MEMORANDUM

Subject	Review and design approach for pile foundation structure
Project code	HGI-General
Date	09 June 2021
Reference	version 04
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Appendices	I. The mechanics schematisation of the support condition (α, β) II. Limiting values of structural deformation and foundation movement
To	Budiwan Adi Tirta
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Document log

version	content and/or update	document date
version 00	first draft	27 May 2019
version 01	first draft was improved by the following: • title: "jetty foundation" changed to "pile foundation" • focus changed from review to review and design and as such "for structural modeling" was omitted • add several references and requirements from Eurocode 7 and API • add further explanation on negative skin friction (NEN 9997-1, 2012) and laterally loaded piles (CUR 228, 2010) • changed parameter terminology used in discussion of vertical stiffness to Eurocode-compliant NEN 9997-1	17 June 2020
version 02	version 01 was improved by the following: • fix typographical error • improve sections axial bearing capacity; vertical and horizontal displacement; horizontal and vertical stiffness calculation • clarification on group pile for vertical and lateral loading	08 July 2020
version 03	version 02 was improved by the following: • add requirement on differential settlement • add project reference in considering scouring for bearing capacity • improve explanation on vertical stiffness • expanded common approach to include seismic effect	11 January 2021

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version 04	version 03 was improved by the following: • combined with memo loading test interpretation as an evaluation for pile capacity design	09 June 2021

1 INTRODUCTION

In geotechnical group, most of our pile-related projects are as support for structural for their modelling input. By implication, the typical calculation workload we need to provide are *bearing capacity* and *stiffness constants* (aside from reviewing the design documents, if required). Specifically for bearing capacity, only geotechnical (GEO) failure or ultimate limit state being discussed. As required in Eurocode (§7.8(1)) [ref. 15] that piles shall be verified against structural failure/internal forces checks (unity checks). This part of the work (i.e., STR failure) will be performed by the structural group.

This document mainly discussed procedures and meant to give a sense of general workflow in providing such data to the structural group. This document mostly discuss pile design for static loading. Other theoretical bases is not covered in depth in this document and can be referred to other available literatures. The content of the work will be dependent on the type of service we provide to the Client (i.e., review vs design vs technical assistance). Whichever, this document should give you some sort of idea on the typical geotechnical side of the work.

2 STARTING POINTS

The availability of the following information will depend on the type of service we provide to the Client and at what stage our services include, but Section 2.1 shall be available for our perusal in whichever case.

2.1 Soil information

Obtain the soil investigation report to understand the site (soil vs rock; clayey vs sandy) and the subsoil characteristics (layers, thicknesses). Identify the firm layer.

With several available boreholes, we can also identify which borehole data that provide the thickest soft layer (as the 'representative susceptible soil layer') to be implemented for the pile design/review calculation process (thus conservative result).

2.2 Pile information

If we are reviewing an existing jetty, real 'existing' information is important and usually can be found on as-built drawing or pile driving record (PDR). Pertinent information are as following (but not limited to):

1. installation
    1.1. location/coordinates → check with nearest borehole data; determine whether the tip had reached firm layer or not
    1.2. driven vs bored and equipment used→ back calculation with Hiley formula if possible
    1.3. raking vs upright → lateral capacity
    1.4. toe level, cut-off level, pile length → to determine the free and embedded length of the piles
    1.5. blow count → found rigid layer and check with adjacent soil layer
    1.6. calendaring/final set → check with refusal layer
2. pile type and dimension
    2.1. concrete spun pile/CSP vs steel pipe pile/SPP and its technical specs (relation with corrosion, etc.)
    2.2. diameter
    2.3. thickness

3. pile field test (discussed separately in Section 7 [ref. 14])
    3.1. static loading test
    3.2. pile driving analyzer/PDA

2.3 Monitoring/descriptive information

Any other usable information/cues from the Client or visual observation during field visit, e.g., swaying, tilting, sinking piles that indicate some sort of bearing capacity problems, etc (if any) and maybe helped validate our calculation result.

3 AXIALLY LOADED PILES

The calculation method is largely dependent on the type of information we have (see Table 3.1). The calculation method that we use is American Petroleum Institute (API) code [ref. 3] if a set of good, conclusive engineering properties of each subsoil layer is available.

Otherwise, empirical calculation method via SPT blow count approximation is commonly used. In Indonesia, CPT is less popular so other methods that used q_c as the input is not often used by us. However, SARY wrote a memo [ref. 4] on how to empirically convert N-SPT values to q_c and vice versa with the aid of average grain size (d_{50}) of the soil. This is specifically relevant in Section 4.1.

Table 3.1 summarized the available formulas we most often use for axial bearing capacity calculation. Detailed explanation for each formula can be found in the following sections.

Table 3.1 Summary of bearing capacity calculation methods

	Cohesive soil		**Non-cohesive soil**	
	Driven pile	**Bored pile**	**Driven pile**	**Bored pile**
Unit skin friction, *f*	<u>α method</u> $f = \alpha\ s_u$ with α according to: API (1987) or NAVFAC (1984)	<u>α method</u> $f = \alpha\ s_u$ with α according to: Kulhawy (1984)	<u>β method</u> $f = \beta\ \sigma'_v = \tan\delta\ K\ \sigma'_v$ β according to: NAVFAC DM 7.2 (1984) API (1987) <u>SPT method (OCDI, 2002)</u> f = 2 N [kPa]	<u>SPT method (see Table 3.6)</u> Meyerhoff (1986): f = 1 N [kPa] Quiros & Reese (1977): f = 2.8 N [kPa] Reese & Wright (1977): f = 3.2 N [kPa]
Unit end bearing, *q*	$q = 9\ s_u$		<u>$q = \sigma'_v\ Nq$</u> with Nq according to: API (1987) or NAVFAC (1984) <u>SPT method (OCDI, 2002)</u> q = 300 N [kPa] *or* <u>SPT method (Meyerhof)</u> q = 40 N (L/D) [kPa] commonly used: q = 40 N (L/D) [kPa] < 300 N [kPa]	<u>SPT method (NAVFAC)</u> $q_{bored\ pile} = 1/3\ q_{drivenpile}$

3.1 Bearing capacity analyses and calculation

3.1.1 Understanding structural output/load

From structural output, we are usually given (options of) SLS or ULS load. For the purpose of bearing capacity calculation, we usually use the SLS value, or the 'unfactored' value. This is because of the structural calculation follows the load and resistance factor design (LRFD) method in American code, whereas we use allowable stress design (ASD) method in calculating bearing capacity. Note that the American code does not refer their design load as 'SLS' or 'ULS' per se; we are kind of mixing the terminologies of American and European codes here.

