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ERRATA AND SUPPLEMENT 3

Section 2.3.2.b, Number 3, Spatial Coherence, second sentence:
Replace:

"For example, 3 second gusts are coherent over shorter distances and therefore affect similar elements…"

with

"For example, 3-second gusts are coherent over shorter distances and therefore affect smaller elements…"

Add:

"(ft)" to end of the 2nd and 3rd bullets after "The recommended coherence spectrum…"

Replace Equation 2.3.2-6 with:

$$\mathrm{coh}(f) = \exp\left\{ -\frac{1}{U_O / 3.28} \times \left[\sum_{i=1}^{3} A_i^2 \right]^{\frac{1}{2}} \right\}$$

Replace Equation 2.3.2-7 with:

$$A_i = \alpha_i \times f^{r_i} \times \frac{\Delta_i^{q_i}}{3.28} \times z_g^{-p_i}$$

Table at the end of the section, change:

"9.66" *to* "13.0"

Section 3.3.1.c, first paragraph, change:

"(Ref. Section 6.7.1)," *to* "(Ref. Section 6.8.1),"

Section 3.4.1.c.2, second paragraph, replace:

"the following table" *with* "Table 3.4.1-1"

Section 4.3.2, equation legend for 4.3-1a and 1b, replace:

"F_y" *with* "F_{yc}"

Figure 4.2-2, see revised figure.

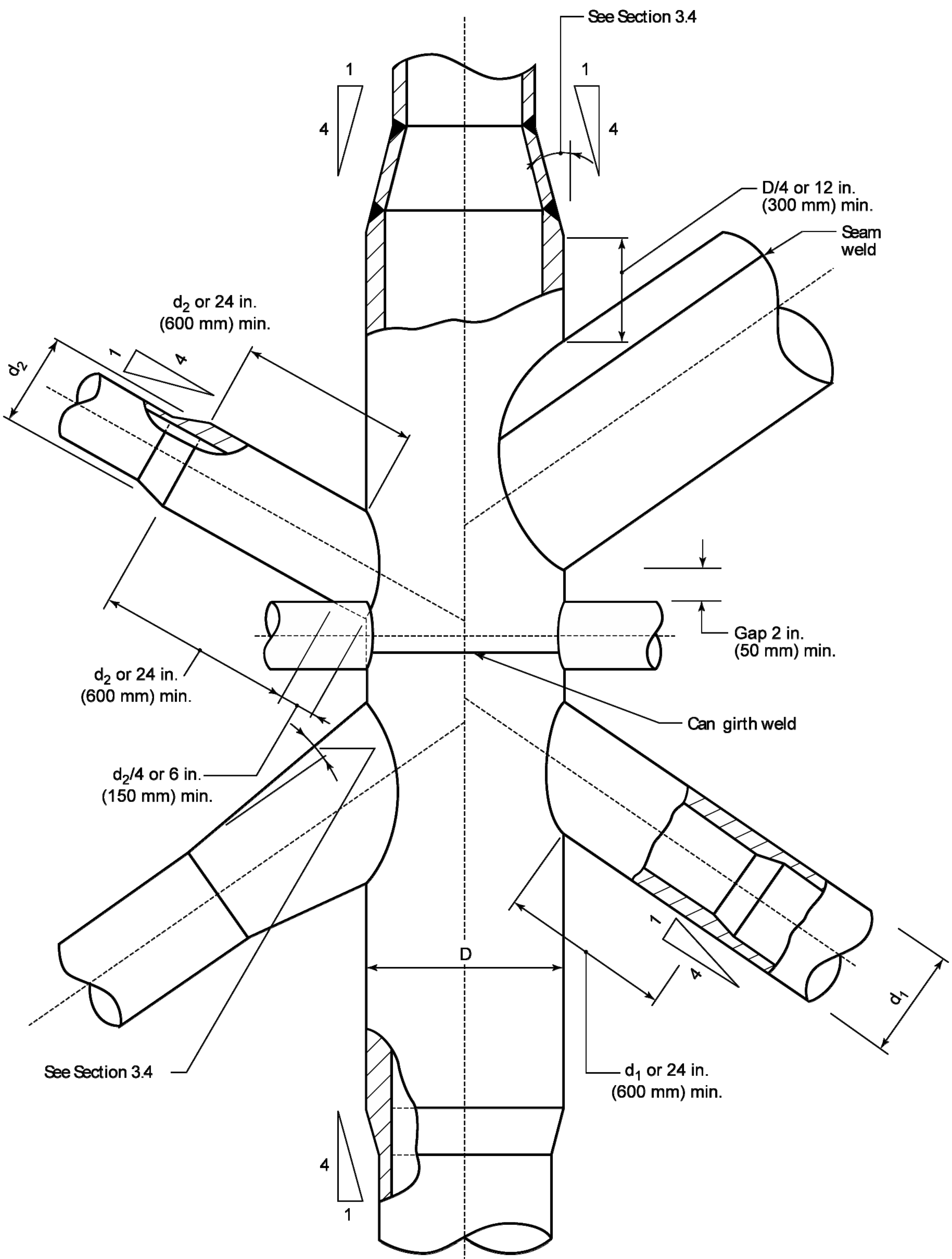
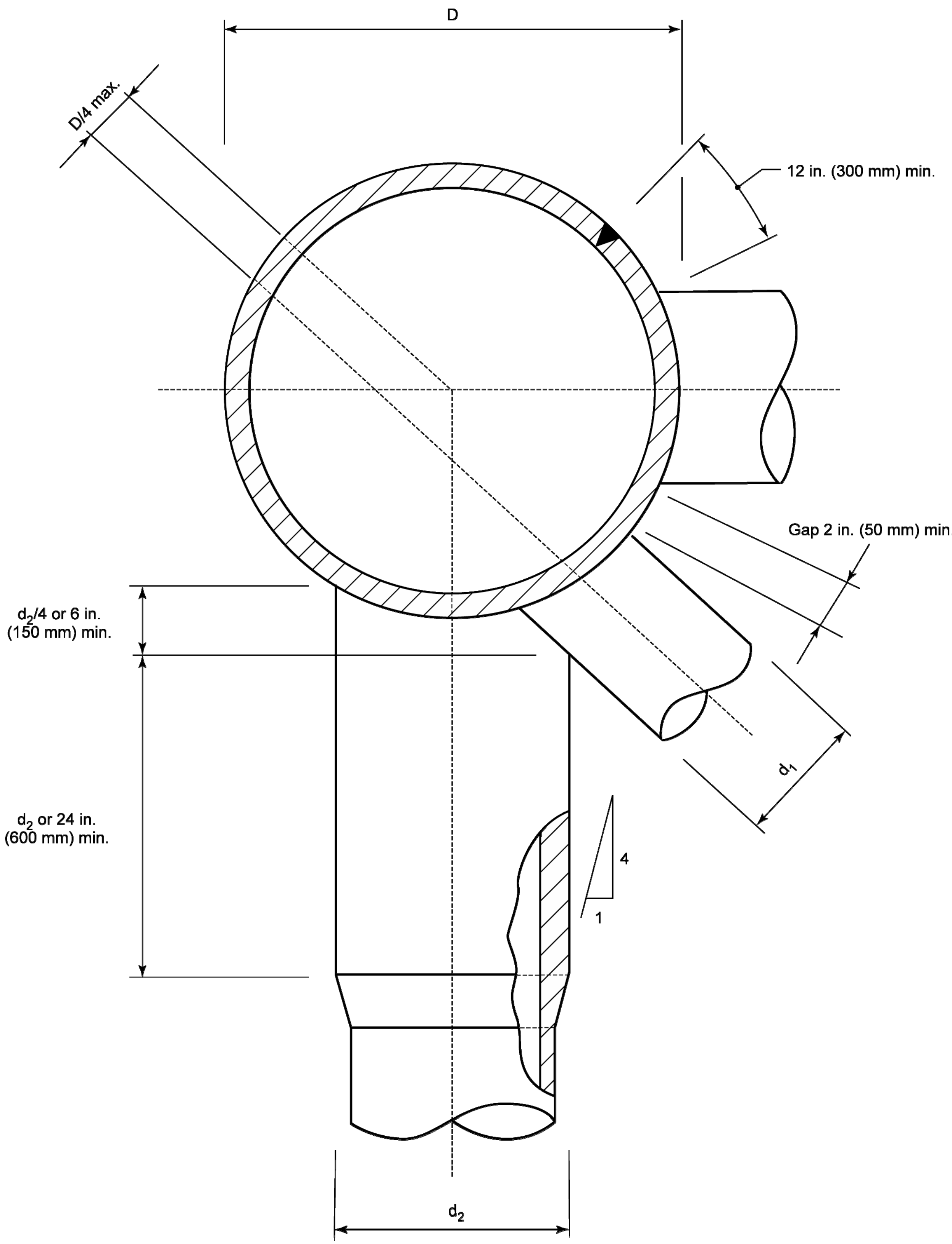


Figure 4.2-3, see revised figure, replace:
"6 in." dimension in the upper right corner *with* "12 in. (300 mm) min."



Section 4.3.4, Equation 4.3-3, replace:
"M_{ipb}" *with* "M_c"

Table 4.3-1, replace equation:
"20.7 + (β - 0.9)

(17γ – 220) for β > 0.9"

with

"20.7 + (β – 0.9) (17γ – 220) for β > 0.9"

Table 4.5-1, add:
"where

$$K_a = \frac{1}{2}\left(1 + 1/\sin\theta\right)$$"

Section 5.5.1, last paragraph, last two sentences, replace with:
"Plots of the *WJ* curves versus data, and information concerning *S-N* curves for joints in seawater without adequate corrosion protection is given in the Commentary.

Fabrication of welded joints should be in accordance with Section 11. The curve for cast joints is only applicable to castings having an adequate fabrication inspection plan; see Commentary."

Table 5.5.3-1 add:
Footnote a on $\tau^{-0.1}$ to read "Chord side only."

