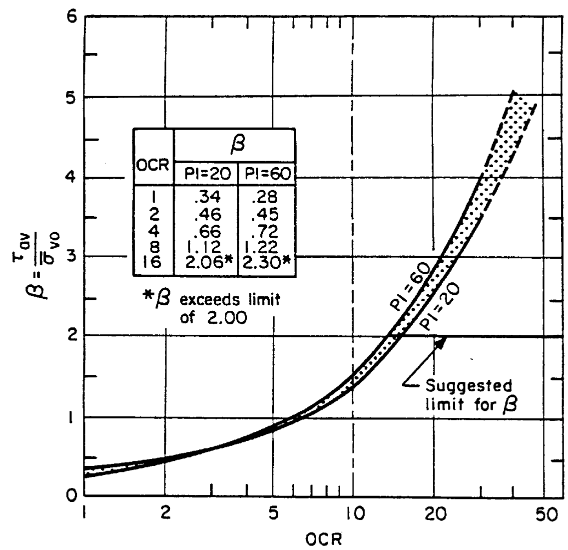


Figure 10.7.3.8.6c-1—β Versus OCR for Displacement Piles after Esrig and Kirby (1979)

10.7.3.8.6d—λ-Method

The λ-method, based on effective stress (though it does contain a total stress parameter), may be used to relate the unit side resistance, in ksf, to passive earth pressure. For this method, the unit skin friction shall be taken as:

$$q_s = \lambda(\sigma'_v + 2S_u) \quad (10.7.3.8.6d\text{-}1)$$

where:

$\sigma'_v + 2S_u$ = passive lateral earth pressure (ksf)

σ'_v = the effective vertical stress at midpoint of soil layer under consideration (ksf)

λ = an empirical coefficient taken from Figure 10.7.3.8.6d-1 (dim)

C10.7.3.8.6d

The value of λ decreases with pile length and was found empirically by examining the results of load tests on steel pipe piles.

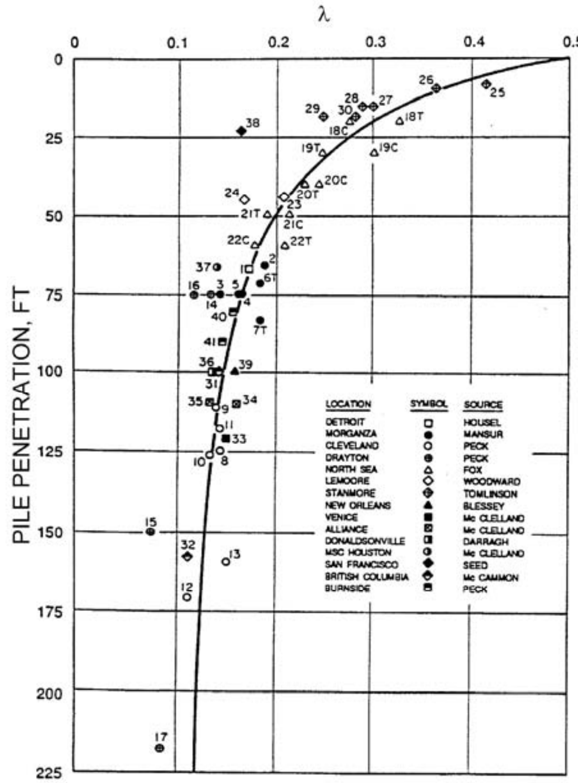


Figure 10.7.3.8.6d-1—λ Coefficient for Driven Pipe Piles after Vijayvergiya and Focht (1972)

10.7.3.8.6e—Tip Resistance in Cohesive Soils

The nominal unit tip resistance of piles in saturated clay, in ksf, shall be taken as:

$$q_p = 9S_u \qquad (10.7.3.8.6e\text{-}1)$$

where:

S_u = undrained shear strength of the clay near the pile tip (ksf)

10.7.3.8.6f—Nordlund/Thurman Method in Cohesionless Soils

This effective stress method should be applied only to sands and nonplastic silts. The nominal unit side resistance, q_s, for this method, in ksf, shall be taken as:

$$q_s = K_\delta C_F \sigma'_v \frac{\sin(\delta + \omega)}{\cos\ \omega} \qquad (10.7.3.8.6f\text{-}1)$$

where:

K_δ = coefficient of lateral earth pressure at mid-point of soil layer under consideration from Figures 10.7.3.8.6f-1 through 10.7.3.8.6f-4 (dim)

C10.7.3.8.6f

Detailed design procedures for the Nordlund/Thurman method are provided in Hannigan et al., (2006). This method was derived based on load test data for piles in sand. In practice, it has been used for gravelly soils as well.

The effective overburden stress is not limited in Eq. 10.7.3.8.6f-1.

For H-piles, the perimeter or "box" area should generally be used to compute the surface area of the pile side.

C_F = correction factor for K_δ when $\delta \neq \phi_f$, from Figure 10.7.3.8.6f-5

σ'_v = effective overburden stress at midpoint of soil layer under consideration (ksf)

δ = friction angle between pile and soil obtained from Figure 10.7.3.8.6f-6 (degrees)

ω = angle of pile taper from vertical (degrees)

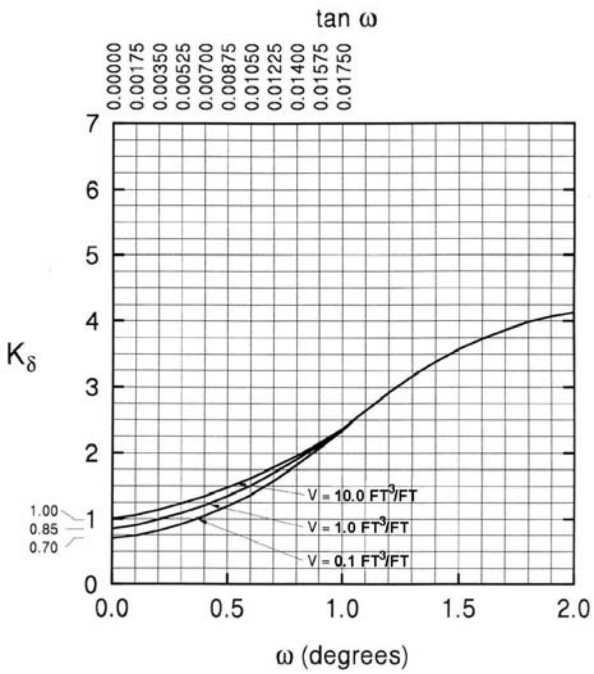


Figure 10.7.3.8.6f-1—Design Curve for Evaluating K_δ for Piles where ϕ_f = 25 degrees (Hannigan et al., 2006 after Nordlund, 1979)

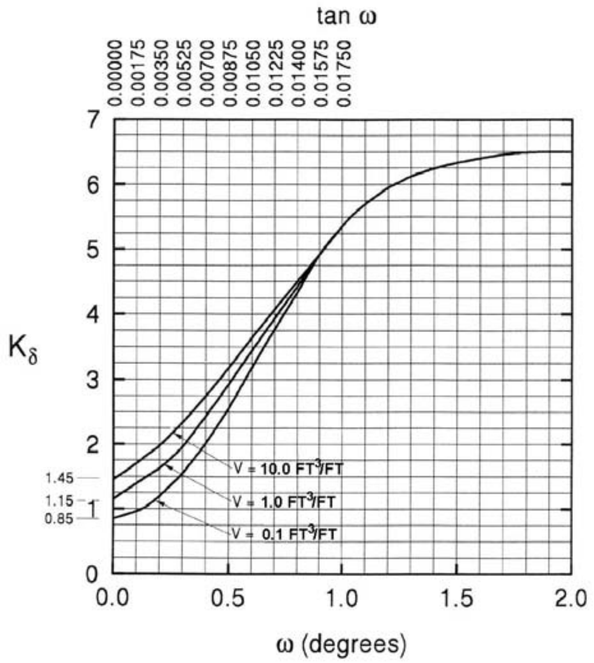


Figure 10.7.3.8.6f-2—Design Curve for Evaluating K_δ for Piles where ϕ_f = 30 degrees (Hannigan et al., 2006 after Nordlund, 1979)