The LRFD concept can be understood like this, where the load is multiplied by a certain factor to account for uncertainty, for example:

(SLS) DL × **1.2**
(SLS) LL × **1.6** +
(ULS)

In ASD method, all sum load were multiplied by a single factor to factor in uncertainties—which for us often to be a 'safety factor' that already include the factors like **1.2** or **1.6** above. This is why we typically use SLS load from structural output, not ULS.

3.1.2 Compression capacity

Ultimate bearing capacity

The ultimate bearing capacity (Q_{ult}) can be computed by,

$$Q_{ult} = Q_f + Q_p \text{ or } f A_s + q A_p$$

where,

Q_f = skin/shaft friction resistance [kN]
Q_p = total end bearing [kN]
f = unit skin friction capacity [kPa]
A_s = side surface area of pile [m^2]
q = unit end bearing capacity [kPa], and
A_p = gross end area of pile [kPa]

To determine the unit skin friction and the unit end bearing capacity, there are several methods available, discussed as follows.

Unit skin/shaft friction (f)

Cohesive soil

1. α method: applicable for driven pile & bored pile

The method is based on undrained shear strength of cohesive soil (s_u), thus it is well suited for short term pile load capacity, as follows,

$$f = \alpha\, s_u$$

where,

s_u = undrained shear strength
α = adhesion coefficient depending on pile material

Presented below are common methods to determine the α value,

1.1. API method [ref. 3] only applicable for driven pile

For $s_u/\sigma'_0 \leq 1$,

$$\alpha = 0.5\left(\frac{s_u}{\sigma'_0}\right)^{-0.5}$$

and for $s_u/\sigma'_0 > 1$,

$$\alpha = 0.5\left(\frac{s_u}{\sigma'_0}\right)^{-0.25}$$

with constraint that $\alpha \leq 1$.

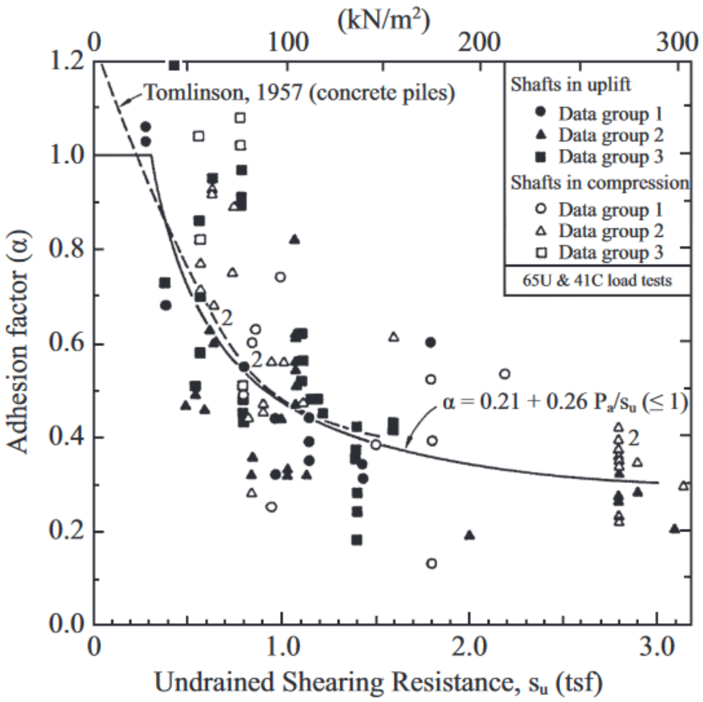
1.2. NAVFAC DM 7.2 [ref. 5]: only applicable for driven piles

Table 3.2 Typical α value vs undrained shear strength by NAVFAC (1984) [ref. 5]

Pile type	Soil consistency	Undrained shear strength, s_u [kPa]	α
Timber and concrete piles	Very soft	0-12	1.00
	Soft	12-24	1.00-0.96
	Medium stiff	24-48	0.96-0.75
	Stiff	48-96	0.75-0.48
	Very stiff	96-192	0.48-0.33
Steel piles	Very soft	0-12	1.00
	Soft	12-24	1.00-0.92
	Medium stiff	24-48	0.92-0.70
	Stiff	48-96	0.70-0.36
	Very stiff	96-192	0.36-0.19

1.3. Kulhawy (1984): only applicable for bored pile

Figure 3.1 Typical α value vs undrained shear strength by Kulhawy (1984); the su is normalized by one atmospheric stress (pa= 101.3 kN/m2



Non-cohesive soil

1. β method: applicable only for driven pile

Computed with following formula,

$$f = \beta\, \sigma'_v = \mu\, K\sigma'_v = \tan\delta\, K\, \sigma'_v$$

The δ and *K* coefficients may be estimated by various methods presented below.