Section 6:
This section has been modified. A complete version of the following appears at the end of the Errata/Supplement:

- Sections 6.1 through 6.4.2 are unchanged
- Section 6.4.3 has been revised
- Sections 6.4.4 through 6.7.3 are unchanged
- Section 6.8.1 has been revised
- Section 6.8.2 is unchanged
- Section 6.8.3 has been revised
- Sections 6.8.4 through 6.17 are unchanged

Table 8.1.4-1, remove grades:
"Spec 2W, Grades 42 and 50T; Spec 2Y, Grades 42 and 50T"

Section 8.3.1, item 2, add:
"(47 Joules)" *after* "35 ft-lbs"

Section 8.4.1, change:
"ASTM 109" *to* "ASTM C109"

Section 11.1.2a, replace:
"Pipe splices should be in accordance with the requirements of Section 3.8, API Spec. 2B, Fabricates Structural Steel Pipe."

with

"Pipe splices should be in accordance with the requirements of API Spec 2B."

Section 11.1.3.d, replace first sentence with:
"Where controlled weld profiling has been considered in the fatigue analysis incorporating moderated thickness effect (see 5.5.2) or profile improvement factor (see 5.5.3), a capping layer should be applied so that the as-welded surface merges smoothly with the adjoining base metal and approximates the concave profiles shown in Figure 11.1.3."

Section 11.1.4, second paragraph, third sentence, replace:
"necessary" *with* "unnecessary"

Section C2.3.1b8, first bullet, replace:
"*A/S* > 2/5" *with* "*A/S* > 2.5"

Section C2.3.6d, item 2, second paragraph, replace:
"Example configurations which do not meet the guidelines are shown in Figure C2.3.6-5."

with

"Example configurations which meet the guidelines are shown in Figure C.2.3.6-5."

Section C2.3.6e, item 2, eighth paragraph, first sentence, replace:
"The use of none-third increase in ..." *with* "The use of one-third increase in ..."

C2, references, (61), change to:
"ASCE 4-98, *Seismic Analysis of Safety-Related Nuclear Structures and Commentary*, ASCE, 1801 Alexander Graham Bell Drive, Reston, VA 20191-4400."

Section C.3.2.2, first paragraph, fifth sentence, replace:
"The normalized rotational capacity, defined as ultimate to yield rotation ratio, (θ_u/θ_y), invariable…"

with

"The normalized rotational capacity, defined as ultimate to yield rotation ratio, (θ_u/θ_y), invariably…"

Section C.3.3.1, change:
"C.3.3.1" *to* "C.3.3.3"

Section C.3.3.2, change:
"C.3.3.2" *to* "C.3.3.4"

Section C4.2.1, third paragraph, change:
"Ksi" *to* "ksi" *and* "Mpa" *to* "MPa"

Section C4.2.4, Joint Classification
Insert new Figures C4.2-1 and C4.2-2

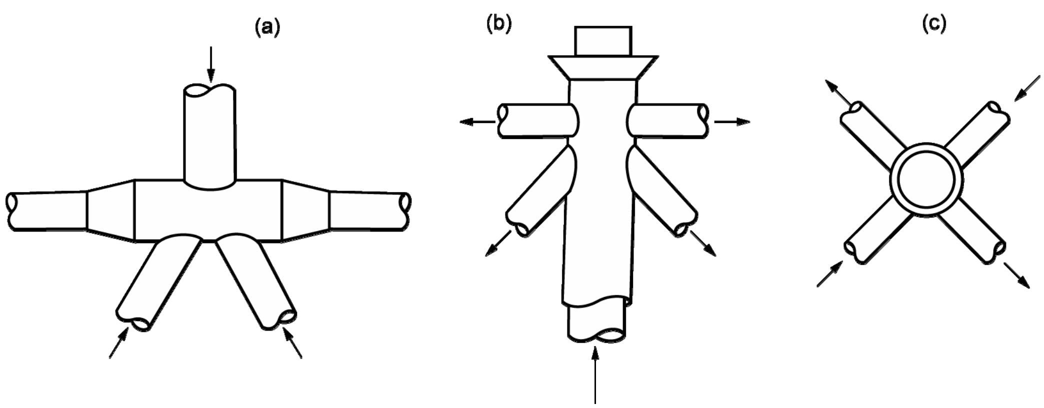


Figure C4.2-1—Adverse Load Patterns with α Up to 3.8 (a) False Leg Termination, (b) Skirt Pile Bracing, (c) Hub Connection

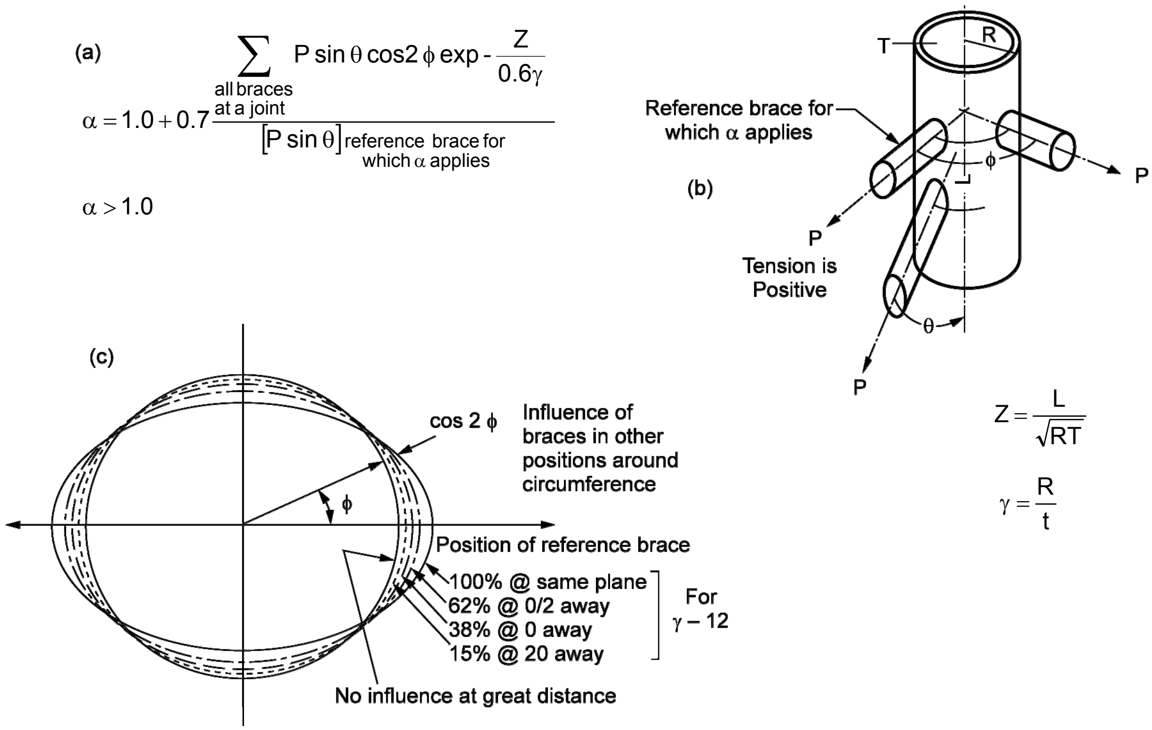


Figure C4.2-2—Computed α (a) Equation, (b) Definitions, (c) Influence Surface

Renumber the Figures (those published in the second Errata/Supplement as C4.2-1 and C4.2-2) as C4.3.3-1 and C4.3.3-2, respectively

Figure C4.3.2-2 — Safety Index Betas, API RP2A…

Change the Title of the Figure to read as follows:

"Safety Index Betas, API RP 2A Edition 21, Supplement 2 Formulation"

Section C4.3.3, twelfth paragraph, change:

"Figure C4.2-1" *to* "Figure C4.3.3-1" *and* "Figure C4.2-2" *to* "Figure C4.3.3-2"

Section C4.3.3, thirteenth paragraph, change:

"Design for axial load in General and Multiplanar connections"

to

"**C4.3.3.1 Design for Axial Load in General and Multiplanar Connections**"

and

"Q_β = defined in 4.3.3(a)" *to* "Q_β = defined in Table 4.3-1, note (a)"

Section C4.3.4, change and renumber throughout:

"Figure C4.3-3" *to* "Figure C4.3.4-1"
"Figure C4.3-4" *to* "Figure C4.3.4-2"
"Figure C4.3-5" *to* "Figure C4.3.4-3"
"Figure C4.3-6" *to* "Figure C4.3.4-4"
"Figure C4.3-7" *to* "Figure C4.3.4-5"
"Figure C4.3-8" *to* "Figure C4.3.4-6"
"Figure C4.3-9" *to* "Figure C4.3.4-7"

Equation C5.1-1, change:

$$\left|SCF_{ax} f_{ax}\right| + \sqrt{\left[\left(SCF_{ipb} f_{ipb}\right) + \left(SCF_{opb} f_{opb}\right)\right]}$$

to

$$\left|SCF_{ax} f_{ax}\right|\sqrt{\left[\left(SCF_{ipb} f_{ipb}\right)^2 + \left(SCF_{opb} f_{opb}\right)^2\right]}$$

Section C17.6.2, item b, change:

"Section C3.2.1" *to* "Section 2.3.1 or C2.3.1"

6 Foundation Design

Note: Sections 6.1 through 6.4.2 are unchanged from 21st Edition.

Replace Section 6.4.3 with the following:

6.4.3 Shaft Friction and End Bearing in Cohesionless Soils

This section provides a simple method for assessing pile capacity in cohesionless soils. The Commentary presents other, recent and more reliable methods for predicting pile capacity. These are based on direct correlations of pile unit friction and end bearing data with cone penetration test (CPT) results. In comparison to the Main Text method described below, these CPT-based methods are considered fundamentally better, have shown statistically closer predictions of pile load test results and, although not required, are in principle the preferred methods. These methods also cover a wider range of cohesionless soils than the Main Text method. However, offshore experience with these CPT methods is either limited or does not exist and hence more experience is needed before they are recommended for routine design, instead of the main text method. CPT-based methods should be applied only by qualified engineers who are experienced in the interpretation of CPT data and understand the limitations and reliability of these methods. Following installation, pile driving (instrumentation) data may be used to give more confidence in predicted capacities.

For pipe piles in cohesionless soils, the unit shaft friction at a given depth, f, may be calculated by the equation:

$$f = \beta \, p_o' \qquad (6.4.3\text{-}1)$$

where

β = dimensionless shaft friction factor,

p_o' = effective overburden pressure at the depth in question.