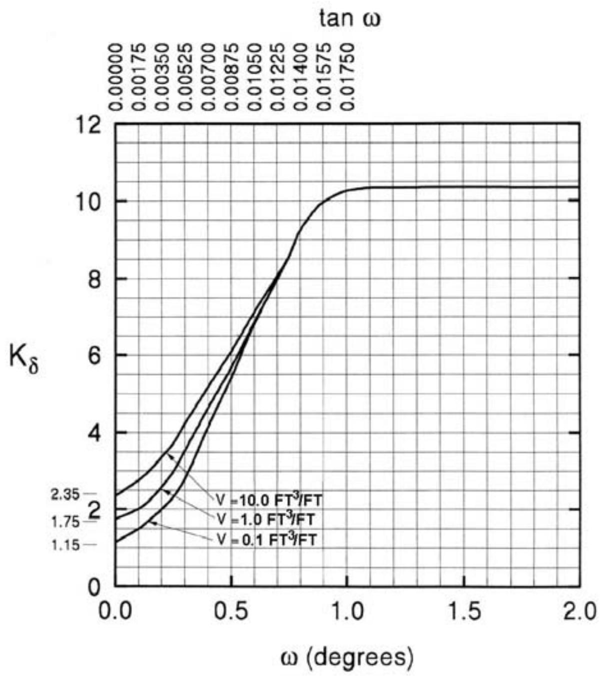


Figure 10.7.3.8.6f-3—Design Curve for Evaluating K_δ for Piles where ϕ_f = 35 degrees (Hannigan et al., 2006 after Nordlund, 1979)

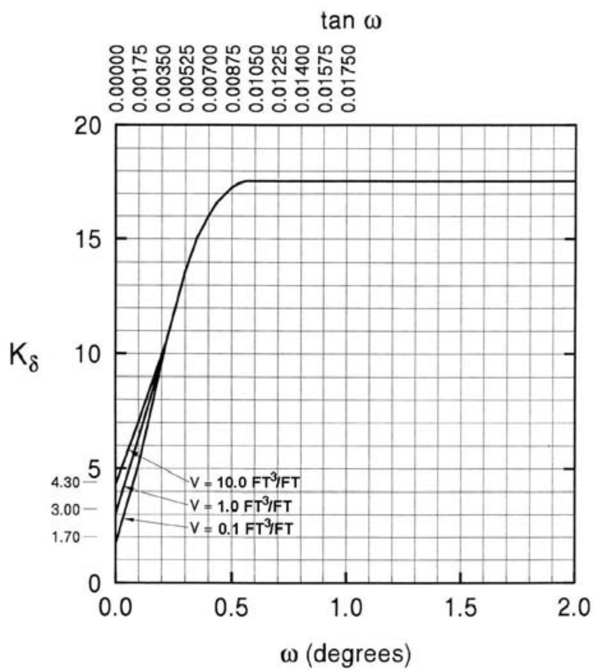


Figure 10.7.3.8.6f-4—Design Curve for Evaluating K_δ for Piles where ϕ_f = 40 degrees (Hannigan et al., 2006 after Nordlund, 1979)

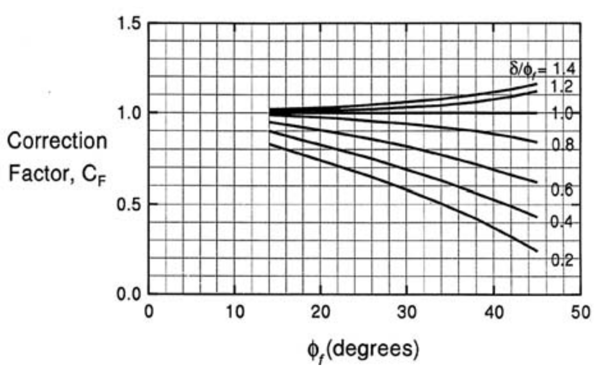


Figure 10.7.3.8.6f-5—Correction Factor for K_δ where $\delta \neq \phi_f$ (Hannigan et al., 2006 after Nordlund, 1979)

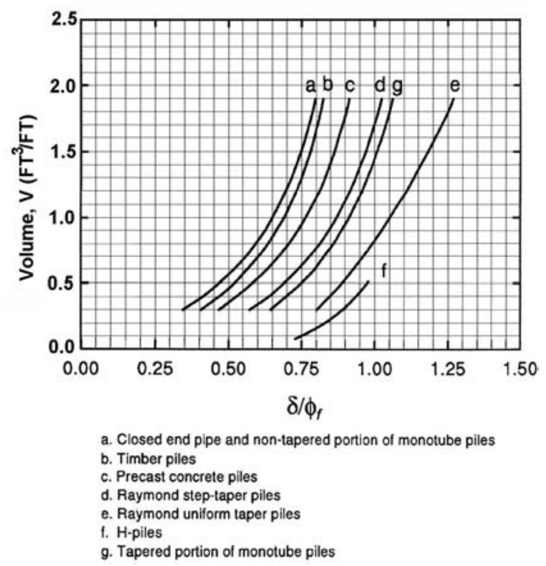


Figure 10.7.3.8.6f-6—Relation of δ/ϕ_f and Pile Displacement, *V*, for Various Types of Piles (Hannigan et al., 2006 after Nordlund, 1979)

The nominal unit tip resistance, q_p, in ksf by the Nordlund/Thurman method shall be taken as:

$$q_p = \alpha_t N'_q \sigma'_v \leq q_L \qquad (10.7.3.8.6f\text{-}2)$$

where:

α_t = coefficient from Figure 10.7.3.8.6f-7 (dim)

N'_q = bearing capacity factor from Figure 10.7.3.8.6f-8

σ'_v = effective overburden stress at pile tip (ksf) ≤3.2 ksf

q_L = limiting unit tip resistance from Figure 10.7.3.8.6f-9

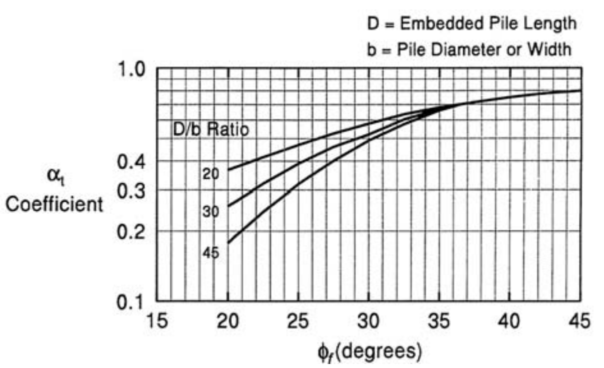


Figure 10.7.3.8.6f-7—α_t Coefficient (Hannigan et al., 2006 modified after Bowles, 1977)

If the friction angle, ϕ_f, is estimated from average, corrected *SPT* blow counts, $N1_{60}$, the $N1_{60}$ values should be averaged over the zone from the pile tip to two diameters below the pile tip.

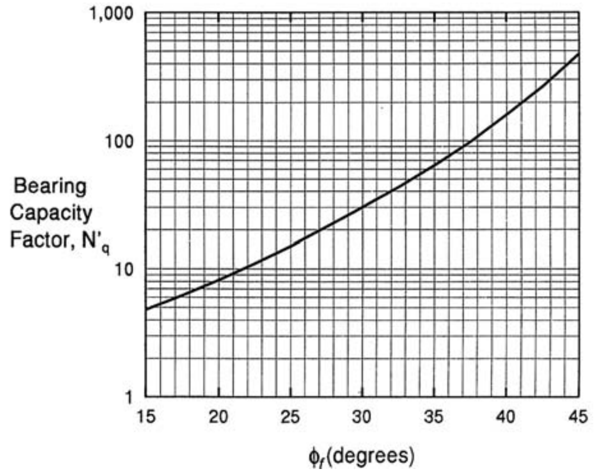


Figure 10.7.3.8.6f-8—Bearing Capacity Factor, N'_q (Hannigan et al., 2006 modified after Bowles, 1977)

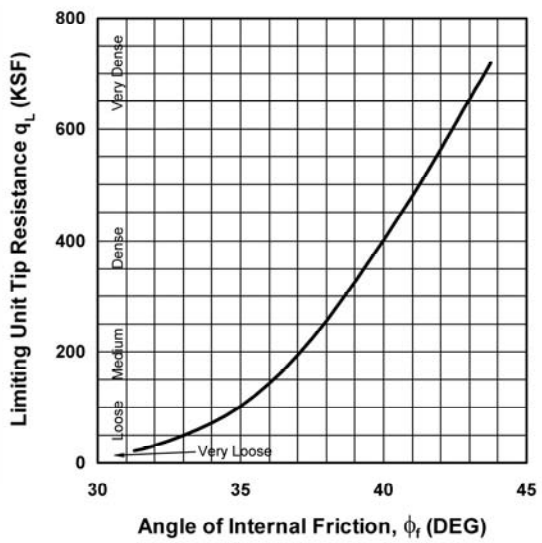


Figure 10.7.3.8.6f-9—Limiting Unit Pile Tip Resistance (Hannigan et al., 2006 after Meyerhof, 1976)

10.7.3.8.6g—Using SPT or CPT in Cohesionless Soils

These methods shall be applied only to sands and nonplastic silts.