1.1. NAFVAC DM 7.2 [ref. 5]: applicable for various piles

Table 3.3 Pile skin friction angle, δ

Pile type	Pile-soil interface friction angle (°)
Steel	20
Timber	¾ ϕ
Concrete	¾ ϕ

Table 3.4 Lateral earth pressure coefficient, *K*

Pile type	*K* (piles under compression)	*K* (piles under tension)
Driven H-piles	0.5-1.0	0.3-0.5
Driven displacement piles (round and square)	1.0-1.5	0.6-1.0
Driven displacement tapered piles	1.5-2.0	1.0-1.3
Driven jetted piles	0.4-0.9	0.3-0.6
Bored piles (diameter, d<60 cm)	0.7	0.4

1.2. API [ref. 3]: applicable for driven pile - open ended pile

For full displacement piles (driven fully plugged or closed ended) may be assumed 25% higher than the value presented in Table 3.5. The parameters in Table 3.5 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.

Table 3.5 Design parameters for cohesionless siliceous soil [ref. 3]

Relative density	Soil description	Shaft friction factor, β (or '*K* tan δ')	Limiting shaft friction values [kips/ft^2; kPa]	End bearing factor, *N*q	Limiting unit end bearing values [kips/ft^2; MPa]
Very loose (0-15 %) Loose (15-35 %) Loose (15-35 %) Medium dense (35-65 %) Dense (65-85 %)	Sand Sand Sand-silt[1] Silt Silt	n.a.[2]	n.a.[2]	n.a.[2]	n.a.[2]
Medium dense (35-65 %)	Sand-silt[1]	0.29	1.4; 67	12	60; 3
Medium dense (35-65 %) Dense (65-85 %)	Sand Sand-silt[1]	0.37	1.7; 81	20	100; 5
Dense (65-85 %) Very dense (85-100 %)	Sand Sand-silt[1]	0.46	2.0; 96	40	200; 10
Very dense (85-100 %)	Sand	0.56	2.4; 115	50	250; 12

[1]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.
[2]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 of these soils.

2. SPT method: applicable for both driven pile and bored pile

According to OCDI [ref. 6], the unit skin friction for driven pile in non-cohesive soil can be determined using SPT method with the following formula,

$$f = 2\,\overline{N}$$

where,
$\overline{N}$ = mean N-value for total penetration length

Whereas formulas applicable for bored piles can be referred to Table 3.6.

Table 3.6 Formulas pertained for bored piles

Reference	Description	
Touma and Reese (1974)	$q_s = K \tan \sigma'_v \tan \phi'$ < 2.5 tsf (=27.5 t/m^2) where K = 0.7 for $D_b \leq 25$ ft K = 0.6 for $25 < D_b \leq 40$ ft K = 0.5 for $D_b > 40$ ft	
Meyerhof (1976)	q_s [tsf] = N/100 = 0.11 N (t/m^2)	
Quiros and Reese (1977)	q_s [tsf] = 0.026 N < 2 tsf = 0.28 N (t/m^2)	
Reese and Wright (1977)	q_s [tsf] = N/34 = 0.32 N (t/m^2) q_s [tsf] = (N-53)/450 + 1.6	for N < 53 for 53 < N ≤ 100
Reese and O'Neill (1988)	q_s [tsf] = β σ'$_v$ ≤ 2 tsf where β = 1.5-0.135√z (z = depth below ground surface)	for 0.25 ≤β≤1.2

Unit end bearing (q) in soil layer

Cohesive soil

This method is both applicable for driven and bored piles. In cohesive soils, the unit end bearing can be determined according to Terzaghi's bearing capacity equation $q = Nc\, S_u$ where the Nc value is bearing capacity coefficient that can be assumed to 9 [ref. 7].

Thus, end bearing unit computed with following formula,

$$q = 9\, s_u$$

where, s_u is the undrained shear strength of cohesive soil under the base of the pile.

Non-cohesive soil

This method is only applicable for driven piles. In non-cohesive soil, the unit end bearing can be computed with the following Terzaghi equation,

$$q = \sigma'_v\, N_q$$

where the Nq value can be determined from either of the following approach,
- API method, see Table 3.5
- NAVFAC, see Table 3.6

Table 3.7 Friction angle ϕ' vs N_q

ϕ (°)	26	28	30	31	32	33	34	35	36	37	38	39	40
Nq for driven piles	10	15	21	24	29	35	42	50	62	77	86	120	145
Nq for bored piles	5	8	10	12	14	17	21	25	30	38	43	60	72

1.1. SPT Method: applicable for driven pile and bored pile

1.1.1. For driven pile

1.1.1.1. OCDI method [ref. 6]

The end bearing unit in non-cohesive soil can also be computed using SPT method,

$$q = 300\ N$$

where,

N = $(N_1+N_2)/2$

N_1 = N-value at the toe of pile

N_2 = mean N-value in the range from the toe of pile to the level $4B$ above

N = diameter or width of pile [m]

1.1.1.2. Meyerhof method

$$q = 40\ N_{SPT}\frac{l}{D}$$

where,

N–SPT = $(N_1+N_2)/2$

N_1 = average value of N from pile tip to depth 10D above

N_2 = average value of N from pile tip to depth 4D below

D = pile diameter

l = depth from top sand layer to pile tip

1.1.2. For bored pile

According to NAVFAC [ref. 5], the unit end bearing (q) for bored pile are 1/3 from driven pile by using formula mentioned above.

Unit end bearing (q) in rock layer

For driven pile

According to OCDI [ref. 6], if pile is supported on soft rock or hard clay, the bearing capacity can be computed as,

$$q = 300\ N\ \text{[kPa]}$$

If q_u has been measured by undisturbed samples, the following equation may alternatively be used.

$$q = 5\ q_u$$

where,

q_u = unconfined compressive strength [kPa]

Further, the value of q_u should be reduced to 1/2 or 1/3 of the measurement depending on the progress of cracking in natural ground. In any event, however, the value of q_u should not exceed 2x10^4 kN/m^2.

Bending moment check

Bending moment check is performed by moment result from the internal forces check in the Structural group. The resultant bending moment must be 1.1 (material factor) less than the crack bending moment in the specification sheet of the pile (i.e., $1.1 \times M_{crack;pile}$).