Table 6.4.3-1 may be used for selection of β values for open-ended pipe piles driven unplugged if other data are not available. Values of β for full displacement piles (i.e., driven fully plugged or closed ended) may be assumed to be 25 % higher than those given in Table 6.4.3-1. For long piles, f may not increase linearly with the overburden pressure as implied by Equation 6.4.3-1. In such cases, it may be appropriate to limit f to the values given in Table 6.4.3-1.

For piles end bearing in cohesionless soils, the unit end bearing q may be computed by the equation:

$$q = N_q \, p_o' \qquad (6.4.3\text{-}2)$$

where

N_q = dimensionless bearing capacity factor,

p_o' = effective overburden pressure at the depth in question.

Recommended N_q values are presented in Table 6.4.3-1. For long piles, q may not increase linearly with the overburden pressure as implied by Equation 6.4.3-2. In such cases it may be appropriate to limit q to the values given in Table 6.4.3-1. For plugged piles, the unit end bearing q acts over the entire cross section of the pile. For unplugged piles, q acts on the pile annulus only. In this case, additional resistance is offered by friction between soil plug and inner pile wall. Whether a pile is considered to be plugged or unplugged may be based on static calculations using a unit skin friction on the soil plug equal to the outer skin friction. It is noted that a pile could be driven in an unplugged condition but can act plugged under static loading.

Load test data for piles in sand (e.g., see Comparison of Measured and Axial Load Capacities of Steel Pipe Piles in Sand with Capacities Calculated Using the 1986 API RP 2A Standard, Final Report to API, Dec. 1987, by R. E. Olson and A Review of Design Methods for Offshore Driven Piles in Siliceous Sand, September 2005, by B. M. Lehane et al.) indicate that variability in capacity predictions using the Main Text method may exceed those for piles in clay. These data also indicate that the above method is conservative for short offshore piles [< 150 ft (45 m)] in dense to very dense sands loaded in compression and may be unconservative in all other conditions. In unfamiliar situations, the designer may want to account for this uncertainty through a selection of conservative design parameters and/or higher safety factors.

For soils that do not fall within the ranges of soil density and description given in Table 6.4.3-1, or for materials with unusually weak grains or compressible structure, Table 6.4.3-1 may not be appropriate for selection of design parameters. For example, very loose silts or soils containing large amounts of mica or volcanic grains may require special laboratory or field tests for selection of design parameters. Of particular importance are sands containing calcium carbonate, which are found extensively in many areas of the oceans. Experience suggests that driven piles in these soils may have substantially lower design strength parameters than given in Table 6.4.3-1. Drilled and grouted piles in carbonate sands, however, may have significantly higher capacities than driven piles and have been used successfully in many areas with carbonate soils. The characteristics of carbonate sands are highly variable and local experience should dictate the design parameters selected. For example, experience suggests that capacity is improved in carbonate soils of high densities and higher quartz contents. Cementation may increase end bearing capacity, but result in a loss of lateral pressure and a corresponding decrease in frictional capacity. The Commentary provides more discussion of important aspects to be considered.

For piles driven in undersized drilled or jetted holes in cohesionless soils, the values of f and q should be determined

Table 6.4.3-1—Design Parameters for Cohesionless Siliceous Soil[1]

Relative Density[2]	Soil Description	Shaft Friction Factor[3] β (–)	Limiting Shaft Friction Values kips/ft[2] (kPa)	End Bearing Factor N_q (–)	Limiting Unit End Bearing Valves kips/ft[2] (MPa)
Very Loose Loose Loose Medium Dense Dense	Sand Sand Sand-Silt[4] Silt Silt	Not Applicable[5]	Not Applicable[5]	Not Applicable[5]	Not Applicable[5]
Medium Dense	Sand-Silt[4]	0.29	1.4 (67)	12	60 (3)
Medium Dense Dense	Sand Sand-Silt[4]	0.37	1.7 (81)	20	100 (5)
Dense Very Dense	Sand Sand-Silt[4]	0.46	2.0 (96)	40	200 (10)
Very Dense	Sand	0.56	2.4 (115)	50	250 (12)

[1] The parameters listed in this table are intended as guidelines only. Where detailed information such as CPT records, strength tests on high quality samples, model tests, or pile driving performance is available, other values may be justified.

[2] The following definitions for relative density description are applicable:

Description	Relative Density [%]
Very Loose	0 – 15
Loose	15 – 35
Medium Dense	35 – 65
Dense	65 – 85
Very Dense	85 – 100

[3] The shaft friction factor β (equivalent to the "K tan δ" term used in previous editions of API RP 2A-WSD) is introduced in this edition to avoid confusion with the δ parameter used in the Commentary.

[4] Sand-Silt includes those soils with significant fractions of both sand and silt. Strength values generally increase with increasing sand fractions and decrease with increasing silt fractions.

[5] Design parameters given in previous editions of API RP 2A-WSD for these soil/relative density combinations may be unconservative. Hence it is recommended to use CPT-based methods from the Commentary for these soils.

by some reliable method that accounts for the amount of soil disturbance due to installation, but they should not exceed values for driven piles. Except in unusual soil types, such as described above, the f and q values given in Table 6.4.3-1 may be used for drilled and grouted piles, with consideration given to the strength of the soil-grout interface.

In layered soils, unit shaft friction values, in cohesionless layers should be computed according to Table 6.4.3-1. End bearing values for piles tipped in cohesionless layers with adjacent layers of lower strength may also be taken from Table 6.4.3-1. This is provided that the pile achieves penetration of two to three diameters or more into the cohesionless layer, and the tip is at least three diameters above the bottom of the layer to preclude punch through. Where these pile tip penetrations are not achieved, some modification in the tabulated values may be necessary. Where adjacent layers are of comparable strength to the layer of interest, the proximity of the pile tip to the layer interface is not a concern.

Note: Sections 6.4.4 through 6.7.3 are unchanged from 21st Edition.

Replace Section 6.8.1 with the following:

6.8 SOIL REACTION FOR LATERALLY LOADED PILES

6.8.1 General

The pile foundation should be designed to sustain lateral loads, whether static or cyclic. Additionally, the designer should consider overload cases in which the design lateral loads on the platform foundation are increased by an appropriate safety factor. The designer should satisfy himself that the overall structural foundation system will not fail under the overloads. The lateral resistance of the soil near the surface is significant to pile design and the effects on this resistance of scour and soil disturbance during pile installation should be considered. Generally, under lateral loading, clay soils behave as a plastic material which makes it necessary to relate pile-soil deformation to soil resistance. To facilitate this procedure, lateral soil resistance deflection (p-y) curves should be constructed using stress-strain data from laboratory soil samples. The ordinate for these curves is soil resistance, p, and the abscissa is soil deflection, y. By iterative procedures, a compatible set of load-deflection values for the pile-soil system can be developed.

For a more detailed study of the construction of p-y curves refer to the following publications:

- **Soft Clay:** OTC 1204, *Correlations for Design of Laterally Loaded Piles in Soft Clay*, by H. Matlock, April 1970.
- **Stiff Clay:** OTC 2312, *Field Testing and Analysis of Laterally Loaded Piles in Stiff Clay*, by L. C. Reese and W. R. Cox, April 1975.
- **Sand:** "An Evaluation of p-y Relationships in Sands," by M. W. O'Neill and J. M. Murchinson. *A report to the American Petroleum Institute*, May 1983.

In the absence of more definitive criteria, procedures recommended in 6.8.2 and 6.8.3 may be used for constructing ultimate lateral bearing capacity curves and p-y curves. It is noted that these p-y curves are recommended to estimate pile bending moment, displacement and rotation profiles for various (static or cyclic) loads. Different criteria may be applicable for fatigue analysis of a pile which has previously been subjected to loads larger than those used in the fatigue analysis which resulted in "gapping" around the top of the pile. A discussion on this subject and associated guidelines are presented in OTC 1204, referred to above.

The methods below are intended as guidelines only. Where detailed information such as advanced testing on high quality samples, model tests, centrifuge tests, or full scale pile testing is available, other methods may be justified.

Note: Section 6.8.2 is unchanged from 21st Edition.

Replace Section 6.8.3 with the following:

6.8.3 Load-deflection (*p-y*) Curves for Soft Clay

Lateral soil resistance-deflection relationships for piles in soft clay are generally non-linear. The p-y curves for the short-term static load case may be generated from the following table:

p/p_u	y/y_c
0.00	0.0
0.23	0.1
0.33	0.3
0.50	1.0
0.72	3.0
1.00	8.0
1.00	∞

where

p = actual lateral resistance, psi (kPa),

y = actual lateral deflection, in. (m),

y_c = $2.5\ \varepsilon_c\ D$, in. (m),

ε_c = strain which occurs at one-half the maximum stress on laboratory unconsolidated undrained compression tests of undisturbed soil samples.

For the case where equilibrium has been reached under cyclic loading, the p-y curves may be generated from the following table:

$X > X_R$		$X < X_R$	
p/p_u	y/y_c	p/p_u	y/y_c
0.00	0.0	0.00	0.0
0.23	0.1	0.23	0.1
0.33	0.3	0.33	0.3
0.50	1.0	0.50	1.0
0.72	3.0	0.72	3.0
0.72	∞	$0.72\ X/X_R$	15.0
		$0.72\ X/X_R$	∞

Note: Sections 6.8.4 through 6.17 are unchanged from 21st Edition.

COMMENTARY ON CARBONATE SOILS, SECTION 6.4.3

Note: Commentary on Carbonate Soils appeared in the 21st Edition as Sections C6.4.3a through C6.4.3e (with references following). These sections have been renumbered as Sections C6.1a through C6.1e (with references following). The content of this Commentary is unchanged.

C6.1a General

Carbonate soils cover over 35 percent of the ocean floor. For the most part, these soils are biogenic, that is they are composed of large accumulations of the skeletal remains of plant and animal life such as coralline algae, coccoliths, foraminifera, echinoderms, etc. To a lesser extent they also exist as non-skeletal material in the form of oolites, pellets, grapestone, etc. These carbonate deposits are abundant in the warm, shallow water of the tropics, particularly between 30 degrees North and South latitudes. Deep sea calcareous oozes have been reported at locations considerably outside the mid latitudes. Since temperature and water conditions (water depth, salinity, etc.) have varied throughout geologic history, ancient deposits of carbonate material may be found buried under more recent terrestrial material outside the present zone of probable active deposition. In the Gulf of Mexico, major carbonate deposits are known to exist along the Florida coastline and in the Bay of Campeche.