The nominal unit tip resistance for the Meyerhof method, in ksf, for piles driven to a depth D_b into a cohesionless soil stratum shall be taken as:

$$q_p = \frac{0.8(N1_{60})D_b}{D} \leq q_\ell \qquad (10.7.3.8.6g\text{-}1)$$

C10.7.3.8.6g

In-situ tests are widely used in cohesionless soils because obtaining good quality samples of cohesionless soils is very difficult. In-situ test parameters may be used to estimate the tip resistance and side resistance of piles.

Two frequently used in-situ test methods for predicting pile axial resistance are the standard penetration test (*SPT*) method (Meyerhof, 1976) and the cone penetration test (*CPT*) method (Nottingham and Schmertmann, 1975).

where:

$N1_{60}$ = representative *SPT* blow count near the pile tip corrected for overburden pressure as specified in Article 10.4.6.2.4 (blows/ft)

D = pile width or diameter (ft)

D_b = depth of penetration in bearing strata (ft)

q_ℓ = limiting tip resistance taken as eight times the value of $N1_{60}$ for sands and six times the value of $N1_{60}$ for nonplastic silt (ksf)

The nominal side resistance of piles in cohesionless soils for the Meyerhof method, in ksf, shall be taken as:

- For driven displacement piles:

$$q_s = \frac{\bar{N}1_{60}}{25} \tag{10.7.3.8.6g-2}$$

- For nondisplacement piles, e.g., steel H-piles:

$$q_s = \frac{\bar{N}1_{60}}{50} \tag{10.7.3.8.6g-3}$$

where:

q_s = unit side resistance for driven piles (ksf)

$\bar{N}1_{60}$ = average corrected *SPT*-blow count along the pile side (blows/ft)

Displacement piles, which have solid sections or hollow sections with a closed end, displace a relatively large volume of soil during penetration. Nondisplacement piles usually have relatively small cross-sectional areas, e.g., steel H-piles and open-ended pipe piles that have not yet plugged. Plugging occurs when the soil between the flanges in a steel H-pile or the soil in the cylinder of an open-ended steel pipe pile adheres fully to the pile and moves down with the pile as it is driven.

Tip resistance, q_p, for the Nottingham and Schmertmann method, in ksf, shall be determined as shown in Figure 10.7.3.8.6g-1.

In which:

$$q_p = \frac{q_{c1} + q_{c2}}{2} \tag{10.7.3.8.6g-4}$$

where:

q_{c1} = average q_c over a distance of yD below the pile tip (path a-b-c); sum q_c values in both the downward (path a-b) and upward (path b-c) directions; use actual q_c values along path a-b and the minimum path rule along path b-c; compute q_{c1} for y-values from 0.7 to 4.0 and use the minimum q_{c1} value obtained (ksf)

CPT may be used to determine:

- The cone penetration resistance, q_c, which may be used to determine the tip resistance of piles, and
- Sleeve friction, f_s, which may be used to determine the side resistance.

q_{c2} = average q_c over a distance of 8D above the pile tip (path c-e); use the minimum path rule as for path b-c in the q_{c1}, computations; ignore any minor "x" peak depressions if in sand but include in minimum path if in clay (ksf)

The minimum average cone resistance between 0.7 and four pile diameters below the elevation of the pile tip shall be obtained by a trial and error process, with the use of the minimum-path rule. The minimum-path rule shall also be used to find the value of cone resistance for the soil for a distance of eight pile diameters above the tip. The two results shall be averaged to determine the pile tip resistance.

The nominal side resistance of piles for this method, in kips, shall be taken as:

$$R_s = K_{s,c}\left[\sum_{i=1}^{N_1}\left(\frac{L_i}{8D_i}\right)f_{si}a_{si}h_i + \sum_{i=1}^{N_2} f_{si}a_{si}h_i\right] \quad (10.7.3.8.6g\text{-}5)$$

where:

$K_{s,c}$ = correction factors: K_c for clays and K_s for sands from Figure 10.7.3.8.6g-2 (dim)

L_i = depth to middle of length interval at the point considered (ft)

D_i = pile width or diameter at the point considered (ft)

f_{si} = unit local sleeve friction resistance from *CPT* at the point considered (ksf)

a_{si} = pile perimeter at the point considered (ft)

h_i = length interval at the point considered (ft)

N_1 = number of intervals between the ground surface and a point 8D below the ground surface

N_2 = number of intervals between 8D below the ground surface and the tip of the pile

This process is described in Nottingham and Schmertmann (1975).

For a pile of constant cross-section (nontapered), Eq. 10.7.3.8.6g-5 can be written as:

$$R_s = K_{s,c}\left[\frac{a_s}{8D}\sum_{i=1}^{N_1} L_i f_{si} h_i + a_s \sum_{i=1}^{N_2} f_{si} h_i\right] \quad (C10.7.3.8.6g\text{-}1)$$

If, in addition to the pile being prismatic, f_s is approximately constant at depths below 8D, Eq. C10.7.3.8.6g-1 can be simplified to:

$$R_s = K_{s,c}\left[a_s f_s (Z - 4D)\right] \quad (C10.7.3.8.6g\text{-}2)$$

where:

Z = total embedded pile length (ft)

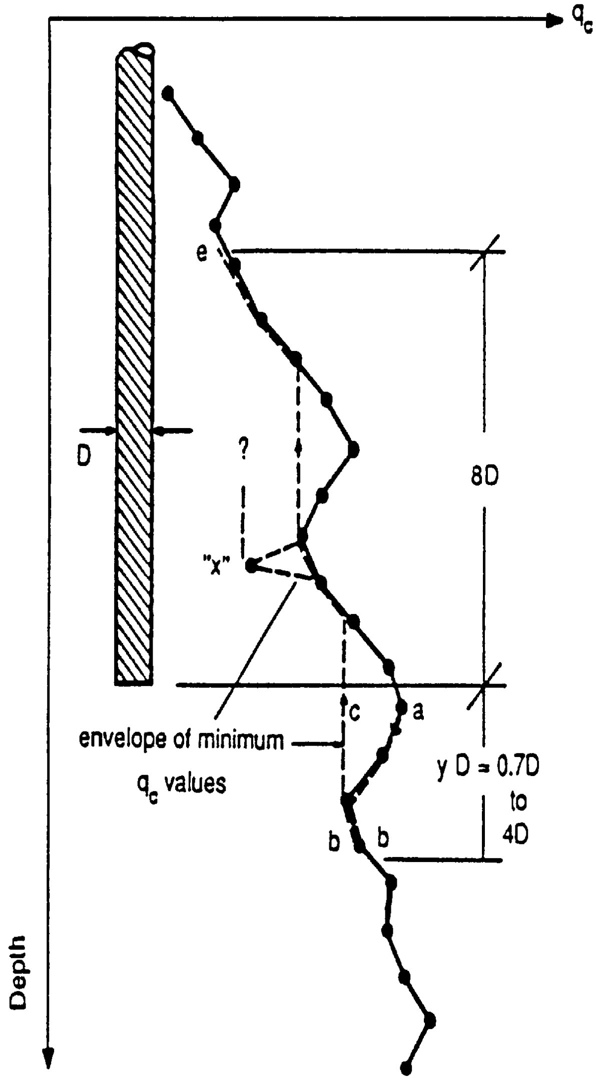


Figure 10.7.3.8.6g-1—Pile End-Bearing Computation Procedure after Nottingham and Schmertmann (1975)

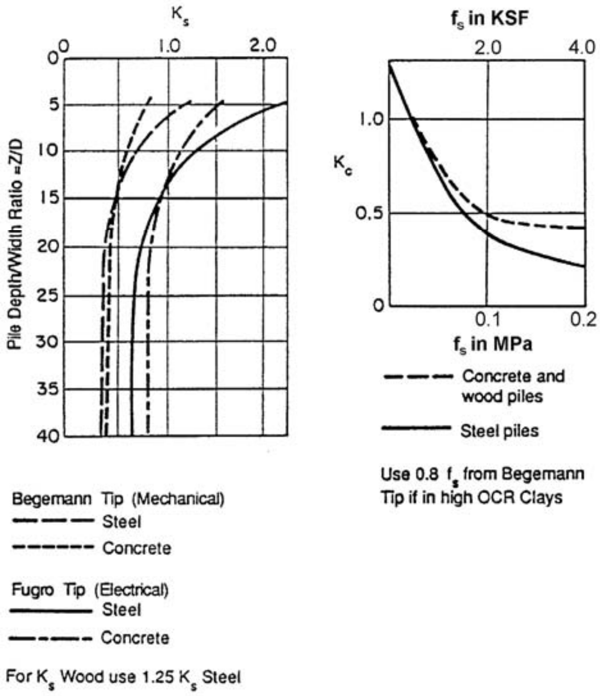


Figure 10.7.3.8.6g-2 —Side Resistance Correction Factors K_s and K_c after Nottingham and Schmertmann (1975)

10.7.3.9—Resistance of Pile Groups in Compression

For pile groups in clay, the nominal bearing resistance of the pile group shall be taken as the lesser of:

- The sum of the individual nominal resistances of each pile in the group, or
- The nominal resistance of an equivalent pier consisting of the piles and the block of soil within the area bounded by the piles.