3.1.3 Tension/pullout capacity

The ultimate pile pullout capacity must not exceed the Q_f the total skin friction resistance. For conservative approach, the total skin friction can be reduced to 70% of the Q_f value. The effective weight of the pile including hydrostatic uplift and the soil plug also need to be considered in the analysis.

$$Q_{pu} = 0.7Q_f + W_p$$

where,
Q_{pu} = pull-out capacity
Q_f = skin/shaft friction resistance
W_p = pile self-weight

3.2 Safety factor and failure criteria

The API standard [ref. 3], of which we usually use, specifically stated safety factors in Table 3.8 to be applied to the calculated ultimate capacity. The safety factor required in the Indonesian Standard (SNI) [ref. 8] and used in WBI previous project are also included as a comparison.

Table 3.8 Typical recommended bearing capacity safety factors

Standard	Condition	Value	Remarks
API [ref. 3]	Design environmental conditions with appropriate drilling loads	1.5	n.a.
	Operating environmental conditions during drilling operations	2.0	n.a.
	Design environmental conditions with appropriate producing loads	1.5	n.a.
	Operating environmental conditions during producing operations	2.0	n.a.
	Design environmental conditions with minimum loads (for pullout)	1.5	n.a.
SNI [ref. 8]	Any types	min 2.5	A single SF proposed, there were no difference for compressive vs tension/pullout capacity
n.a.			SF that WBI had used in prior projects (that I know of)—the consensus is that we use SF not lower by ones recommended by API.
	Compression	2.0-2.5	e.g., compression 2.0 and tension 2.5 for PT. Vopak at unknown (104563) and for PT Smart at Lampung (108127);
	Tension	2.5-3.0	compression 2.5 and tension 3.0 for Toyo at Bojonegara (INA881-1)

3.3 Critical depth

Critical depth is to be observed in analysis of bearing capacity in coarse grained soil. Critical depth means the unit shaft *f* and unit end bearing *q* of a pile do not increase linearly with depth without bound, but rather are limited to maximum values f_{max} and q_{max} at critical depth z_c and z_b, respectively (may or may not be a same value). The unit end bearing z_b is based on relative density of the coarse grained soil. For reference, z_b = 10*d*; 15*d*; and 20*d* for loose; medium; and compacted sand, respectively [ref. 5].

3.4 Vertical displacement

We followed the Dutch standard (NEN 9997-1 2012) [ref. 9] as given in §7.6.4.2. The displacement on the top side of a foundation of a structure supported by piles as a result of the loads acting on that pile and negative skin friction on the pile, determined by,

$$s = s_1 + s_2$$

where,

s = the amount displacement of the pile at the **top** '*bovenkant*' [m]

s_1 = the displacement of a single pile **top** '*boveneinde*', determined from field test loads result [m]; SARY discussed field test in a separate memo [ref. 14] or for analytical method see Section 3.4.1

s_2 = the initial/immediate compression of the base layer **below the pile toe** [m]; see Section 3.4.2

This formula is based on assumption that the pile was embedded in a hard sand layer. To accommodate clayey base layer, there are additional components to also account for consolidation settlement (Section 3.4.3), or even land subsidence (Section 3.4.5).

The amount of vertical displacement *s*, as expressed in §7.6.1.1.(3), practically limited to: "For piles in compression it is often difficult to define an ultimate limit state from a load settlement plot showing a continuous curvature. In these cases, **settlement of the pile top equal to 10% of the pile base diameter should be adopted as the failure criterion**." Other components of vertical displacement in pile as described below.

3.4.1 Pile toe displacement

Pile toe displacement concerned in this section contributed from the amount of the
If pile displacement s_1 cannot be directly derived from field test, an analytical solution can also be used with following formula,

$$s_1 = s_b + s_{el}$$

where,

s_1 = the displacement of the **top** end '*boveneinde*' of a single pile [mm]; or s_{total}

s_b = the displacement of pile **toe** '*paalpunt*' from the effect of the mobilized load on the pile toe from both toe and shaft capacity [mm]

s_{el} = the elastic deformation at the pile **top** '*boveneinde*' (relative to the pile toe) due to loading to the material of the pile itself [mm]

To further simplify the calculation process, we usually determine s_b arbitrarily so that we can calculate s_{el} easier. See Section 5.2 Step 1-4 for detailed discussion. This displacement is particularly relevant if pile embedded in a rigid layer as in weaker layer the elastic deformation is smaller. This component is usually already included in the Structural modeling.

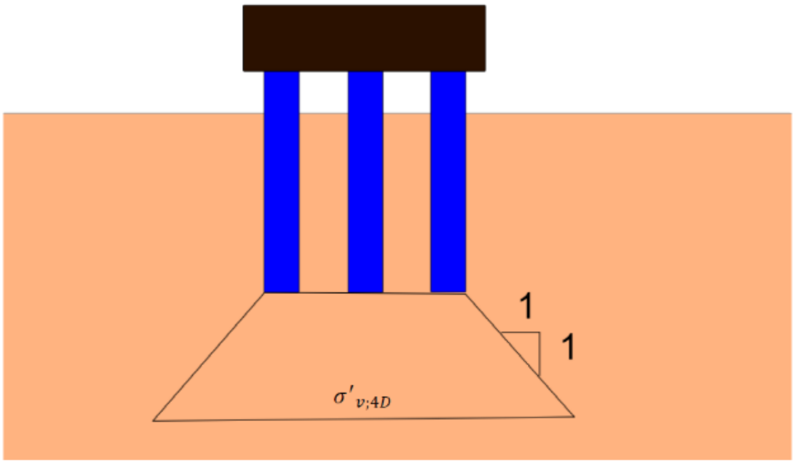
3.4.2 Initial settlement/immediate compression

This section is only relevant if the toe of pile is resting at a hard sand layer. For pile toe embedded in the clay layer, even in stiff consistency, skip to the next section. There are two commonly used method to determine initial/immediate compression of the sand layer below pile toe.