C6.1.b Characteristic Features of Carbonate Soils

Carbonate soils differ in many ways from the silica rich soils of the Gulf of Mexico. An important distinction is that the major constituent of carbonate soils is calcium carbonate which has a low hardness value compared to quartz, the predominant constituent of the silica rich sediments. Susceptibility of carbonate soils to disintegration (crushing) into smaller fractions at relatively low stress levels is partly attributed to this condition. Typically, carbonate soils have large interparticle and intraparticle porosity resulting high void ratio and low density and hence are more compressible than soils from a terrigenous silica deposit. Furthermore, carbonate soils are prone to post-depositional alterations by biological and physio-chemical processes under normal pressure and temperature conditions which results in the formation of irregular and discontinuous layers and lenses of cemented material. These alterations, in turn, profoundly affect mechanical behavior.

The fabric of carbonate soils is an important characteristic feature. Generally, particles of skeletal material will be angular to subrounded in shape with rough surfaces and will have intraparticle voids. Particles of non-skeletal material, on the other hand, are solid with smooth surfaces and without intraparticle voids. It is generally understood that uncemented carbonate soils consisting of rounded nonskeletal grains that are resistant to crushing are stronger foundation materials than carbonate soils which show partial cementation and a low to moderate degree of crushing.

There is information that indicates the importance of carbonate content as it relates to the behavior of carbonate sediments. A soil matrix which is predominately carbonate is more likely to undergo degradation due to crushing and compressibility of the material than soil which has low carbonate fraction in the matrix. Other important characteristic features that influence the behavior of the material are grain angularity, initial void ratio, compressibility and grain crushing. These are interrelated parameters in the sense that carbonate soils with highly angular particles often have a high in situ void ratio due to particle orientation. These soils are more susceptible to grain crushing due to angularity of the particles and thus will be more apt to be compressed during loading.

The above is a general overview of the mechanical behavior of carbonate soils. For a more detailed understanding of material characteristics, readers are directed to the references cited below.

C6.1.c Soil Properties

Globally, it is increasingly evident that there is no unique combination of laboratory and in situ testing program that is likely to provide all the appropriate parameters for design of foundations in carbonate soils. Some laboratory and in situ tests have been found useful. As a minimum, a laboratory testing program for carbonate soils should determine the following:

1. Material composition; particularly carbonate content.
2. Material origin to differentiate between skeletal and nonskeletal sediments.
3. Grain characteristics; such as particle angularity, porosity and initial void ratio.
4. Compressibility of the material.
5. Soil strength parameters; particularly friction angle.
6. Formation cementation; at least in a qualitative sense.

For site characterization, maximum use of past local experience is important particularly in the selection of an appropriate in situ program. In new unexplored territories where the presence of carbonate soils is suspected, selection of an in situ test program should draw upon any experience with carbonate soils where geographical and environmental conditions are similar.

C6.1d Foundation Performance and Current Trends

C6.1d.1 Driven Piles

Several case histories have been reported that describe some of the unusual characteristics of foundations on carbonate soils and their often poor performance. It has been shown from numerous pile load tests that piles driven into weakly cemented and compressible carbonate soils mobilize only a fraction of the capacity (as low as 15 percent) predicted by conventional design/prediction methods for siliceous material of the type generally encountered in the Gulf of Mexico. On the other hand, dense, strongly cemented carbonate deposits can be very competent foundation material. Unfortunately, the difficulty in obtaining high quality samples and the lack of generalized design methods sometimes make it difficult to predict where problems may occur.

C6.1d.2 Other Deep Foundation Alternatives

The current trend for deep foundations in carbonate soils is a move away from driven piles. However, because of lower installation costs, driven piles still receive consideration for support of lightly loaded structures or where extensive local pile load test data and experience exists to substantiate the design premise. Furthermore, driven piles may be appropriate in competent carbonate soils. At present, the preferred alternative to the driven pile is the drilled and grouted pile. These piles mobilize significantly higher unit skin friction. The result is a substantial reduction in the required pile penetration compared with driven piles. Because of the high construction cost of drilled and grouted piles, an alternative driven and grouted pile system has received some attention in the recent past. This system has the potential to reduce installation costs while achieving comparable capacity.

C6.1d.3 Shallow Foundations

Some evidence indicates that the bearing capacity of shallow foundations in weakly cemented and compressible granular carbonate deposits can be significantly lower than the capacity in the siliceous material generally encountered in the Gulf of Mexico. On the other hand, higher bearing capacities have been reported where the soil is dense, strongly cemented, competent material.

C6.1e Assessment

To date, general design procedures for foundations in carbonate soils are not available. Acceptable design methods have evolved but remain highly site specific and dependent on local experience. Stemming from some recent publications describing poor foundation performance in carbonate soils and the financial consequences of the remedial measures, there is a growing tendency to take a conservative approach to design at the mere mention of carbonate soils even if the carbonate content in the sediment fraction is relatively low. This is not always warranted. As with other designs, the judgment of knowledgeable engineers remains a critical link in economic development of offshore resources in carbonate soil environments.

References

To develop an understanding and appreciation for the State-of-the-Practice in carbonate soil, a starting point would be to review the proceedings from two major conferences on carbonate soils listed below:

1. Symposium on Performance and Behavior of Calcareous Soils Sponsored by ASTM Committee D-18 on Soil and Rock, Ft. Lauderdale, Florida, January 1981.

2. International Conference on Calcareous Sediments, Perth, Australia, March, 1988.

Other selected references are:

1. Abbs, A. F., and Needham, A. D., *Grouted Piles in Weak Carbonate Rocks*, Proceedings, 17th Offshore Technology Conference, Houston, Texas, Paper No. 4852, May 1985.

2. Angemeer, J., Carlson, E., and Klick, J. H., *Techniques and Results of Offshore Pile Load Testing in Calcareous Soils*, Proceedings, 5th Offshore Technology Conference, Houston, Texas, Paper No. 1894, May 1973.

3. Barthelemy, H. C., Martin, R., Le Tirant, P. M., and Nauroy, J. F., *Grouted Driven Piles: An Economic and Safe Alternate for Pile Foundation*, Proceedings, 19th Offshore Technology Conference, Houston, Texas, Paper No. 5409, 1987.

4. Clark, A. R., and Walker, B. F., *A Proposed Scheme for the Classification and Nomenclature for Use in the Engineering Description of Middle Eastern Sedimentary Rocks*, Geotechnique, Vol. 27, No. 1, 1977.

5. Datta, M., Gulhati, S. K., and Rao, G. V., *Crushing of Calcareous Sands During Shear*, Proceedings 11th Offshore Technology Conference, Houston, Paper No. 3525, 1979.

6. Dutt, R. N., and Cheng, A. P., *Frictional Response of Piles in Calcareous Deposits*, Proceedings, 16th Offshore Technology Conference, Houston, Texas, Paper No. 4838, May 1984.

7. Dutt, R. N., Moore, J. E., Mudd, R. W., and Rees, T. E., *Behavior of Piles in Granular Carbonate Sediments from Offshore Philippines*, Proceedings, 17th Offshore Technology Conference, Houston, Texas, Paper No. 4849, May 1985.

8. Fragio, A. G., Santiago, J. L., and Sutton, V. J. R., *Load Tests on Grouted Piles in Rock*, Proceedings, 17th Offshore Technology Conference, Houston, Texas, Paper No. 4851, May 1985.

9. Gilchrist, J. M., *Load Tests on Tubular Piles in Coralline Strata*, Journal of Geotechnical Engineering, ASCE, Vol, III, No. 5, 1985.

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COMMENTARY ON AXIAL PILE CAPACITY IN CLAY, SECTION 6.4

Note: Commentary on Axial Pile Capacity in Clay has been revised and renumbered as Sections C6.4.2a through C6.4.2e (with references following).

C6.4.2a Load Test Database for Piles in Clay

A number of studies[1,2,3,4,5,6] have been carried out, aimed at collecting and comparing axial capacities from relevant pile load tests to those predicted by traditional offshore pile design procedures. Studies such as these can be very useful in tempering one's judgement in the design process. It is clear, for example, that there is considerable scatter in the various plots of measured versus predicted capacities. The designer should be aware of the many limitations of such comparisons when making use of these results. Limitations of particular importance include the following:

1. There is considerable uncertainty in the determination of both predicted capacities and measured capacities. For example, determination of the predicted capacities is very sensitive to the selection of the undrained shear strength profile, which itself is subject to considerable uncertainty. The measured capacities are also subject to interpretation as well as possible measurement errors.
2. The conditions under which the pile load tests are conducted generally vary significantly from the design loads and field conditions. One clear limitation is the limited number of tests on deeply embedded, large diameter, high capacity piles. Generally, pile load tests have capacities that are 10 % or less of the prototype capacities. Another factor is that the rate of loading and the cyclic load history are usually not well represented in the load tests[7,8]. For practical reasons, the pile load tests are often conducted before full set-up occurs[9]. Furthermore, the pile tip conditions (closed versus open-ended) may differ from offshore piles.
3. In most of the studies an attempt has been made to eliminate those tests that are thought to be significantly affected by extraneous conditions in the load test, such as protrusions on the exterior of the pile shaft (weld beads, cover plates, etc.), installation effects (jetting, drilled out plugs, etc.), and artesian conditions, but it is not possible to be absolutely certain in all cases.

The database includes a number of tests that were specially designed for offshore applications as well as a number of published tests that are fortuitously relevant to offshore conditions (appropriate pile type, installation method, soil conditions, etc.). The former are generally higher quality and larger scale, and hence are particularly important in calibrating the design method. The tests most relevant to offshore applications have all been conducted in the United States or in Europe. As regional geology and particularly operating experience are considered very important in foundation design, care should be exercised in applying these results to other regions of the world. In addition, the designer should note that certain important tests in silty clays of low plasticity, such as at the Pentre site[9] indicate overprediction of frictional resistance by the Equations (6.4.2-1) and (6.4.2-2). The reason for this overprediction is not well understood and has been an area of active research. The designer is thus cautioned that pile design for soils of this type should be given special consideration.

Additional considerations that apply to drilled and grouted piles are discussed in References 10 and 11.