If the cap is not in firm contact with the ground and if the soil at the surface is soft, the individual nominal resistance of each pile shall be multiplied by an efficiency factor η, taken as:

- η = 0.65 for a center-to-center spacing of 2.5 diameters,
- η = 1.0 for a center-to-center spacing of 6.0 diameters.

For intermediate spacings, the value of η should be determined by linear interpolation.

C10.7.3.9

The equivalent pier approach checks for block failure and is generally only applicable for pile groups within cohesive soils. For pile groups in cohesionless soils, the sum of the nominal resistances of the individual piles always controls the group resistance.

When analyzing the equivalent pier, the full shear strength of the soil should be used to determine the friction resistance. The total base area of the equivalent pier should be used to determine the end bearing resistance.

In cohesive soils, the nominal resistance of a pile group depends on whether the cap is in firm contact with the ground beneath. If the cap is in firm contact, the soil between the pile and the pile group behave as a unit.

At small pile spacings, a block type failure mechanism may prevail, whereas individual pile failure may occur at larger pile spacings. It is necessary to check for both failure mechanisms and design for the case that yields the minimum capacity.

For a pile group of width *X*, length *Y*, and depth *Z*, as shown in Figure C10.7.3.9-1, the bearing capacity for block failure, in kips, is given by:

If the cap is in firm contact with the ground, no reduction in efficiency shall be required. If the cap is not in firm contact with the ground and if the soil is stiff, no reduction in efficiency shall be required.

The nominal bearing resistance of pile groups in cohesionless soil shall be the sum of the resistance of all the piles in the group. The efficiency factor, η, shall be 1.0 where the pile cap is or is not in contact with the ground for a center-to-center pile spacing of 2.5 diameters or greater. The resistance factor is the same as that for single piles, as specified in Table 10.5.5.2.3-1.

For pile groups in clay or sand, if a pile group is tipped in a strong soil deposit overlying a weak deposit, the block bearing resistance shall be evaluated with consideration to pile group punching as a group into the underlying weaker layer. The methods in Article 10.6.3.1.2a of determining bearing resistance of a spread footing in a strong layer overlying a weaker layer shall apply, with the notional footing located as shown in Article 10.7.2.3.

$$Q_g = (2X + 2Y)Z\overline{S_u} + XYN_cS_u \quad \text{(C10.7.3.9-1)}$$

in which:

for $\frac{Z}{X} \leq 2.5$:

$$N_c = 5\left(1+\frac{0.2X}{Y}\right)\left(1+\frac{0.2Z}{X}\right) \quad \text{(C10.7.3.9-2)}$$

for $\frac{Z}{X} > 2.5$:

$$N_c = 7.5\left(1+\frac{0.2X}{Y}\right) \quad \text{(C10.7.3.9-3)}$$

where:

$\overline{S_u}$ = average undrained shear strength along the depth of penetration of the piles (ksf)

S_u = undrained shear strength at the base of the group (ksf)

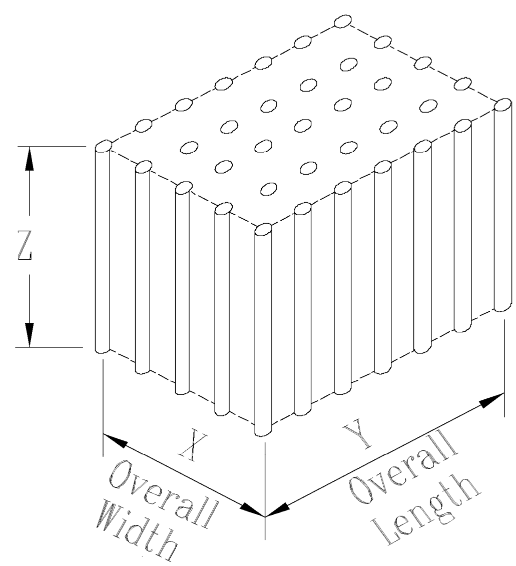


Figure C10.7.3.9-1—Pile Group Acting as a Block Foundation

10.7.3.10—Uplift Resistance of Single Piles

Uplift on single piles shall be evaluated when tensile forces are present. The factored nominal tensile resistance of the pile due to soil failure shall be greater than the factored pile loads.

The nominal uplift resistance of a single pile should be estimated in a manner similar to that for estimating the side resistance of piles in compression specified in Article 10.7.3.8.6.

Factored uplift resistance in kips shall be taken as:

$$R_R = \varphi R_n = \varphi_{up} R_s \qquad (10.7.3.10\text{-}1)$$

where:

R_S = nominal uplift resistance due to side resistance (kips)

φ_{up} = resistance factor for uplift resistance specified in Table 10.5.5.2.3-1

C10.7.3.10

The factored load effect acting on any pile in a group may be estimated using the traditional elastic strength of materials procedure for a cross-section under thrust and moment. The cross-sectional properties should be based on the pile as a unit area.

Note that the resistance factor for uplift already is reduced to 80 percent of the resistance factor for static side resistance. Therefore, the side resistance estimated based on Article 10.7.3.8.6 does not need to be reduced to account for uplift effects on side resistance.

Nominal uplift resistance of single piles may be determined by static load test or by dynamic test with signal matching. If a static uplift test is to be performed, it shall follow the procedures specified in ASTM D 3689. Dynamic tests with signal matching, if conducted, shall be performed as specified in Article 10.7.3.8.3. If dynamic tests with signal matching are used to determine uplift, a maximum of 80 percent of the uplift determined from the dynamic test should be used.

The static pile uplift load test(s) should be used to calibrate the static analysis method, i.e., back calculate soil properties, to adjust the calculated uplift resistance for variations in the stratigraphy. The minimum penetration criterion to obtain the desired uplift resistance should be based on the calculated uplift resistance using the static pile uplift load test results.

Static uplift tests should be evaluated using a modified Davisson Method as described in Hannigan et al. (2006).

If using dynamic tests with signal matching to determine uplift resistance, it may be difficult to separate the measured end bearing resistance from the side resistance acting on the bottom section of the pile, especially if the soil stiffness at the pile tip is not significantly different from the soil stiffness acting on the sides of the pile near the pile tip. If it is not clear what is end bearing and what is side friction near the pile tip, the side resistance acting on the bottom pile element should be ignored when estimating uplift resistance using this method. If the pile length is shorter than 30 ft. in length, caution should be exercised when using dynamic tests with signal matching to estimate uplift.

10.7.3.11—Uplift Resistance of Pile Groups

The nominal uplift resistance of pile groups shall be evaluated when the foundation is subjected to uplift loads.

Pile group factored uplift resistance, in kips, shall be taken as:

$$R_R = \varphi R_n = \varphi_{ug} R_{ug} \qquad (10.7.3.11\text{-}1)$$

where:

φ_{ug} = resistance factor specified in Table 10.5.5.2.3-1

R_{ug} = nominal uplift resistance of the pile group (kips)

C10.7.3.11

A net uplift force can act on the foundation. An example of such a load is the construction load induced during the erection of concrete segmental girder bridges.

The nominal uplift resistance, R_{ug}, of a pile group shall be taken as the lesser of:

- The sum of the individual pile uplift resistance, or
- The uplift resistance of the pile group considered as a block.

For pile groups in cohesionless soil, the weight of the block that will be uplifted shall be determined using a spread of load of 1*H* in 4*V* from the base of the pile group taken from Figure 10.7.3.11-1. Buoyant unit weights shall be used for soil below the groundwater level.