NEN 9997-1

If the pile center-to-center distance (*S*) is more than 10 times the diameter of the pile base, s_2 can be assumed 0. If the S does not fulfill this condition, the initial settlement s_2 must be calculated from the amount of stress being applied from the forces through the pile to the layer below pile toe at 45° angle to all directions (as demonstrated in Figure 3.2).

Figure 3.2 Schematic of stress below pile tip that affecting the initial settlement calculation [ref. 20]



The contribution of settlement s_2 from the sum amount of compression forces in the piles F_{fund} can be determined with the following equation,

$$s_2 = \frac{m^* \times \sigma'_{v;4D} \times 0.9 \times \sqrt{A_{4D}}}{E_{ea;gem}}$$

where,

F_{fund} = the sum of the loads on the group piles [kN]; SLS provided by the Structural group

s_2 = the initial/immediate compression of the base layer below the pile toe [m]

m^* = shape factor of the load surface (see Table 3.9) [-]

$\sigma'_{v;4D}$ = the effective vertical stress due to F_{fund} at the surface $b_1 \times b_2$ at a depth of 4*D* below the pile toe [kPa];

$$\sigma'_{v;4D} = \frac{F_{fund}}{A_{4D}}$$

A_{4D} = the area of the loaded surface at depth of 4*D* below the pile toe [m^2]

D = the smallest diameter of the pile base [m]

$E_{ea;gem}$ = the average modulus of elasticity of the soil below the 4*D* level below the pile toe [kN/m^2]; can also use *E* values in Table 2.b in NEN 9997-1

Table 3.9 Values of *m** [ref. 9]

*m**	shape of the loaded surface							
	circle	square	rectangle b_1/b_2[a]					
			1.5	2	3	5	10	100
	0.96	0.95	0.94	0.92	0.88	0.82	0.71	0.37

[a]) b_1 and b_2 are the dimensions of it at a depth of 4*D* below the pile tips loaded surface, in m, where $b_2 \leq b_1$

Boussinesq method

There is also an alternative to use Boussinesq method which we used when designing the pumping station for Island 2A [ref. 19]. The idea is to assume the load of the structure is impacting the soft layer only and the location of the load is transferred to the top of the soft layer (Figure 3.3). The pressure bulbs arise from that loading is dictated by the Boussiness isobar graph/graphical method. In the pumping station design, we used the square footing isobar (see Figure 3.4).

NEN 9997-1 [ref. 9] also mentioned Boussinesq method although used analytically in §6.6.2.3(d) for shallow foundation/footing. However, it only mentioned for point and strip loadings.

Figure 3.3 Boussinesq method used in calculating settlement for pumping station in KNI Island 2A [ref. 19]

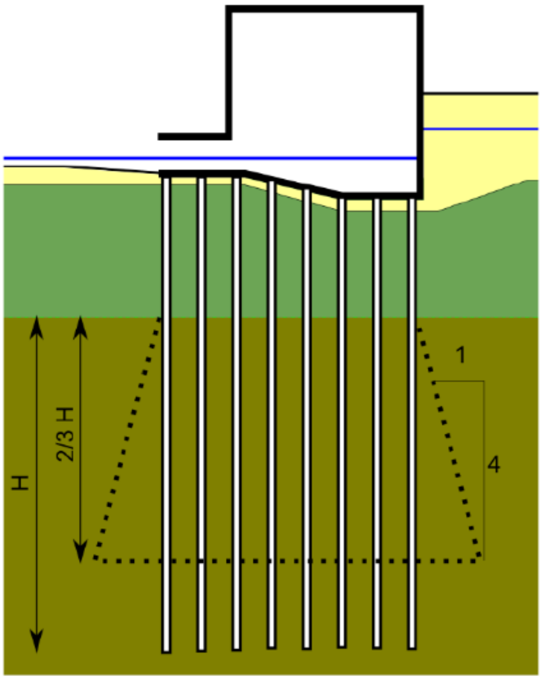
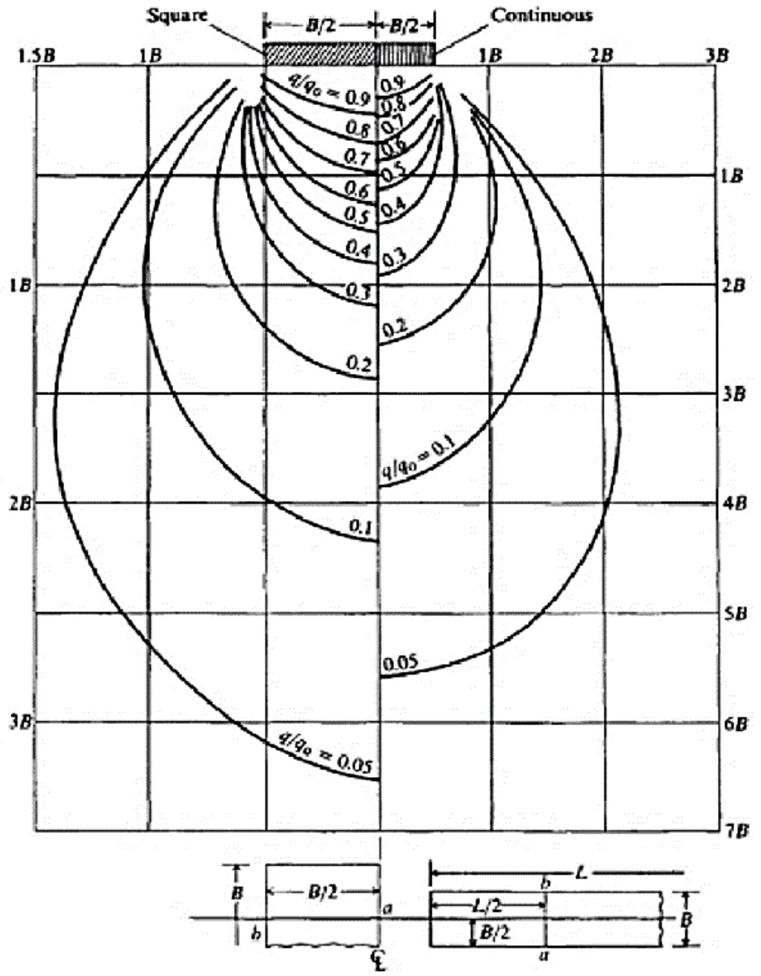