C6.4.2b Alternative Methods of Determining Pile Capacity

Alternative methods of determining pile capacity in clays, which are based on sound engineering principles and are consistent with industry experience, exist and may be used in practice. One such method is described below:

For piles driven through clay, f may be equal to or less than, but should not exceed the undrained shear strength of the clay c_u, as determined by unconsolidated-undrained (UU) triaxial tests and miniature vane shear tests.

Unless test data indicate otherwise, f should not exceed cu or the following limits:

1. For highly plastic clays, f may be equal to cu for underconsolidated and normally consolidated clays. For overconsolidated clays, f should not exceed 1 kips/ft^2 (48 kPa) for shallow penetrations or the equivalent value of cu for a normally consolidated clay for deeper penetrations, whichever is greater.
2. For other types of clay:

$$f = c_u \qquad \text{for } c_u < 0.5 \text{ kips/ft}^2 \text{ (24 kPa)}$$

$$f = c_u/2 \qquad \text{for } c_u > 1.5 \text{ kips/ft}^2 \text{ (72 kPa)}$$

f varies linearly for values of cu between the above limits.

For other methods, see References 1, 2, 3 and 5.

It has been shown[6] that, on the average, the above cited methods predict the available but limited pile load test database results with comparable accuracy. However, capacities for specific situations computed by different methods can differ by a significant amount. In such cases, pile capacity determination should be based on engineering judgement, which takes into account site-specific soils information, available pile load test data, and industry experience in similar soils.

C6.4.2c Establishing Design Strength and Effective Overburden Stress Profiles

The axial pile capacity in clay determined by these procedures is directly influenced by the undrained shear strength and effective overburden stress profiles selected for use in analyses. The wide variety of sampling techniques and the potentially large scatter in the strength data from the various types of laboratory tests complicate appropriate selection.

UU triaxial compression tests on high quality samples, preferably taken by pushing a thin-walled sampler with a diameter of 3 in. (75 mm) or more into the soil, are recommended for establishing strength profile variations because of their consistency and repeatability. In selecting the specific shear strength values for design, however, consideration should be given to the sampling and testing techniques used to correlate the procedure to any available relevant pile load test data. The experience with pile performance is another consideration that can play an important role in assessing the appropriate shear strength interpretation.

Miniature vane tests on the pushed samples should correlate well with the UU test results and will be particularly beneficial in weak clays. In-situ testing with a vane or cone penetrometer will help in assessing sampling disturbance effects in gassy or highly structured soils. Approaches such as the SHANSEP technique (Stress History and Normalized Soil Engineering Properties)[12], can help provide a more consistent interpretation of standard laboratory tests and will provide history information used to determine the effective overburden stress in normally or underconsolidated clays.

C6.4.2d Pile Length Effect

Long piles driven in clay soils can experience capacity degradation due to the following factors:

1. Continued shearing of a particular soil horizon during pile installation.
2. Lateral movement of soil away from the pile due to "pile whip" during driving.
3. Progressive failure in the soil due to strength reduction with continued displacement (softening).

The occurrence of degradation due to these effects depends on many factors related to both installation conditions and soil behavior. Methods of estimating the possible magnitude of reduction in capacity of long piles can be found in References 2, 3, 5, 13, 14 and 15.

C6.4.2e Changes in Axial Capacity in Clay with Time

Existing axial pile capacity calculation procedures for piles in clay are based on experience tempered by the results of axial pile load tests. In these tests, few of the piles were instrumented and in most cases little or no consideration was given to the effects of time after driving on the development of shear transfer in the soil. Axial capacity of a driven pipe pile in clay computed according to the guidelines set forth in Sections 6.4.1 and 6.4.2 is intended to represent the long-term static capacity of piles in undrained conditions when subjected to axial loads until failure, after dissipation of excess pore water pressure caused by the installation process. Immediately after pile driving, pile capacity in a cohesive deposit can be significantly lower than the ultimate static capacity. Field measurements[9,16,17] have shown that the time required for driven piles to reach ultimate capacity in a cohesive deposit can be relatively long, as much as two to three years. However, it should be noted that the rate of strength gain is highest immediately after driving, and this rate decreases during the dissipation process. Thus a significant strength increase can occur in a relatively short time.

During pile driving in normally to lightly overconsolidated clays, the soil surrounding a pile is significantly disturbed, the stress state is altered, and large excess pore pressures can be generated. After installation, these excess pore pressures begin to dissipate, i.e. the surrounding soil mass begins to consolidate and the pile capacity increases with time. This process is usually referred to as "set-up." The rate of excess pore pressure dissipation is a function of the coefficient of radial (horizontal) consolidation, pile radius, plug characteristics (plugged versus unplugged pile), and soil layering.

In the case of driven pipe piles supporting a structure where the design loads can be applied to the piles shortly after installation, the time-consolidation characteristics should be considered in pile design. In such cases, the capacity of piles immediately after driving and the expected increase in capacity with time are important design variables that can impact the safety of the foundation system during early stages of the consolidation process.

A number of investigators[18,19] have proposed analytical models of pore pressure generation and the subsequent dissipation process for piles in normally consolidated to lightly overconsolidated clays. Since excess pore pressures are generated by pile driving operations, any dissipation of the excess pore pressures after installation should correspond to an equivalent increase in the shear strength of the surrounding soil mass and hence an increase in the capacity of the pile. After dissipation of excess pore pressures, the capacity of a pile approaches the long-term capacity, although some strength gain may continue due to secondary processes. In some overconsolidated clays, pile capacity can decrease as pore pressures dissipate, provided the rate of change of radial total stress decreases faster than the rate of change of pore pressure. The analytical models account for the degree of plugging by assuming various degrees of plug formation, ranging from closed- to open-ended pile penetration modes. Input necessary for the analysis includes the soil characteristics (compressibility, stress history, strength, etc.) and the initial site conditions.

In Reference 16, the behavior of piles subjected to significant axial loads in highly plastic, normally consolidated clays was studied using a large number of model pile tests and some full scale pile load tests. From the study of pore pressure dissipation and load test data at different times after pile driving, empirical correlations were obtained between the degree of consolidation, degree of plugging, and pile shaft shear transfer capacity. The analysis is dependent on the shear strength of the surrounding soil mass. The method is presently limited to use in highly plastic, normally consolidated clays of the type encountered in the Gulf of Mexico, since validation data have been published only for those soils.

In Reference 17, pile capacity in highly overconsolidated glacial till was shown to undergo significant short-term reduction associated with pore pressure redistribution and reduction in radial effective stresses during the early stages of the equalization process. The capacity at the end of installation was never fully recovered. Test results for closed-ended steel piles in heavily overconsolidated London clay indicate that there is no significant change in capacity with time[20]. This is contrary to tests on 10.75 in. (0.273 m) diameter closed-ended steel piles in overconsolidated Beaumont clay, where considerable and rapid set-up (in 4 days) was found[21].

Caution should be exercised in using the above mentioned procedures to evaluate set-up, particularly for soils with different plasticity characteristics and under different states of consolidation (especially overconsolidated clays) and piles with *D/t* ratios greater than 40.

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COMMENTARY ON AXIAL PILE CAPACITY IN SAND, SECTION 6.4.3

Note: Commentary on Axial Pile Capacity in Sand has been added as Sections C6.4.3a through C6.4.3f (with references following).

C6.4.3a General

Estimating axial pile capacity in cohesionless soils requires considerable engineering judgment in selecting an appropriate method and associated parameter values. Some of the items that should be considered by geotechnical engineers are detailed in the following paragraphs.

The term "sand" is used hereafter for all cohesionless siliceous soils. Exceptions (e.g., carbonate sands and gravels) are dealt with in Section C6.4.3e below.

The piles are assumed to be open-ended steel piles of uniform outer diameter. Installation is by impact driving into significant depths of clean siliceous sand. In general, such piles drive "unplugged" (i.e., they core). However, when statically loaded in compression, sufficient inner friction is generally mobilized to cause the pile to act as fully "plugged", (i.e., the soil plug does not undergo overall "slip" relative to the pile wall during compression pile loading).

Notation is given in Section C6.4.3b below. In this Commentary, the symbol σ'_{vo} is used instead of p_o' (as in the Main Text, Section 6.4.3) to denote soil in-situ vertical effective stress, and p'_m is used to denote soil in-situ mean effective stress.

The appropriate safety factors to be used with the methods below are not provided herein. The designer should carefully evaluate, for each design case, whether the safety factors provided in the main text are appropriate or not.

C6.4.3b Notation

A_p	=	pile gross end area = $\pi D_o^2/4$
A_r	=	pile displacement ratio
	=	$1 - (D_i/D_o)^2$
D_{CPT}	=	CPT tool diameter
	≅	36 mm for a standard 10 cm^2 base area cone
D	=	pile outer diameter = D_o
D_i	=	pile inner diameter = $D_o - 2WT$
D_o	=	pile outer diameter
D_r	=	sand relative density [0 – 1]
e	=	base natural logarithms ≈ 2.718
f_z	=	pile-soil unit skin friction capacity = $f_{c,z}$ (compression) or = $f_{t,z}$ (tension)
$f_{c,z}$	=	pile-soil unit skin friction capacity in compression, a function of depth (z) and pile geometry (L,D,WT)
$f_{t,z}$	=	pile-soil unit skin friction capacity in tension, a function of depth (z) and pile geometry (L,D,WT)
h	=	distance above pile tip = $L - z$
K_o	=	ratio effective horizontal:vertical in-situ soil stresses $\sigma'_{ho}/\sigma'_{vo}$
L	=	pile embedded length (below original seabed)
L_s	=	sand plug length
ln	=	natural logarithm (base e)
p_a	=	atmospheric pressure = 100 kPa
P_o	=	pile outer perimeter = πD_o
$q_{c,av,1.5D}$	=	average $q_{c,z}$ value between $1.5D_o$ above pile tip to $1.5D_o$ below pile tip level
	=	$\int_{L-1.5\cdot D_o}^{L+1.5\cdot D_o} q_{c,z}\, dz / 3D_o$
$q_{c,av}$	=	average $q_{c,z}$ value
$q_{c,z}$	=	CPT cone tip resistance q_c at depth z
Q_d	=	pile ultimate bearing capacity
	=	$Q_f + Q_p$
q_p	=	unit end bearing at penetration L of pile gross tip area (fully plugged)
Q_f	=	pile ultimate skin friction capacity in compression
	=	$P_o \int f_{c,z} dz$
$Q_{f,i,clay}$	=	cumulative skin friction on clay layers within soil plug
Q_p	=	pile ultimate end bearing resistance
	=	$q_p A_p$
Q_t	=	pile ultimate tensile capacity
	=	$P_o \int f_{t,z} dz$
WT	=	pile wall thickness at pile tip (including driving shoe)
z	=	depth below original seabed
δ_{cv}	=	pile-soil constant volume interface friction angle
σ'_{ho}	=	soil effective horizontal in-situ stress at depth z
σ'_{vo}	=	soil effective vertical in-situ stress at depth z

C6.4.3c CPT-based Methods for Pile Capacity

C6.4.3c.1 Introduction

The Main Text (Section 6.4.3) presented a simple method for assessing pile capacity in cohesionless soils, which is a modification of methods recommended in previous editions of API RP 2A-WSD. Changes were made to remove potential unconservatism in previous editions. This Commentary presents recent and more reliable CPT-based methods for predicting pile capacity. These methods are all based on direct correlations of pile unit friction and end bearing data with cone tip resistance (q_c) values from cone penetration tests (CPT). In comparison to the Main Text method, these CPT-based methods cover a wider range of cohesionless soils, are considered fundamentally better and have shown statistically closer predictions of pile load test results.