In cohesive soils, the block used to resist uplift in undrained shear shall be taken from Figure 10.7.3.11-2. The nominal group uplift resistance may be taken as:

$$R_n = R_{ug} = (2XZ + 2YZ)\bar{S}_u + W_g \qquad (10.7.3.11\text{-}2)$$

where:

X = width of the group, as shown in Figure 10.7.3.11-2 (ft)

Y = length of the group, as shown in Figure 10.7.3.11-2 (ft)

Z = depth of the block of soil below pile cap taken from Figure 10.7.3.11-2 (ft)

$\bar{S}_u$ = average undrained shear strength along the sides of the pile group (ksf)

W_g = weight of the block of soil, piles, and pile cap (kips)

The resistance factor for the nominal group uplift resistance, R_{ug}, determined as the sum of the individual pile resistances, shall be taken as the same as that for the uplift resistance of single piles as specified in Table 10.5.5.2.3-1.

The resistance factor for the uplift resistance of the pile group considered as a block shall be taken as specified in Table 10.5.5.2.3-1 for pile groups in all soils.

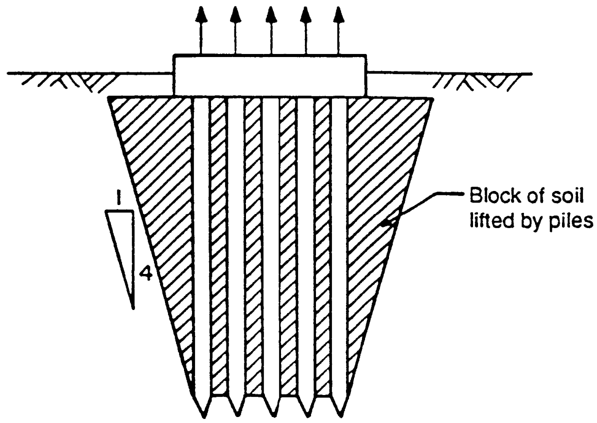


Figure 10.7.3.11-1—Uplift of Group of Closely Spaced Piles in Cohesionless Soils after Tomlinson (1987)

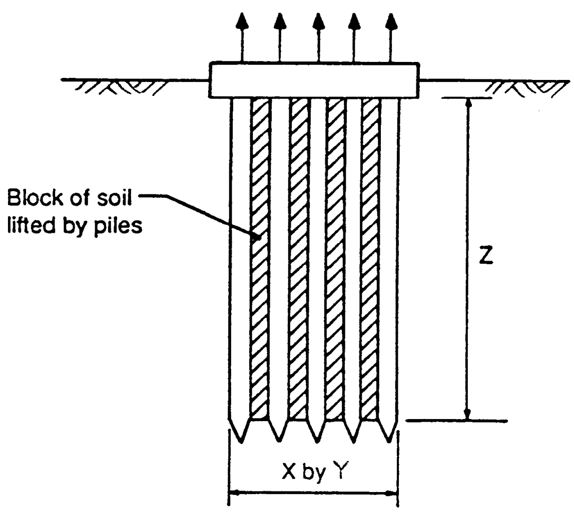


Figure 10.7.3.11-2—Uplift of Group of Piles in Cohesive Soils after Tomlinson (1987)

10.7.3.12—Nominal Lateral Resistance of Pile Foundations

The nominal resistance of pile foundations to lateral loads shall be evaluated based on both geomaterial and structural properties. The lateral soil resistance along the piles should be modeled using *P-y* curves developed for the soils at the site.

The applied loads shall be factored loads and they must include both lateral and axial loads. The analysis may be performed on a representative single pile with the appropriate pile top boundary condition or on the entire pile group. The *P-y* curves shall be modified for group effects. The *P*-multipliers in Table 10.7.2.4-1 should be used to modify the curves. If the pile cap will always be embedded, the *P-y* lateral resistance of the soil on the cap face may be included in the nominal lateral resistance.

C10.7.3.12

Pile foundations are subjected to lateral loads due to wind, traffic loads, bridge curvature, stream flow, vessel or traffic impact and earthquake. Batter piles are sometimes used but they are somewhat more expensive than vertical piles and vertical piles are more effective against dynamic loads.

Additional details regarding methods of analysis using *P-y* curves, both for single piles and pile groups, are provided in Article 10.7.2.4. As an alternative to *P-y* analysis, strain wedge theory may be used (see Article 10.7.2.4).

When this analysis is performed, the loads are factored since the strength limit state is under consideration, but the resistances as represented by the *P-y* curves are not factored since they already represent the ultimate condition.

The minimum penetration of the piles below ground (see Article 10.7.6) required in the contract should be established such that fixity is obtained. For this determination, the loads applied to the pile are factored as specified in Section 3, and a soil resistance factor of 1.0 shall be used as specified in Table 10.5.5.2.3-1.

If fixity cannot be obtained, additional piles should be added, larger diameter piles used if feasible to drive them to the required depth, or a wider spacing of piles in the group should be considered to provide the necessary lateral resistance. Batter piles may be added to provide the lateral resistance needed, unless downdrag is anticipated. If downdrag is anticipated, batter piles should not be used. The design procedure, if fixity cannot be obtained, should take into consideration the lack of fixity of the pile.

Lateral resistance of single piles may be determined by static load test. If a static lateral load test is to be performed, it shall follow the procedures specified in ASTM D3966.

The strength limit state for lateral resistance is only structural (see Sections 5 and 6 for structural limit state design requirements), though the determination of pile fixity is the result of soil-structure interaction. A failure of the soil does not occur; the soil will continue to displace at constant or slightly increasing resistance. Failure occurs when the pile reaches the structural limit state, and this limit state is reached, in the general case, when the nominal combined bending and axial resistance is reached.

If the lateral resistance of the soil in front of the pile cap is included in the lateral resistance of the foundation, the effect of soil disturbance resulting from construction of the pile cap should be considered. In such cases, the passive resistance may need to be reduced to account for the effects of disturbance.

For information on analysis and interpretation of load tests, see Article 10.7.2.4.

10.7.3.13—Pile Structural Resistance

10.7.3.13.1—Steel Piles

The nominal axial compression resistance in the structural limit state for piles loaded in compression shall be as specified in Article 6.9.4.1 for noncomposite piles and Article 6.9.5.1 for composite piles. If the pile is fully embedded, λ shall be taken as 0.

The nominal axial resistance of horizontally unsupported noncomposite piles that extend above the ground surface in air or water shall be determined from Eqs. 6.9.4.1.1-1 or 6.9.4.1.1-2. The nominal axial resistance of horizontally unsupported composite piles that extend above the ground surface in air or water shall be determined from Eqs. 6.9.5.1-1 or 6.9.5.1-2.

The effective length of laterally unsupported piles should be determined based on the provisions in Article 10.7.3.13.4.

The resistance factors for the compression limit state are specified in Article 6.5.4.2.

C10.7.3.13.1

Composite members refer to steel pipe piles that are filled with concrete.

The effective length given in Article C10.7.3.13.4 is an empirical approach to determining effective length. Computer methods are now available that can determine the axial resistance of a laterally unsupported compression member using a P-Δ analysis that includes a numerical representation of the lateral soil resistance (Williams et al., 2003). These methods are preferred over the empirical approach in Article C10.7.3.13.4.

10.7.3.13.2—Concrete Piles

The nominal axial compression resistance for concrete piles and prestressed concrete piles shall be as specified in Article 5.7.4.4.

The nominal axial compression resistance for concrete piles that are laterally unsupported in air or water shall be determined using the procedures given in Articles 5.7.4.3 and 4.5.3.2. The effective length of laterally unsupported piles should be determined based on the provisions in Article 10.7.3.13.4.

The resistance factor for the compression limit state for concrete piles shall be that given in Article 5.5.4.2.1 for concrete loaded in axial compression.

C10.7.3.13.2

Article 5.7.4 includes specified limits on longitudinal reinforcement, spirals and ties. Methods are given for determining nominal axial compression resistance but they do not include the nominal axial compression resistance of prestressed members. Article C5.7.4.1 notes that compression members are usually prestressed only where they are subjected to high levels of flexure. Therefore, a method of determining nominal axial compression resistance is not given.

Article 5.7.4.5 specifically permits an analysis based on equilibrium and strain compatibility. Methods are also available for performing a stability analysis (Williams et al., 2003).

10.7.3.13.3—Timber Piles

The nominal axial compression resistance for timber piles shall be as specified in Article 8.8.2. The methods presented there include both laterally supported and laterally unsupported members.

The effective length of laterally unsupported piles should be determined based on the provisions in Article 10.7.3.13.4.

C10.7.3.13.3

Article 8.5.2.3 requires that a reduction factor for long term loads of 0.75 be multiplied times the resistance factor for Strength Load Combination IV.