Figure 3.4 Boussinesq pressure isobar/pressure bulbs for square foundation and strip footings



3.4.3 Consolidation settlement

The consolidation settlement is of attention when pile is installed in a compressible layer. The amount of settlement particularly in relation of time (e.g., design lifetime) is often of our interest. The previous method in Section 3.4.2 that presented s_2 without timespan is not preferred and thus more applicable if pile was installed in hard sandy layer. We have experience in designing pile in this situation [ref. 20]. The method utilized the previous method in NEN 9997, expressed as,

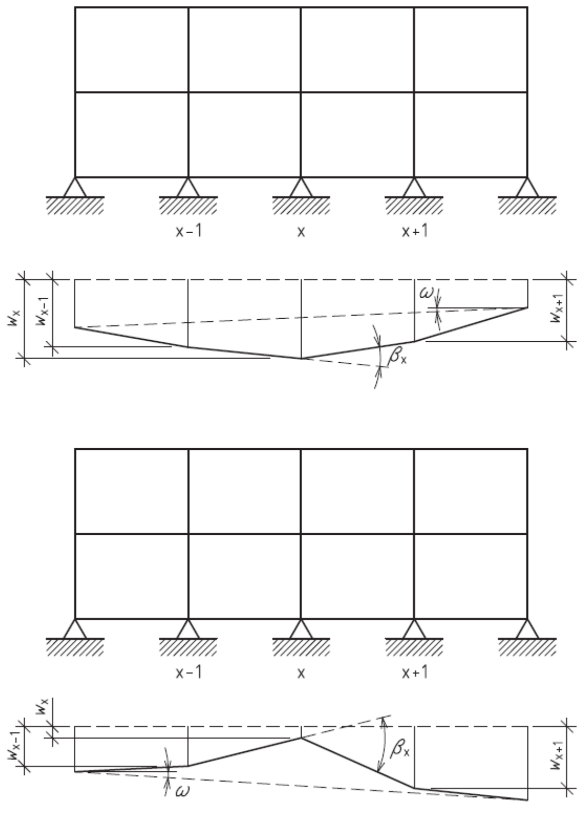
$$\sigma'_{v;4D} = \frac{F_{fund}}{A_{4D}}$$

the nominal $\sigma'_{v;4D}$ then used in the D-settlement model as uniform load with height of 1 m with $\Upsilon_{application}$ at the depth of 4*D* below the pile toe. Soil layer and parameter are modeled as usual. The amount of settlement of interest is within the design lifetime of the structure.

3.4.4 Differential settlement

We considered differential settlement in the cases where in one structure there are more than one designed center-to-center spacing (*S*) in a group pile thus the load distribution to the pile may vary. The load should be obtained from the Structural group, and the settlement calculation can be performed for each typical *S* and the discrepancy is the differential settlement in the structure. The typical guideline value for differential settlement can be found in Appendix II based on Eurocode 7.

Figure 3.5 Skew (ω) and relative rotation (β)



Figuur 1.5.d — Scheefstand (*ω*) en relatieve rotatie (β_X)

The relative rotation (βx) is the rotation angle in foundation point X and its either side of X in a line, i.e., (X - 1) - X and X - (X + 1). Whereas skew (ω) is the angle from the horizontal connecting the top of the outer foundation elements. See sketch in Figure 3.5. From the Dutch annex, the requirement are:

Table 3.10 Differential settlement criteria based on Eurocode (Dutch annex)

condition	criteria
ULS, type B	β=1:100 max
SLS (either or, not both)	ω=1:300 βx=1:300

Usually, most of the deformation values are supplied from the Structural group from each pile. The βx and ω then calculated manually.

3.4.5 Land subsidence

Land subsidence must be considered in a project where soft soil is common, whatever the design object is. Pile is often used to support a structure to a deep hard soil in case of unsuitable soil present on top. In most of coastal area the sediment layer is very thick, thus sometimes the toe of installed pile was not embedded in a hard layer (i.e., floating pile).

As the land subsidence usually occurred in a vast area, the rate we use is uniform throughout the structure. With the exception of a presence of reliable data that showed variation of rate throughout our project area. This is possible if we have a very broad and extensive project area that may cover a variety of land subsidence rate, e.g., a toll road project.

The land subsidence rate adopted for structure supported by pile in practice is the same with the rate for other design object, e.g., embankment [ref. 18].

The land subsidence aspect does not contribute loading to other systems in terms of calculation, e.g., additional effect of negative skin friction. This is because the whole soil system is considered displaced, not just the pile against the soil. Moreover, during design other effects such as previously discussed, already decoupled and separately considered. Land subsidence is used to determine the design level of the structure.

3.5 Other considerations

3.5.1 Effect of negative skin friction

Especially in clayey soil, we want to consider the possible effect on the additional downdrag load that is caused by negative skin friction. Although in jetty settings, there were no additional load/fill on top of the soft soil layer so this effect may be mute. However, it can be included in the calculation in a case where the clay layer is exceptionally thick. Guideline per NCHRP can be used as a criteria, if one of following condition occur:

1. total settlement from the soil surface >100 mm
2. surface settlement after driving the piles were >10 mm
3. fill height of more than 10 m applied on the compressible soil
4. consolidating soil of more than 10 m
5. groundwater lowering for more than 4 m; and
6. pile longer than 25 m.

