Geocentrix

Repute 2.5

Reference Manual

Onshore pile design and analysis

Set in Optimum using Corel® WordPerfect® X7. Printed in the UK.

Acknowledgments

Repute 2.0 was developed with the generous support of Corus, Atkins, and Stent Foundations.

Repute was designed and written by Dr Andrew Bond of Geocentrix, with assistance from Ian Spencer of Honor Oak Systems.

PGroupN was designed and written by Dr Francesco Basile of Geomarc. Special recognition goes to the late Dr Ken Fleming of Cementation Foundations Skanska for his invaluable advice and support during the development of PGroupN.

The *Repute Reference Manual* was written by Andrew Bond and Francesco Basile.

The following people and organizations assisted with the production of the program and its documentation: Francesco Basile, Jenny Bond, Joe Bond, Tom Bond, Jack Offord, and Claire Bond.

Revision history

Last revised 28th July 2021 (for version 2.5.11).

Table of contents

Acknowledgments 2
Revision history 2
Table of contents 3

Chapter 1
Documentation 4

Chapter 2
Calculations 5
Boundary element analysis 5
Fleming's analysis 13
Longitudinal ULS 13
Randolph's analysis 15
Validation 15

Chapter 3
Design standards 17
Partial (safety) factors 17

Chapter 4
Actions 19
Sign convention 19
Combinations of actions 19
Forces 20
Moments 20

Chapter 5
Material and section properties 21
Soils, rocks, and fill 21
Concretes 27
Steels 28
Bearing piles 28
Circular section 28
Custom section 28
Rectangular section 29

Chapter 6
Algorithms 30
Alpha algorithm 30
Bearing capacity algorithm 31
Bearing pressure limit 32
Beta algorithm 32
Earth pressure coefficient 32
No contact algorithm 33
Plugging algorithm 33
Rock bearing algorithm 33
Rock friction algorithm 34
Rock pressure coefficient 34
Skin friction limit 35
35

Chapter 7
References 36

Chapter 1
Documentation

Repute is supplied with a detailed *Quick-Start Guide*, comprehensive *User Manual*, and authoritative *Reference Manual*. The latest versions of these manuals (including any corrections and/or additions since the program's first release) are available in electronic (Adobe® Acrobat®) format from the Geocentrix website. Please visit www.geocentrix.co.uk/repute and follow the links to Repute's documentation.

Quick-Start guide

The *Repute Quick-Start Guide* includes six tutorials that show you how to use the main features of Repute. Each tutorial provides step-by-step instructions on how to drive the program. There are three tutorials dealing with single pile design and three with pile group design. The tutorials increase in difficulty and are designed to be followed in order.

User manual

The *Repute User Manual* explains how to use Repute. It provides a detailed description of the program's user interface, which is being rolled out across all of Geocentrix's software applications. The manual assumes you have a working knowledge of Microsoft Windows, but otherwise provides detailed instructions for getting the most out of Repute.

Reference manual (this book)

The *Repute Reference Manual* gives detailed information about the engineering theory that underpins Repute's calculations. The manual assumes you have a working knowledge of the geotechnical design of single piles and pile groups, but provides appropriate references for further study if you do not.

Chapter 2 Calculations

Repute® provides a variety of calculations that you can perform on single-piles and pile-groups:

- "Boundary element analysis" predicts the load vs displacement behaviour of a single pile or pile group under vertical, horizontal, and moment loading
- "Fleming's analysis" predicts the load vs settlement behaviour of a single pile
- "Longitudinal ULS" checks the ultimate limit state of a single pile under vertical loading
- "Randolph's analysis" predicts the settlement of a single pile
- "Validation" checks single piles and pile groups are properly specified

Boundary element analysis

Repute's boundary element analysis predicts the load vs displacement behaviour of a single pile or pile group using the calculation engine PGroupN, developed by Dr Francesco Basile of Geomarc. PGroupN provides a complete 3D non-linear boundary element solution of the soil continuum, i.e. the simultaneous influence of all the pile elements within the group is considered. This overcomes limitations of traditional interaction-factor methods and gives more realistic predictions of deformations and the load distribution between piles.

The PGroupN program is based on a complete boundary element (BEM) formulation, extending an idea first proposed by Butterfield and Banerjee [1] and then developed by Basile [2], [3], [4]. The method employs a substructuring technique in which the piles and the surrounding soil are considered separately and then compatibility and equilibrium conditions are imposed at the interface. Given unit boundary conditions, i.e. pile group loads and moments, these equations are solved, thereby leading to the distribution of stresses, loads and moments in the piles for any loading condition.