10.7.3.13.4—Buckling and Lateral Stability

In evaluating stability, the effective length of the pile shall be equal to the laterally unsupported length, plus an embedded depth to fixity.

The potential for buckling of unsupported pile lengths and the determination of stability under lateral loading should be evaluated by methods that consider soil-structure interaction as specified in Article 10.7.3.12.

C10.7.3.13.4

For preliminary design, the depth to fixity below the ground, in ft, may be taken as:

- For clays:

$$1.4\,[E_p\, l_w / E_s\,]^{0.25} \tag{C10.7.3.13.4-1}$$

- For sands:

$$1.8\,[E_p\, l_w / n_h\,]^{0.2} \tag{C10.7.3.13.4-2}$$

where:

E_p = modulus of elasticity of pile (ksi)

l_w = weak axis moment of inertia for pile (ft^4)

E_s = soil modulus for clays = 0.465 S_u (ksi)

S_u = undrained shear strength of clays (ksf)

n_h = rate of increase of soil modulus with depth for sands as specified in Table C10.4.6.3-2 (ksi/ft)

This procedure is taken from Davisson and Robinson (1965).

In Eqs. C10.7.3.13.4-1 and C10.7.3.13.4-2, the loading condition has been assumed to be axial load only, and the piles are assumed to be fixed at their ends. Because the equations give depth to fixity from the ground line, the Engineer must determine the boundary conditions at the top of the pile to determine the total unbraced length of the pile. If other loading or pile tip conditions exist, see Davisson and Robinson (1965).

The effect of pile spacing on the soil modulus has been studied by Prakash and Sharma (1990), who found that, at pile spacings greater than 8 times the pile width, neighboring piles have no effect on the soil modulus or buckling resistance. However, at a pile spacing of three times the pile width, the effective soil modulus is reduced to 25 percent of the value applicable to a single pile. For intermediate spacings, modulus values may be estimated by interpolation.

10.7.4—Extreme Event Limit State

The provisions of Article 10.5.5.3 shall apply.

For the applicable factored loads, including those specified in Article 10.7.1.6, for each extreme event limit state, the pile foundations shall be designed to have adequate factored axial and lateral resistance. For seismic design, all soil within and above the liquefiable zone, if the soil is liquefiable, shall not be considered to contribute bearing resistance. Downdrag resulting from liquefaction

C10.7.4

See Article C10.5.5.3.3.

induced settlement shall be determined as specified in Article 3.11.8 and included in the loads applied to the foundation. Static downdrag loads should not be combined with seismic downdrag loads due to liquefaction.

The pile foundation shall also be designed to resist the horizontal force resulting from lateral spreading, if applicable, or the liquefiable soil shall be improved to prevent liquefaction and lateral spreading. For lateral soil resistance of the pile foundation, the *P-y* curve soil parameters should be reduced to account for liquefaction. To determine the amount of reduction, the duration of strong shaking and the ability of the soil to fully develop a liquefied condition during the period of strong shaking should be considered.

When designing for scour, the pile foundation design shall be conducted as described in Article 10.7.3.6, except that the check flood and resistance factors consistent with Article 10.5.5.3.2 shall be used.

10.7.5—Corrosion and Deterioration

The effects of corrosion and deterioration from environmental conditions shall be considered in the selection of the pile type and in the determination of the required pile cross-section.

As a minimum, the following types of deterioration shall be considered:

- Corrosion of steel pile foundations, particularly in fill soils, low pH soils, and marine environments;
- Sulfate, chloride, and acid attack of concrete pile foundations; and
- Decay of timber piles from wetting and drying cycles or from insects or marine borers.

The following soil or site conditions should be considered as indicative of a potential pile deterioration or corrosion situation:

- Resistivity less than 2,000 ohm-cm,
- pH less than 5.5,
- pH between 5.5 and 8.5 in soils with high organic content,
- Sulfate concentrations greater than 1,000 ppm,
- Landfills and cinder fills,
- Soils subject to mine or industrial drainage,

C10.7.5

Resistivity, pH, chloride content, and sulfate concentration values have been adapted from those in Fang (1991) and Tomlinson (1987).

Some states use a coal tar epoxy paint system as a protective coating with good results.

The criterion for determining the potential for deterioration varies widely. An alternative set of recommendations is given by Elias (1990).

A field electrical resistivity survey or resistivity testing and pH testing of soil and groundwater samples may be used to evaluate the corrosion potential.

The deterioration potential of steel piles may be reduced by several methods, including protective coatings, concrete encasement, cathodic protection, use of special steel alloys, or increased steel area. Protective coatings should be resistant to abrasion and have a proven service record in the corrosive environment identified. Protective coatings should extend into noncorrosive soils a few feet because the lower portion of the coating is more susceptible to abrasion loss during installation.

Concrete encasement through the corrosive zone may also be used. The concrete mix should be of low permeability and placed properly. Steel piles protected by concrete encasement should be coated with a dielectric coating near the base of the concrete jacket.

The use of special steel alloys of nickel, copper, and potassium may also be used for increased corrosion resistance in the atmosphere or splash zone of marine piling.

Sacrificial steel area may also be used for corrosion resistance. This technique over sizes the steel section so that the available section after corrosion meets structural requirements.

- Areas with a mixture of high resistivity soils and low resistivity high alkaline soils, and
- Insects (wood piles).

The following water conditions should be considered as indicative of a potential pile deterioration or corrosion situation:

- Chloride content greater than 500 ppm,
- Sulfate concentration greater than 500 ppm,
- Mine or industrial runoff,
- High organic content,
- pH less than 5.5,
- Marine borers, and
- Piles exposed to wet/dry cycles.

When chemical wastes are suspected, a full chemical analysis of soil and groundwater samples shall be considered.

Deterioration of concrete piles can be reduced by design procedures. These include use of a dense impermeable concrete, sulfate resisting Portland cement, increased steel cover, air-entrainment, reduced chloride content in the concrete mix, cathodic protection, and epoxy-coated reinforcement. Piles that are continuously submerged are less subject to deterioration. ACI 318, Section 4.5.2, provides maximum water-cement ratio requirements for special exposure conditions. ACI 318, Section 4.5.3, lists the appropriate types of cement for various types of sulfate exposure.

For prestressed concrete, ACI 318 recommends a maximum water-soluble chloride ion of 0.06 percent by weight of cement.

Cathodic protection of reinforcing and prestressing steel may be used to protect concrete from corrosion effects. This process induces electric flow from the anode to the cathode of the pile and reduces corrosion. An external DC power source may be required to drive the current. However, cathodic protection requires electrical continuity between all steel and that necessitates bonding the steel for electric connection. This bonding is expensive and usually precludes the use of cathodic protection of concrete piles.

Epoxy coating of pile reinforcement has been found in some cases to be useful in resisting corrosion. It is important to ensure that the coating is continuous and free of holidays.

More detail on design for corrosion or other forms of deterioration is contained in Hannigan et al. (2006).

10.7.6—Determination of Minimum Pile Penetration

The minimum pile penetration, if required for the particular site conditions and loading, shall be based on the maximum depth (i.e., tip elevation) needed to meet the following requirements as applicable:

- Single and pile group settlement (service limit state)
- Lateral deflection (service limit state)
- Uplift (strength limit state)
- Penetration into bearing soils needed to get below soil causing downdrag loads on the pile foundation resulting from static consolidation stresses on soft soil or downdrag loads due to liquefaction (strength and extreme event limit state, respectively)
- Penetration into bearing soils needed to get below soil subject to scour
- Penetration into bearing soils necessary to obtain fixity for resisting the applied lateral loads to the foundation (strength limit state)

C10.7.6

A minimum pile penetration should only be specified if necessary to ensure that all of the applicable limit states are met. A minimum pile penetration should not be specified solely to meet axial compression resistance, i.e., bearing, unless field verification of the pile nominal bearing resistance is not performed as described in Article 10.7.3.8.

- Axial uplift, and nominal lateral resistance to resist extreme event limit state loads

The contract documents should indicate the minimum pile penetration, if applicable, as determined above only if one or more of the requirements listed above are applicable to the pile foundation. The contract documents should also include the required nominal axial compressive resistance, R_{ndr} as specified in Article 10.7.7 and the method by which this resistance will be verified, if applicable, such that the resistance factor(s) used for design are consistent with the construction field verification methods of nominal axial compressive pile resistance.