The Dutch standard (NEN 9997-1 2012) [ref. 9], explicitly determined in §7.6.4.2(4)(c) that if the ground settlement is less than 10 cm, it is unnecessary to consider the negative skin friction. But if it does, the upper limit values of the negative skin friction should usually be used as calculation values for the strength and stiffness of the ground in motion.

The resultant downdrag load can be used to reduce the bearing capacity of the pile or as an additional contributing design load towards the pile. We usually incorporate this negative skin friction as additional design load, not as decrease in bearing capacity (i.e., $F_{tot} = F_{c;rep} + F_{nk;rep}$).

NEN 9997-1 [ref. 9] §7.3.2.2 presented the formula to compute the additional Q_{neg} or $F_{nk;rep}$ in this instance due to soil displacement. WBI had used this method in previous project, such as piling work at KNI Island 2A [ref. 17]. The formula is distinguished by the condition of the center-to-center distance of the piles (S):

(I) single pile, piles in one row/on the edge of a pile group, and pile group that meet → $S > 10 \times D \times d$
(II) piles in the pile group that meet → $S \leq \sqrt{10 \times D \times d}$

where,

D = the diameter of the pile shaft, or the equivalent diameter of the pile shafts of the group [m]
d = the thickness of the layer or layers in which the negative adhesive acts [m]

The formula for **condition (I)** is presented below,

$$F_{nk;rep} = O_{s;gem} \times \sum_{j=1}^{j=n} d_j \times K_{0;j;k} \times \tan\delta_{j;k} \times \frac{\sigma'_{v;j-1;sur;rep} + \sigma'_{v;j;sur;rep}}{2}$$

while the value of $K_{0;j;k} \times \tan\delta_j$ must be at least 0.25

and for **condition (II)**,

$$F_{nk;rep} = A \times \sum_{j=1}^{j=n} \sigma'_{v;j;sur;rep} - \sigma'_{v;j;m;rep}$$

where,

$F_{nk;rep}$	= the representative value of load due to negative skin friction [kN]
$O_{s;gem}$	= the average circumference of the pile shaft and for an in-situ cast pile is the circumference of the pile tip [m]
j	= the compressible layer affected by negative skin friction [-]
n	= the total amount of compressible soil layers where negative skin friction applies [-]
A	= the area of the soil surrounding pile shaft that located in the compressible layer, all within ½ *S* from the pile centreline to both directions [m^2]
$\sigma'_{v;j-1;sur;rep}$	= the representative value of the effective stress at the top of layer *j* (compressible layer) due to surcharge/overburden/sandfill [kPa] (per layer)
$\sigma'_{v;j;sur;rep}$	= the representative value of the effective stress at the bottom of layer *j* (compressible layer) due to surcharge/overburden/sandfill, without the influence of other piles [kPa] (per layer)
$\sigma'_{v;j;m;rep}$	= the representative value of amount decrease in the effective stress in the bottom layer *j* (compressible layer) caused by negative skin friction distributed among pile group members [kPa] (per layer)

$$\frac{\gamma'_{j;k}}{m_j} \times \left(1 - e^{-m_j \times d_j}\right) + \sigma'_{v;j-1;m;rep} \times e^{-m_j \times d_j}$$

with,

$m_j = \frac{O_{s;gem}}{A} \times K_{0;j;k} \times \tan\delta_j$; while the value of $K_{0;j;k} \times \tan\delta_j$ must be at least 0.25

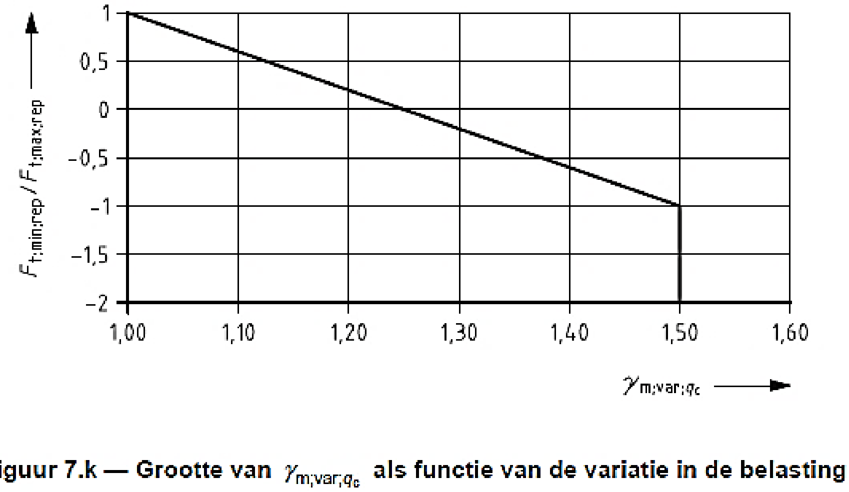
d_j	= the thickness of layer *j* (compressible layer) (per layer)
$\gamma'_{j;k}$	= the effective unit weight [kN/m^3]; submerged unit weight γ_{sat}-γ_w if underwater, γ_{dry} if above water
$K_{0;j;k}$	= the at rest lateral earth coefficient [-]; $K_{0;j;k}$ = 1-sin φ' (per layer)
$\tan\delta_j$	= the characteristic value of interface angle [-] (remember, per layer): for cast in-situ concrete pile, δ=φ' for precast concrete, timber pile, and steel cast pile, δ=0.75φ'

The NEN 9997-1 (2012) [ref. 9] formula is based on the pile resting on the stiff layer. However in practice we also use this formula in cases where pile toe is located in compressible layer. This is acceptable in terms of the amount of resultant load is greater than the actual load (i.e., conservative), although we neglected the amount of the pile vertical displacement. For obtain the displacement, we supply the pile downdrag load and bearing capacity to the Structural group.