These new CPT-based methods for assessing pile capacity in sand are preferred to the method in the Main Text. However, more experience is required with all these new methods before any single one can be recommended for routine design instead of the Main Text method. These new CPT-based methods should be used only by qualified engineers who are experienced in interpreting CPT data, and understand the limitations and reliability of these CPT-based methods.

The assumption is made that friction and end bearing components are uncoupled. Hence, for all methods, the ultimate bearing capacity in compression (Q_d) and tensile capacity (Q_t) of plugged open-ended piles is determined by the equations:

$$Q_d = Q_f + Q_p = P_o \int f_{c,z} dz + A_p q_p \qquad \text{(C6.4.3-1)}$$

$$Q_t = P_o \int f_{t,z} dz \qquad \text{(C6.4.3-2)}$$

Note that since the friction component, Q_f, involves numerical integration, results are sensitive to the depth increment used, particularly for CPT-based methods. As guidance, depth increments for CPT-based methods should be in the order of 1/100 of the pile length (or smaller). In any case, the depth increment should not exceed 0.5 ft (0.2 m).

The four recommended CPT-based methods discussed herein are:

1. Simplified ICP-05 (this publication)
2. Offshore UWA-05 (Lehane et al., 2005a,b)
3. Fugro-05 (Lehane et al., 2005a, Kolk et al., 2005)
4. NGI-05 (Lehane et al., 2005a, Clausen et al., 2005)

The first method is a simplified version of the design method recommended by Jardine et al., (2005), whereas the second is a simplified version of the UWA-05 method applicable to offshore pipe piles. Methods 2, 3 and 4 are summarised by Lehane et al., (2005a). Friction and end-bearing components should not be taken from different methods. Following a general description applicable to the first three methods, details of individual methods are presented in subsections below.

The unit skin friction (f_z) formulae for open ended steel pipe piles for the first three recommended CPT-based methods (Simplified ICP-05, Offshore UWA-05 and Fugro-05) can all be considered as special cases of the general formula:

$$f_z = u \cdot q_{c,z} \left(\frac{\sigma'_{vo}}{p_a}\right)^a A_r^{\,b} \left[\max\left(\frac{L-z}{D}, v\right)\right]^{-c} [\tan\delta_{cv}]^d \times \left[\min\left(\frac{L-z}{D}\frac{1}{v}, 1\right)\right]^e \qquad \text{(C6.4.3-3)}$$

Recommended values for parameters *a, b, c, d, e, u* and *v* for compression and tension are given in Table C6.4.3-1.

Additional recommendations for computing unit friction and end bearing of all four CPT-based methods are presented in the following subsections.

Table C6.4.3-1—Unit Skin Friction Parameter Values for Driven Open-ended Steel Pipes (Simplified ICP-05, Offshore UWA-05 and Fugro-05 Methods)

Method	Parameter						
	a	b	c	d	e	u	v
Simplified ICP-05							
compression	0.1	0.2	0.4	1	0	0.023	$4\sqrt{A_r}$
tension	0.1	0.2	0.4	1	0	0.016	$4\sqrt{A_r}$
Offshore UWA-05							
compression	0	0.3	0.5	1	0	0.030	2
tension	0	0.3	0.5	1	0	0.022	2
Fugro-05							
compression	0.05	0.45	0.90	0	1	0.043	$2\sqrt{A_r}$
tension	0.15	0.42	0.85	0	0	0.025	$2\sqrt{A_r}$

C6.4.3c.2 Simplified ICP-05

Friction

Jardine et al., (2005) presented a comprehensive overview of research work performed at Imperial College on axial pile design criteria of open and closed ended piles in clay and sand. The design equations for unit friction in sand in this publication include a soil dilatancy term, implying that unit friction is favourably influenced by soil dilatancy. This influence diminishes with increasing pile diameter. The Simplified ICP-05 method for unit skin friction of open ended pipe piles, given by Equation C6.4.3-3 and parameter values in Table C6.4.3-1, is a conservative approximation of the full ICP-05 method since dilatancy is ignored and some parameter values were conservatively rounded up/down.

Use of the original "full" design equations in Jardine et al., (2005) may be considered [particularly for small diameter piles, D < 30 in. (0.76 m)], provided that larger factors of safety be considered in the WSD design. Reference should be made to Jardine et al., (2005) for a discussion on reliability based design using the "full" ICP-05 method.

End bearing

The ultimate unit end bearing for open ended pipe piles, q_p, follows the recommendations of Jardine et al., (2005). These specify an ultimate unit end bearing for plugged piles given by:.

$$q_p = q_{c,\,av,\,1.5D}(0.5 - 0.25\log_{10}(D/D_{CPT})) \geq 0.15 q_{c,\,av,\,1.5D} \quad \text{(C6.4.3-4)}$$

Jardine et al., (2005) specify that plugged pile end bearing capacity applies, that is the unit end bearing q_p acts across the entire tip cross section, only if both the following conditions are met:

$$D_i < 2(D_r - 0.3) \quad \text{(C6.4.3-5)}$$

Note: D_i units are [m] and D_r units are [–], not [%]

and

$$D_i/D_{CPT} < 0.083 q_{c,\,z}/p_a \quad \text{(C6.4.3-6)}$$

Should either of the above conditions not be met, then the pile will behave unplugged and the following equation should be used for computing the end bearing capacity:

$$Q_p = \pi WT(D - WT)q_{c,\,z} \quad \text{(C6.4.3-7)}$$

The full pile end bearing computed using equation (C6.4.3-4) for a plugged pile should not be less than the end bearing capacity of an unplugged pile computed according to equation (C6.4.3-7).

C6.4.3c.3 Offshore UWA-05

Friction

Lehane et al., (2005a) summarize the results of recent research work at the University of Western Australia on development of axial pile design criteria of open and closed ended piles driven into silica sands. The full design method (described in Lehane et al., 2005a,b) for unit friction on pipe piles includes a term allowing for favourable effects of soil dilatancy (similar to ICP-05) and an empirical term allowing for partial soil plugging during pile driving. Lehane et al., recommend for offshore pile design to ignore these two favourable effects (dilatancy and partial plugging), resulting in the recommended Equation C6.4.3-3 and associated Table C6.4.3-1 parameter values. Use of the original ("full") design equations in Lehane et al., (2005a) may be considered [particularly for small diameter piles, D < 30 in. (0.76 m)], provided that larger factors of safety be considered in the WSD design. Reference should be made to Lehane et al., (2005a) for a discussion on reliability based design using the UWA-05 method.

End Bearing

Lehane et al., (2005a,b) present design criteria for ultimate unit end bearing of plugged open ended pipe piles. Their "full" design method for pipe piles includes an empirical term allowing for the favourable effect of partial plugging during pile driving. For offshore pile design, Lehane et al., (2005a,b) recommend to ignore this effect, resulting in the recommended design equation for plugged piles in the Offshore UWA-05 method:

$$q_p = q_{c,\,av,\,1.5D}(0.15 + 0.45A_r) \quad \text{(C6.4.3-8)}$$

Since the UWA-05 method considers non-plugging under static loading to be exceptional for typical offshore piles, the method does not provide criteria for unplugged piles. The unit end bearing q_p calculated in C6.4.3.-8 is therefore acting across the entire tip cross section. The use of $q_{c,av,1.5D}$ in Equation C6.4.3-8 is not recommended in sand profiles where the CPT q_c values shows significant variations in the vicinity of the pile tip or when penetration into a founding stratum is less than five pile diameters. For these situations, Lehane et al., (2005a) provide guidance on the selection of an appropriate average q_c value for use in place of $q_{c,av,1.5D}$.

C6.4.3c.4 Fugro-05

Friction

The Fugro-05 method is a modification of the ICP-05 method and was developed as part of a research project for API The unit friction equations were unfortunately misprinted in (Fugro 2004; Kolk et al., 2005) and these references are not to be used in design. However, the correct equations are presented both by Lehane et al., (2005a) and by

Equation C6.4.3-3 and the parameter values in Table C6.4.3-1. Like the “full” ICP-05 and the “full” UWA-05 methods, it is recommended to consider larger factors of safety when using the Fugro-05 method. Reference is made to CUR (2001), for a discussion on reliability based design using this method.

End Bearing

The basis for the ultimate unit end bearing on pipe piles according to Fugro-05 is presented in the research report to API (Fugro 2004) and summarised by Kolk et al., (2005). The recommended design criterion for plugged piles is given by:

$$q_p = 8.5 p_a \left(\frac{q_{c,av,1.5D}}{p_a}\right)^{0.5} A_r^{0.25} \qquad \text{(C6.4.3-9)}$$

Both UWA-05 and Fugro-05 do not specify unplugged end bearing capacity because typical offshore piles behave in a plugged mode during static loading (CUR, 2001). It can be shown that plugged behaviour applies if either:

a. The cumulative thickness of sand layers within a soil plug is in excess of 8D, or

b. The total end bearing (Q_p) is limited as follows:

$$Q_p \le Q_{f,i,clay} e^{L_s/D} \qquad \text{(C6.4.3-10)}$$

Where the cumulative frictional capacity of the clay layers within the soil plug ($Q_{f,i,clay}$) can be estimated using similar criteria as for computing estimated pile friction in clay (Section 6.4.2).