10.7.7—Determination of R_{ndr} Used to Establish Contract Driving Criteria for Nominal Bearing Resistance

The value of R_{ndr} used for the construction of the pile foundation to establish the driving criteria to obtain the nominal bearing resistance shall be the value that meets or exceeds the following limit states, as applicable:

- Strength limit state nominal bearing resistance specified in Article 10.7.3.8
- Strength limit state nominal bearing resistance, including downdrag specified in Article 10.7.3.7
- Strength limit state nominal bearing resistance, accounting for scour specified in Article 10.7.3.6
- Extreme event limit state nominal bearing resistance for seismic specified in Article 10.7.4
- Extreme event limit state nominal bearing resistance for scour specified in Article 10.7.4

10.7.8—Drivability Analysis

The establishment of the installation criteria for driven piles should include a drivability analysis. Except as specified herein, the drivability analysis shall be performed by the Engineer using a wave equation analysis, and the driving stresses (σ_{dr}) anywhere in the pile determined from the analysis shall be less than the following limits:

Steel Piles, compression and tension:

$$\sigma_{dr} = 0.9\varphi_{da} f_y \quad (10.7.8\text{-}1)$$

where:

f_y = yield strength of the steel (ksi)

C10.7.8

Wave equation analyses should be conducted during design using a range of likely hammer/pile combinations, considering the soil and installation conditions at the foundation site. See Article 10.7.3.8.4 for additional considerations for conducting wave equation analyses. These analyses should be used to assess feasibility of the proposed foundation system and to establish installation criteria with regard to driving stresses to limit driving stresses to acceptable levels. For routine pile installation applications, e.g., smaller diameter, low nominal resistance piles, the development of installation criteria with regard to the limitation of driving stresses, e.g., minimum or maximum ram weight, hammer size, maximum acceptable driving resistance, etc., may be based on local experience,

φ_{da} = resistance factor as specified in Table 10.5.5.2.3-1

Concrete piles:

- In compression:

$$\sigma_{dr} = \varphi_{da} 0.85 f_c' \tag{10.7.8-2}$$

- In tension, considering only the steel reinforcement:

$$\sigma_{dr} = 0.7\varphi_{da} f_y \tag{10.7.8-3}$$

where:

f'_c = compressive strength of the concrete (ksi)

f_y = yield strength of the steel reinforcement (ksi)

Prestressed concrete piles, normal environments:

- In compression:

$$\sigma_{dr} = \varphi_{da} \left(0.85 f_c' - f_{pe}\right) \tag{10.7.8-4}$$

- In tension:

$$\sigma_{dr} = \varphi_{da} \left(0.095\sqrt{f_c'} + f_{pe}\right) \tag{10.7.8-5}$$

where:

f_{pe} = effective prestressing stress in concrete (ksi)

Prestressed concrete piles, severe corrosive environments:

- In tension:

$$\sigma_{dr} = \varphi_{da} f_{pe} \tag{10.7.8-6}$$

Timber piles, in compression and tension:

$$\sigma_{dr} = \varphi_{da} (F_{co}) \tag{10.7.8-7}$$

where:

F_{co} = base resistance of wood in compression parallel to the grain as specified in Article 8.4.1.3 (ksi)

rather than conducting a detailed wave equation analysis that is project specific. Local experience could include previous drivability analysis results and actual pile driving experience that are applicable to the project specific situation at hand. Otherwise, a project specific drivability study should be conducted.

Drivability analyses may also be conducted as part of the project construction phase. When conducted during the construction phase, the drivability analysis shall be conducted using the contractor's proposed driving system. This information should be supplied by the contractor. This drivability analysis should be used to determine if the contractor's proposed driving system is capable of driving the pile to the maximum resistance anticipated without exceeding the factored structural resistance available, i.e., σ_{dr}.

In addition to this drivability analysis, the best approach to controlling driving stresses during pile installation is to conduct dynamic testing with signal matching to verify the accuracy of the wave equation analysis results. Note that if a drivability analysis is conducted using the wave equation for acceptance of the contractor's proposed driving system, but a different method is used to develop driving resistance, i.e., blow count, criterion to obtain the specified nominal pile resistance, e.g., a driving formula, the difference in the methods regarding the predicted driving resistance should be taken into account when evaluating the contractor's driving system. For example, the wave equation analysis could indicate that the contractor's hammer can achieve the desired bearing resistance, but the driving formula could indicate the driving resistance at the required nominal bearing is too high. Such differences should be considered when setting up the driving system acceptance requirements in the contract documents, though it is preferable to be consistent in the method used for acceptance of the contractor's driving system and the one used for developing driving criteria.

The selection of a blow count limit as a definition of refusal is difficult because it can depend on the site soil profile, the pile type, hammer performance, and possibly hammer manufacturer limitations to prevent hammer damage. In general, blow counts greater than 10–15 blows per inch should be used with care, particularly with concrete or timber piles. In cases where the driving is easy until near the end of driving, a higher blow count may sometimes be satisfactory, but if a high blow count is required over a large percentage of the depth, even ten blows per inch may be too large.

This drivability analysis shall be based on the maximum driving resistance needed:

- To obtain minimum penetration requirements specified in Article 10.7.6,
- To overcome resistance of soil that cannot be counted upon to provide axial or lateral resistance throughout the design life of the structure, e.g., material subject to scour, or material subject to downdrag, and
- To obtain the required nominal bearing resistance.

10.7.9—Probe Piles

Probe piles should be driven at several locations on the site to establish order length. If dynamic measurements are not taken, these probe piles should be driven after the driving criteria have been established.

If dynamic measurements during driving are taken, both order lengths and driving criteria should be established after the probe pile(s) are driven.

C10.7.9

Probe piles are sometimes known as test piles or indicator piles. It is common practice to drive probe piles at the beginning of the project (particularly with concrete piles) to establish pile order lengths and/or to evaluate site variability whether or not dynamic measurements are taken.

10.8—DRILLED SHAFTS

10.8.1—General

10.8.1.1—Scope

The provisions of this Section shall apply to the design of drilled shafts. Throughout these provisions, the use of the term “drilled shaft” shall be interpreted to mean a shaft constructed using either drilling (open hole or with drilling slurry) or casing plus excavation equipment and technology.

These provisions shall also apply to shafts that are constructed using casing advancers that twist or rotate casings into the ground concurrent with excavation rather than drilling.

The provisions of this Section shall not be taken as applicable to drilled piles, e.g., augercast piles, installed with continuous flight augers that are concreted as the auger is being extracted.

C10.8.1.1

Drilled shafts may be an economical alternative to spread footing or pile foundations, particularly when spread footings cannot be founded on suitable soil or rock strata within a reasonable depth or when driven piles are not viable. Drilled shafts may be an economical alternative to spread footings where scour depth is large. Drilled shafts may also be considered to resist high lateral or axial loads, or when deformation tolerances are small. For example, a movable bridge is a bridge where it is desirable to keep deformations small.

Drilled shafts are classified according to their primary mechanism for deriving load resistance either as floating (friction) shafts, i.e., shafts transferring load primarily by side resistance, or end-bearing shafts, i.e., shafts transferring load primarily by tip resistance.

It is recommended that the shaft design be reviewed for constructability prior to advertising the project for bids.

10.8.1.2—Shaft Spacing, Clearance, and Embedment into Cap

If the center-to-center spacing of drilled shafts is less than 4.0 diameters, the interaction effects between adjacent shafts shall be evaluated. If the center-to-center spacing of drilled shafts is less than 6.0 diameters, the sequence of construction should be specified in the contract documents.

Shafts used in groups should be located such that the distance from the side of any shaft to the nearest edge of the cap is not less than 12.0 in. Shafts shall be embedded sufficiently into the cap to develop the required structural resistance.

C10.8.1.2

Larger spacing may be required to preserve shaft excavation stability or to prevent communication between shafts during excavation and concrete placement.

Shaft spacing may be decreased if casing construction methods are required to maintain excavation stability and to prevent interaction between adjacent shafts.

10.8.1.3—Shaft Diameter and Enlarged Bases

If the shaft is to be manually inspected, the shaft diameter should not be less than 30.0 in. The diameter of columns supported by shafts should be smaller than or equal to the diameter of the drilled shaft.

C10.8.1.3

Nominal shaft diameters used for both geotechnical and structural design of shafts should be selected based on available diameter sizes.

If the shaft and the column are the same diameter, it should be recognized that the placement tolerance of drilled shafts is such that it will likely affect the column location. The shaft and column diameter should be determined based on the shaft placement tolerance, column and shaft reinforcing clearances, and the constructability of placing the column reinforcing in the shaft. A horizontal construction joint in the shaft at the bottom of the column reinforcing will facilitate constructability. Making allowance for the tolerance where the column connects with the superstructure, which could affect column alignment, can also accommodate this shaft construction tolerance.