3.5.2 Effect of fatigue

Pile structure, especially for jetty application where there is variable load, where the pile load varies between a maximum (tension) and a minimum (compression) value, the friction between pile and soil will decrease. The fatigue is incorporated in the calculation by additional factor (to the factor that was already required in the codes). The determination of the magnitude of the applied safety factor to account for the fatigue is as described in EN-NEN 9997-1 [ref. 9] (see Figure 3.2).

Figure 3.6 The magnitude of fatigue additional factor as a function of the extreme load ratio [ref. 9]



where,

$F_{t;min;rep}$ = is the minimum value of the axial load in kN. If the pile is also loaded with compression, the minimum load withdrawn is negative (compression <0)

$F_{t;max;rep}$ = is the maximum value of the axial tensile load, in kN (tensile >0)

$\gamma_{m;var;qc}$ = is a factor that reflects the impact of fatigue (or 'fatigue SF')

For a conservative result, the maximum magnitude of 1.5 is used as the **additional** factor irrespective of the load ratio.

3.5.3 Efficiency and group effect

Bearing capacity of a single pile is different of a group pile configuration. From literature there are various method to determine the group efficiency (E_g). Below are the two common ones based on the type of the soil the piles are embedded in. This section is extracted from a book by H.C. Hardiyatmo, Analisis dan Perancangan Fondasi II [ref. 10]. As an additional information, SNI [ref. 8] stated that in general group reduction factor does not have to be considered if the pile is spaced 8*d* center-to-center and shall not be less than pile circumference. For circle pile, spacing shall not be less than 2.5*d*. **Please note that this group effect only relevant for vertical loading.** For group effect in lateral loading, see Section 5.1.

In cohesive soil

In cohesive soil, the increase of compression zone with the increasing amount of piles in a group will thus reducing the overall structure allowable bearing capacity. O'Neill (1983) observed that in clay soil, the E_g is always lower than 1. The E_g is dependent on the excess pore pressure dissipation caused by the pile driving which took longer time in a group pile configuration. As excess pore pressure build up occurred near the pile, a sparser pile group configuration is preferred in clayey soil as seen in Table 3.9. Kerisel (1967) proposed following E_g for various pile distance that can be used as a guideline.

Table 3.11 Recommended group efficiency for piles in clay soil by Kerisel (1967) (*d*=pile diameter)

Center to center distance	Group efficiency, E_g
10*d*	1
8*d*	0.95
6*d*	0.90
5*d*	0.85
4*d*	0.75

Center to center distance	Group efficiency, E_g
3*d*	0.65
2.5*d*	0.55

Otherwise, an empirical Converse-Labarre formula can be used to calculate the E_g in cohesive soil (as endorsed in SNI [ref. 8]).

$$E_g = 1 - \theta \frac{(n' - 1)m + (m - 1)n'}{90mn'}$$

where,

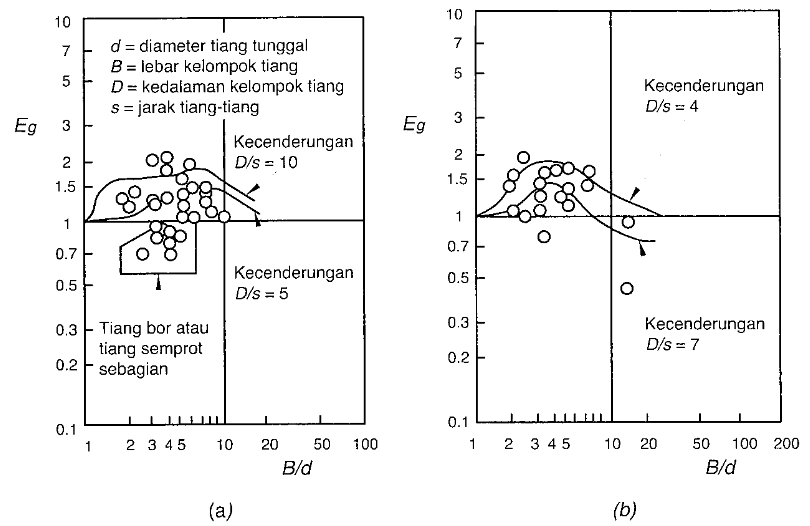
E_g = is a group efficiency
m = is the amount of rows in the pile group
n' = is the amount of columns in the pile group
θ = is the $\tan^{-1}\frac{d}{s}$ [°]
d = is the pile diameter [m]
s = is the center-to-center distance among the piles [m]

In granular soil

Vesic (1967) observed model pile group configured in a narrow spacing in homogenous sandy soil, the group efficiency is always greater than 1. O'Neill (1983) also produced a similar observation with full scale pile group model test as provided in Figure 3.3. In general, the E_g of driven piles tend to increase with the most conservative E_g value of 1. Predrilling and water jetting (i.e., damage to soil structure and reduce the earth pressure) will reduced the group bearing capacity.

This increase of E_g can be attributed to several reasons. First, increase of effective lateral pressure will also increase shaft friction. Second, pile group in narrow spacing will increase the relative density of the soil thus increasing the soil internal friction angle. Similarly, there are also difference in piling in loose and dense sand soil. The pile driving in loose sand will increase the shear strength due to compaction, whereas in dense sand, dilative behavior of sand will reduce the density thus decreasing the group bearing capacity.

Figure 3.7 Efficiency of full scale pile group (O'Neill 1983). Figure (a) is relevant to pile in jetty settings where the pile cap is not in contact with the top soil. *B/d* = width of pile group/single pile diameter



Gambar 2.52 Efisiensi tiang dari uji beban skala penuh untuk tiang dalam tanah granuler (O'Neill, 1983). (a) Pelat penutup tiang tidak menyentuh tanah. (b) Pelat penutup tiang menyentuh tanah.