The above criteria apply for fully drained behaviour of sand within the pile plug. Criteria for undrained/partially drained sand plug behaviour are presented by Randolph et al., (1991).

For the exceptional case of unplugged end bearing behaviour in fully drained conditions, reference is made to CUR (2001) and Lehane & Randolph (2002) for estimating end bearing capacity.

C6.4.3c.5 NGI-05

Friction

Ultimate unit skin friction values for tension ($f_{t,z}$) and compression ($f_{c,z}$) for driven open-ended steel pipe piles in the NGI-05 method are given by Clausen et al., (2005):

$$f_{t,z} = (z/L) p_a F_{sig} F_{Dr} > 0.1\sigma'_{vo} \qquad \text{(C6.4.3-11)}$$

$$f_{c,z} = 1.3(z/L) p_a F_{sig} F_{Dr} > 0.1\sigma'_{vo} \qquad \text{(C6.4.3-12)}$$

where

$$F_{sig} = (\sigma'_{vo}/p_a)^{0.25} \qquad \text{(C6.4.3-13)}$$

$$F_{Dr} = 2.1(D_r - 0.1)^{1.7} \qquad \text{(C6.4.3-14)}$$

$$D_r = 0.4 \ln[q_{c,z}/(22(\sigma'_{vo} p_a)^{0.5})] > 0.1 \qquad \text{(C6.4.3-15)}$$

Note: $D_r > 1$ should be accepted and used.

Like the “full” ICP-05, “full” UWA-05 and the Fugro-05 methods, it is recommended to consider higher factors of safety when using the NGI-05 method.

End Bearing

The recommended design equation for ultimate unit end bearing of plugged open-ended steel pipe piles in NGI-05 method (Clausen et al., 2005) is:

$$q_p = \frac{0.7 q_{c,av,1.5D}}{1 + 3D_r^2} \qquad \text{(C6.4.3-16)}$$

where

$$D_r = 0.4 \ln[q_{c,av,1.5D}/(22(\sigma'_{vo} p_a)^{0.5})] > 0.1 \qquad \text{(C6.4.3-17)}$$

Note: $D_r > 1$ should be accepted and used.

The resistance of non-plugging piles should be computed using an ultimate unit wall end bearing value ($q_{w,z}$) given by:

$$q_{w,z} = q_{c,z} \qquad \text{(C6.4.3-18)}$$

and an ultimate unit friction ($f_{p,z}$) between the soil plug and inner pile wall given by:

$$f_{p,z} = 3 f_{c,z} \qquad \text{(C6.4.3-19)}$$

The lower of the plugged resistance (Equation C6.4.3-16) and unplugged resistance (Equations C6.4.3.18 and C6.4.3.19) should be used in design.

C6.4.3d Parameter Value Assessment

The geotechnical site investigation should provide information adequate to capture the spatial variability, horizontally and vertically, of layer boundaries and layer parameter values.

For any CPT method, the computed pile capacity in sand is most sensitive to cone penetration resistance q_c, followed by tan δ_{cv} and σ'_{vo}. Since an accurate capacity assessment is a function of the accuracy of both the model and parameters, guidance regarding selecting appropriate parameter values is given below.

• **Parameter q_c**

The CPT should measure q_c with apparatus and procedures that are in general accordance with those published by ASTM (2000). In particular, ISO (2005) prescribes cones with a base area in the range of 500 mm^2 to 2000 mm^2 and a penetration rate 20 ± 5 mm/s.

It is noted that the CPT-based design methods were established for cone resistance values up to 100 MPa. Caution should be used when applying the enclosed methods to sands with higher resistances.

A measured, continuous q_c profile is preferable to an assumed/interpolated discontinuous profile but is generally not achievable offshore at large depths below seabed with downhole CPT apparatus. This is generally due to factors such as limited stroke and/or maximum resistance being achieved. When (near) continuous qc profiles are needed, one can consider overlapping CPT push strokes.

With discontinuous CPT data, a “blocked” q_c profile can be used: the soil profile is divided into layers, in each of which q_c is assumed to vary linearly with depth. “Blocked” profiles should be carefully assessed, particularly when they contain maximum q_c values at the ends of CPT push strokes. When the push strokes contain no maximum q_c data, a moving window may be used to determine the average (and standard deviation) profile, through which a straight line can be fitted. If present, thin layers of weaker material (e.g., silt or clay) need to be modelled conservatively.

For geotechnical investigations, where several vertical CPT profiles have been made (e.g., one per platform leg), it is suggested that at least two approaches be employed: capacity should be first based on the combined averaged q_c profile and then based on individual q_c profiles. Judgment is required to select the most appropriate q_c profile and associated final axial capacity.

• **Parameter σ'_{vo}**

Usually, pore water pressures in sands are hydrostatic, and, in this case, σ'_{vo} equals (γ_{sub}*z), where γ_{sub} is the submerged soil unit weight. Offshore sands are generally very dense and often silty. In general, design γ_{sub} values in sands should be based on measured laboratory values (corrected for sampling disturbance effects) which should be compatible with relative density (D_r) estimated from q_c and laboratory maximum and minimum dry unit weight values.

• **Parameter D_r**

Common practice is to use the Ticino Sand relationship between q_c and D_r as proposed by Jamiolkowski et al., (1988):

$$D_r = \frac{1}{2.93}\ln\left(\frac{q_c}{205(p'_m)^{0.51}}\right) \qquad \text{(C6.4.3-20)}$$

where

p'_m = soil effective mean in-situ soil stress at depth

z = $(\sigma'_{vo} + 2\,\sigma'_{ho})/3$ with p'_m and q_c in kPa.

Ticino Sand is a medium grained silica sand with no fines. A reasonably comprehensive database is available for this sand (Baldi et al., 1986). However, D_r assessment for the NGI-05 method should be according to Equations C6.4.3-15 and C6.4.3-17. Most $q_c - D_r$ relationships are not valid for silty sands. However, q_c may be adjusted for such materials to derive a “Clean Sand Equivalent Normalised Cone Resistance” (e.g., Youd et al., 2001).

• **Parameter tan δ_{cv}**

The constant volume interface friction angle, δ_{cv}, should be measured directly in laboratory interface shear tests. The recommended test method is by ring shear apparatus, but the direct shear box may also be used. Guidance on test procedures is provided in Jardine et al., (2005).

If site-specific tests cannot be performed, the constant volume interface friction angle may be estimated as a function of mean effective particle diameter (D_{50}) using Jardine et al., (2005). An upper limit of tan δ_{cv} = 0.55 (δ_{cv} = 28.8 degrees) applies to all methods as shown on Figure C6.4.3-1. For materials with unusually weak grains or compressible structures, this method may not be appropriate. Of particular importance are sands containing calcium carbonate, for which specific advice is given in Section C6.4.3e.

C6.4.3e Application of CPT-based Methods

• **‘*t-z*’ Data for Axial Load-deformation Response**

No strain softening is applicable. However, unlike for the method in the main text, the peak unit skin friction in compression and tension at a given depth, $f_{c,z}$ and $f_{t,z}$ are not unique and are both dependent on pile geometry. They depend not only on the pile diameter and wall thickness but also on the pile total penetration. An increased pile penetration will decrease these ultimate values at a given depth.

• **‘*q-z*’ Data for Axial Load-deformation Response**

Unit end bearing (q_p) is assumed to be fully mobilized at a pile tip displacement value of 0.1D_o. This displacement is consistent with the manner in which pile load test data were interpreted.

• **Other Sands—Carbonate Sands, Micaceous Sands, Glauconitic Sands and Volcanic Sands, Silts and Clayey Sands.**

Some cohesionless soils have unusually weak structures/compressible grains. These may require special in-situ and/or laboratory tests for selection of an appropriate design method and design parameters. Reference is made to Thompson and Jardine (1998) and Kolk (2000) for pile design in carbonate sand, and to Jardine et al., (2005) for guidelines on pile

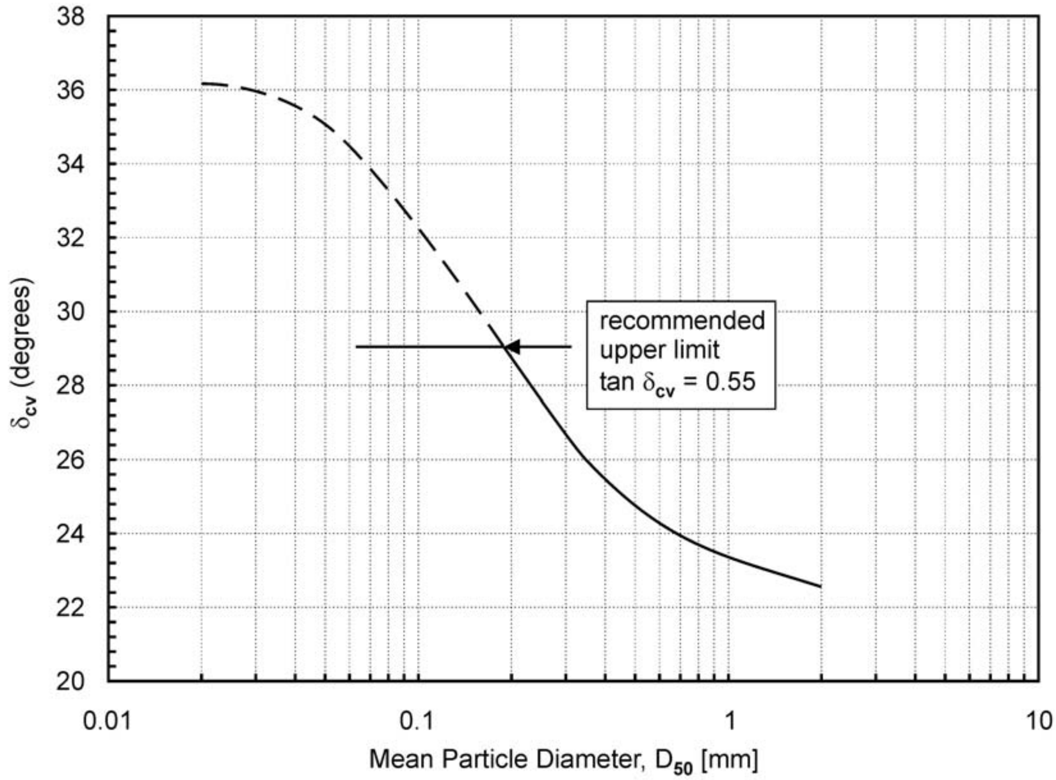

Figure C6.4.3-1—Interface Friction Angle in Sand, δ_{cv}, from Direct Shear Interface Tests

design in other sands and silts. Consideration should be given to using a design method for clays in case of low permeability sands and silts. All former methods should be applied cautiously since limited data are available to support their reliability in these sediments.