In drilling rock sockets, it is common to use casing through the soil zone to temporarily support the soil to prevent cave-in, allow inspection and to produce a seal along the soil-rock contact to minimize infiltration of groundwater into the socket. Depending on the method of excavation, the diameter of the rock socket may need to be sized at least 6 in. smaller than the nominal casing size to permit seating of casing and insertion of rock drilling equipment.

In stiff cohesive soils, an enlarged base (bell, or underream) may be used at the shaft tip to increase the tip bearing area to reduce the unit end bearing pressure or to provide additional resistance to uplift loads.

Where the bottom of the drilled hole is dry, cleaned and inspected prior to concrete placement, the entire base area may be taken as effective in transferring load.

Where practical, consideration should be given to extension of the shaft to a greater depth to avoid the difficulty and expense of excavation for enlarged bases.

10.8.1.4—Battered Shafts

Battered shafts should be avoided. Where increased lateral resistance is needed, consideration should be given to increasing the shaft diameter or increasing the number of shafts.

C10.8.1.4

Due to problems associated with hole stability during excavation, installation, and with removal of casing during installation of the rebar cage and concrete placement, construction of battered shafts is very difficult.

Allen, T. M. 2005. *Development of Geotechnical Resistance Factors and Downdrag Load Factors for LRFD Foundation Strength Limit State Design*, FHWA-NHI-05-052, Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Allen, T. M., 2007, “Development of New Pile-Driving Formula and Its Calibration for Load and Resistance Factor Design,” *Transportation Research Record: Journal of the Transportation Research Board*, No. 2004, Transportation Research Board of the National Academies, Washington, D.C., pp. 20–27.

Arman, A., N. Samtani, R. Castelli, and G. Munfakh. 1997. *Subsurface Investigations: Training Course in Geotechnical and Foundation Engineering*, FHWA-HI-97-021. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Ashour, M., G. M. Norris, and P. Pilling. 1998, “Lateral Loading of a Pile in Layered Soil Using the Strain Wedge Model,” *ASCE Journal of Geotechnical and Geoenvironmental Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 124, No. 4, pp. 303–315.

ASTM D6429, *Standard Guide for Selecting Surface Geophysical Methods*

Baguelin, F., J. F. Jezequel, and D. H. Shields. 1987. *The Pressuremeter and Foundation Engineering*. Trans Tech Publications, Clausthal-Zellerfeld, Germany, p. 617.

Barkdale, R. D., and R. C. Bachus. 1983. *Design and Construction of Stone Columns—Vol. 1*, FHWA/RD-83/02C. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Barker, R. M., J. M. Duncan, K. B. Rojiani, P. S. K. Ooi, C. K. Tan, and S. G. Kim. 1991. *Manuals for the Design of Bridge Foundations*. *NCHRP Report 343*. Transportation Research Board, National Research Council, Washington, DC.

Barton, N. R. 1976. “The Shear Strength of Rock and Rock Joints,” *Engineering Geology*. Elsevier Science Publishing Company, Inc., New York, NY, Vol. 7, pp. 287–332.

Bieniawski, Z. T. 1984. *Rock Mechanics Design in Mining and Tunneling*. A. A. Balkema, Rotterdam/Boston, p. 272.

Bogard, J. D., and H. Matlock. 1990. “Application of Model Pile Tests to Axial Pile Design.” In *Proc., 22nd Annual Offshore Technology Conference*, Houston, TX, Vol. 3, pp. 271–278.

Boulanger, R. W., and I. M. Idriss. 2006. “Liquefaction Susceptibility Criteria for Silts and Clays,” *Journal of Geotechnical and Geoenvironmental Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 132, No. 11, pp. 1413–1426.

Bowles, J. E. 1977. *Foundation Analysis and Design*, Second Edition. McGraw–Hill Book Company, New York, NY.

Bowles, J. E. 1988. *Foundation Analysis and Design*, 4th Edition. McGraw–Hill Book Company, New York, NY, p. 1004.

Bray, J. D., and R. B. Sancio. 2006. “Assessment of the Liquefaction Susceptibility of Fine-Grained Soils,” *Journal of Geotechnical and Geoenvironmental Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 132, No. 9, pp. 1165–1176.

Bray, J. D., R.B. Sancio, M. F. Riemer, and T. Durgunoglu. 2004. “Liquefaction Susceptibility of Fine-Grained Soils.” In *Proc., 11th International Conference on Soil Dynamics and Earthquake Engineering and 3rd International Conference On Earthquake Geotechnical Engineering*, Berkeley, CA, January 2004, Doolin, Kammerer, Nogami, Seed and Towhata, Eds., Vol. 1, pp. 655–662.

Broms, B. B. 1964a. “Lateral Resistance of Piles in Cohesive Soil,” *ASCE Journal for Soil Mechanics and Foundation Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 90, SM2, pp. 27–63.

Broms, B.B. 1964b. “Lateral Resistance of Piles in Cohesionless Soil,” *ASCE Journal for Soil Mechanics and Foundation Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 90, SM3, pp. 123–156.

Campanella, R. G. 1994. "Field Methods for Dynamic Geotechnical Testing: An Overview of Capabilities and Needs." In *Dynamic Geotechnical Testing II,* Special Technical Publication No. 1213, American Society for Testing and Materials, Philadelphia, PA, pp. 3–23.

Canadian Geotechnical Society. 1985. *Canadian Foundation Engineering Manual*, Second Edition. Bitech Publishers, Ltd., Vancouver, British Columbia, Canada, p. 460.

Carter, J. P., and F. H. Kulhawy. 1988. *Analysis and Design of Foundations Socketed into Rock*, Report No. EL-5918. Empire State Electric Engineering Research Corporation and Electric Power Research Institute, New York, NY, p. 158.

Cetin, K. O., R. B. Seed, A. Der Kiureghian, K. Tokimatsu, L. F, Harder, R. E. Kayen, and R. E. S. Moss. 2004. "Standard Penetration Test-Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential," *Journal of Geotechnical and Geoenvironmental Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 130, No. 12, pp. 1314–1340.

Cheney, R. and R. Chassie. 2000. *Soils and Foundations Workshop Reference Manual*, NHI-00-045. National Highway Institute, Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Davisson, M. T. 1972. "High Capacity Piles." *Proceedings Lecture Series, Innovations in Foundation Construction*, Illinois Section, American Society of Civil Engineers, Reston, VA.

Davisson, M. T., and K. E. Robinson. 1965. "Bending and Buckling of Partially Embedded Piles." In *Proc., Sixth International Conference S. M. and F. E.* University of Toronto Press, Montreal, Canada, pp. 243–246.

DiMillio, A. F. 1982. *Performance of Highway Bridge Abutments Supported by Spread Footings on Compacted Fill*, FHWA/RD-81/184 (NTIS PB83-201822). FHWA Staff Study.

Duncan, J. M., and A. L. Buchignani. 1976. *An Engineering Manual for Settlement Studies*. Department of Civil Engineering, University of California at Berkley, Berkley, CA, p. 94. Geotechnical engineering report.

Duncan, J. M. 2000. "Factors of Safety and Reliability in Geotechnical Engineering," *Journal of Geotechnical and Geoenvironmental Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 126, No. 4, pp. 307–316.

Elias, V. 1990. Durability*/Corrosion of Soil Reinforced Structures*, FHWA/R-89/186. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, p. 173.

Elias, V., J. Welsh, J. Warren, and R. Lukas. 2000. *Ground Improvement Technical Summaries—Vols. 1 and 2*, Demonstration Project 116, FHWA-SA-98-086. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

EPRI. 1983. *Transmission Line Structure Foundations for Uplift-Compression Loading*, EL-2870. Electric Power Research Institute, Palo Alto, CA.

Esrig, M. E., and R. C. Kirby. 1979. "Advances in General Effective Stress Method for the Prediction of Axial Capacity for Driven Piles in Clay." In *Proc., 11th Annual Offshore Technology Conference*, pp. 437–449.

Fang, H. Y. 1991. *Foundation Engineering Handbook*, Second Edition. Van Nostrand Reinhold, New York, NY.

Gifford, D. G., J. R. Kraemer, J. R. Wheeler, and A. F. McKown. 1987. Spread *Footings for Highway Bridges*, FHWA/RD-86/185. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, p. 229.

Goodman, R. E. 1989. "Introduction." In *Rock Mechanics*, Second Edition, John Wiley and Sons, Inc., New York, NY, p. 562.

Hannigan, P. J., G. G. Goble, G. Thendean, G. E. Likins, and F. Rausche. 2006. *Design and Construction of Driven Pile Foundations*, FHWA-NHI-05-042 and NHI-05-043, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., Vols. I and II.