- **q_c, in Gravel**

The measured q_c data should not be taken at face value in this cohesionless soil type and appropriate adjustments should be made. For example, CPTs made in (coarse) gravels, especially when particle sizes are in excess of 10 % % of the CPT cone diameter, are misleading, and one possible approach could be to use the lower bound q_c profile. Alternatively, one may estimate an appropriate design q_c profile from adjacent sand layers.

- **Weaker Clay Layers Near Pile Tip**

The use of q_c data averaged between $1.5D_o$ above pile tip to $1.5D_o$ below pile tip level should generally be satisfactory provided q_c does not vary significantly. This may not necessarily be the case when clay layers occur: the q_c data used may have a substantial impact on q_p (fully plugged unit end bearing). If significant q_c variations occur, then the UWA-05 Dutch method (Figure 2.2 of Lehane et al., 2005a) should be used to compute $q_{c,av}$.

Thin (less than around 0.1 D_o thick) clay layers are problematic, particularly when CPT data are discontinuous vertically and/or not all pile locations have been investigated. Factors to be considered should include the variance of layer thickness, strength and compression parameters. If no direct data are available, a cautious interpretation should be made based on the engineering geology of the surrounding sand soil unit. Offshore piles usually develop only a small percentage of q_p under extreme loading conditions. Hence, capacity and settlement calculations, using a finite element model of a pile tip on sand containing weaker layers, may be considered to assess axial pile response under such conditions.

For thick clay layers, shallow geophysical data may be useful to assess layer thickness and elevation. The Main Text recommends reducing the end-bearing component should the pile tip be within a zone up to ± 3D from such layers. When q_c data averaging is also applied to this ± 3D zone, the combined effects may be unduly cautious and such results should be critically reviewed. Similarly, for large diameter [D say > 2 m] piles, the Main Text reduction method should be carefully reviewed.

- **Near-shore and Onshore Piles**

In general, the assumptions (listed in Sections C6.4.3a and C6.4.3c) may not necessarily be valid for near-shore and onshore piles, and should be checked.

Near-shore and onshore pipe piles may respond "unplugged" when loaded (due to insufficient inner friction mobilization). Similarly, dilatancy effects (neglected for offshore piles) may be considered for smaller diameter piles. Scour (especially general scour) may be significant for near-shore pile foundations. In addition closed-ended (rather than open-ended) steel piles may be driven.

The original publications (i.e., CUR, 2001, Jardine et al., 2005, Clausen et al., 2005 and Lehane et al., 2005a) should be consulted for assumptions made and further guidance—most include methods to provide the capacity of "unplugged" pipe piles and closed-ended piles.

- **Scour**

Scour (seabed erosion due to wave and current action) can occur around offshore piles. Common types of scour are (a) general scour (overall seabed erosion) and (b) local scour (steep sided scour pits around single piles or pile groups). There is no generally accepted method to account for scour in axial capacity for offshore piles. Publications like Whitehouse (1998) give techniques for scour depth assessment. In addition, general scour data may be obtained from national authorities.

In lieu of project specific data, Commentary C6.8 gives advice on local scour depth.

Scour decreases axial pile capacity in sand. Both friction and end bearing components may be affected. This is because scour reduces both q_c and σ'_v (vertical effective stress). For excavations (i.e., general scour), NNI (1993) recommends that q_c is simply proportional to σ'_v , i.e.,

$$q_{c,f} = \chi q_{c,o} \quad \text{(C6.4.3-21)}$$

where

$q_{c,f}$ = final (i.e., after general scour) q_c value,

$q_{c,o}$ = original (i.e., before general scour) q_c value,

χ = dimensionless scour reduction factor = $\sigma'_{vf}/\sigma'_{vo}$,

σ'_{vf} = final σ'_v (vertical effective stress) value,

σ'_{vo} = original σ'_v (vertical effective stress) value.

For high general scour depths, an alternative conservative approach (Fugro, 1995) for normally consolidated sands may be to take

$$\chi = \left(\frac{1}{1 + 2K_o} \sqrt{\frac{z_S + 2K_o\sqrt{Sz_S + z_S^2}}{S + z_S}} \right) \quad \text{(C6.4.3-22)}$$

where

z_S = depth below final seabed level = $z - S$,

S = general scour depth.

Commentary C6.8 gives a σ'_v reduction method due to both general and local scour.

C6.4.3f Summary

This commentary has discussed four CPT q_c-based methods for axial pile capacity that incorporate length effects and friction fatigue. Some of these methods have been recently made available in the literature. They have not yet been frequently compared for routine offshore pile projects. Hence, geotechnical engineering judgment will be needed to select the most appropriate method for the design case under consideration.

Additional care is required in cases of clay layers at/near pile tip level.

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COMMENTARY ON SOIL REACTION FOR LATERALLY-LOADED PILES, SECTION 6.8

Note: Commentary on Soil Reactions for Laterally-loaded Piles, Section 6.8 has been added.

C6.8 SOIL REACTION FOR LATERALLY-LOADED PILES

Generally, under lateral loads, clay soils behave as a plastic material which makes it necessary to relate pile-soil deformation to soil resistance. To facilitate this procedure, lateral soil resistance-displacement *p-y* curves should be constructed using stress-strain data from laboratory soil samples. The ordinate for these curves is soil resistance *p* and the abscissa is pile wall displacement, *y*. By iterative procedures, a compatible set of lateral resistance-displacement values for the pile-soil system can be developed.

For a more detailed study of the construction of *p-y* curves, see Matlock (1970) for soft clay, Reese and Cox (1975) for stiff clay, O'Neill and Murchison (1983) for sand and Georgiadis (1983) for layered soils.

Scour (seabed sediment erosion due to wave and current action) can occur around offshore piles. Scour reduces lateral soil support, leading to an increase in pile maximum bending stress. Scour is generally not a problem for cohesive soils, but should be considered for cohesionless soils. Common types of scour are:

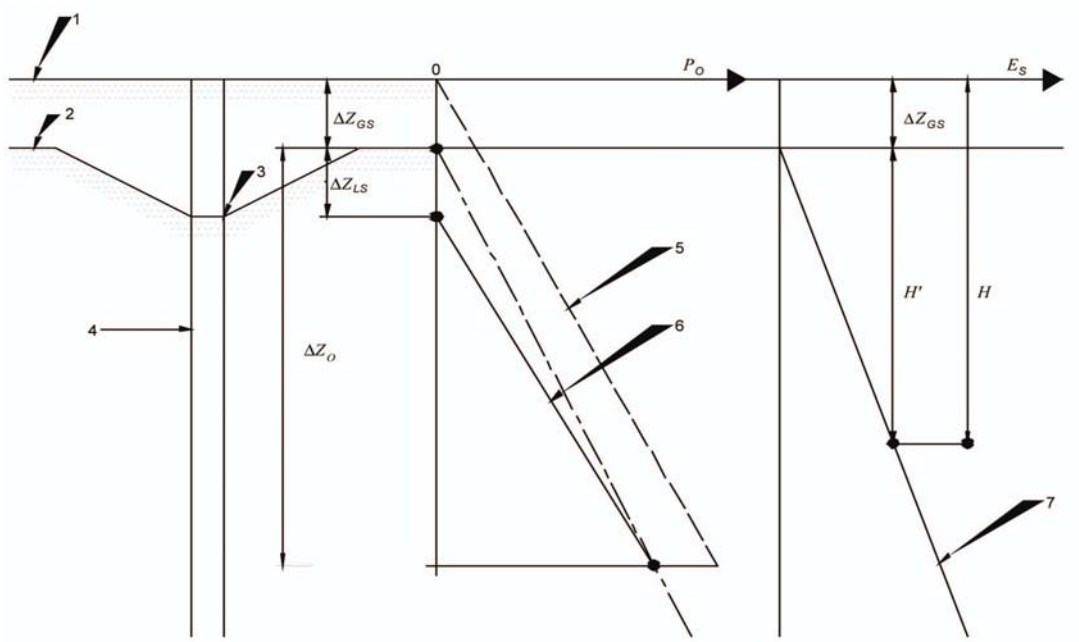
a. general scour (overall seabed erosion), and

b. local scour (steep sided scour pits around single piles).

Publications like Whitehouse (1998) give techniques for scour depth assessment. In addition, general scour data may be obtained from national authorities. In the absence of project specific data, for an isolated pile a local scour depth equal to 1.5*D* and an overburden reduction depth equal to 6*D* may be adopted, *D* being the pile outside diameter; see Figure C6.8-1.

Reduction in lateral soil support is due to two effects:

- a lower ultimate lateral pressure caused by decreased vertical effective stress p_o, and
- a decreased initial modulus of subgrade reaction modulus (E_S).



Key

1	Original sea floor level	ΔZ_{GS}	General scour depth
2	Level after general scour	ΔZ_{LS}	Local scour dept (1,5 × Δ typical)
3	Level of local scour	ΔZ_O	Overburden reduction depth (6,0 × Δ typical)
4	Pile	p_O	Vertical effective stress
5	No scour case	E_s	Initial modulus of subgrade reaction
6	Local scour case	H	Depth below original sea floor
7	$E_s = kH'$	H'	Depth below final general sea floor

Figure C6.8-1—*p-y* Lateral Support—Scour Model

There is no general accepted method to allow for scour in the *p-y* curves for offshore piles. Figure C.6.8-1 suggests one of the methods for evaluating p_o and E_S as a function of scour depths. In this method general scour reduces the p_o profile uniformly with depth, whereas local scour reduces *p* linearly with depth to a certain depth below the base of the scour pit. Subgrade modulus reaction values (E_S) may be computed assuming the general scour condition only. Other methods, based upon local practice and/or experience, may be used instead.

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Note: Commentary on Foundations, Sections 6.14 through 6.17 are unchanged.

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