A general pile group arrangement is shown in **Figure 1** to **Figure 3** (refer to Chapter 4 for the full definition of forces and sign convention).

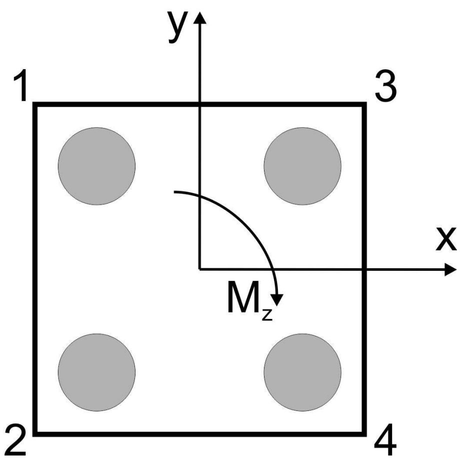


Figure 1. Plan view of a 2 x 2 pile group in the XY plane

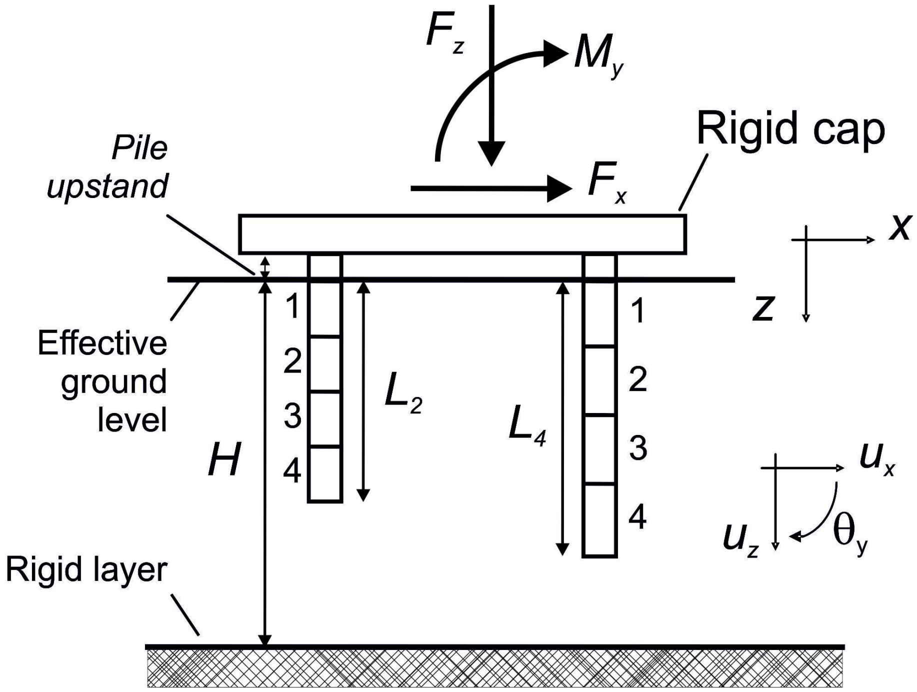


Figure 2. Profile of a 2 x 2 pile group in the XZ plane

Modelling the pile-soil interface (interface discretization)

The PGROUPN analysis involves discretization of only the pile-soil interface into a number of cylindrical elements, while the base is represented by a circular (disc) element. The behaviour of each element is considered at a node which is located at the mid-height of the element on the centre line of the pile. The stress on each element is assumed to be constant, as shown in **Figure 3**.

With regard to the axial and torsional response, the pile-soil interface is discretized into a number N of shaft cylindrical elements over which (axial) shear stresses and torsional stresses are applied, while the base is represented by a circular (disc) element over which normal stresses are acting.

With regard to the lateral response in the *X*- and *Y*-directions (which are considered separately), the pile is assumed to be a thin rectangular strip which is subdivided into a number N of rectangular elements. Only normal stresses on the compressive face are considered. Further, if the pile base is assumed to be smooth, the effects of the tangential components of stresses over the base area can be ignored. Thus, each pile is characterised by ($4N$+1) surface elements (where '+1' accounts for the base element). As an example, with reference to the pile-soil interface discretization into N = 6 elements illustrated in **Figure 3**, the vector of soil tractions (t_s) has a dimension equal to 25 (i.e. six components for the axial soil tractions on the shaft plus one axial component on the base, six components for the transverse soil tractions in the *X*-direction, six components for the transverse soil tractions in the *Y*-direction, and six components for the torsional soil tractions in the *XY* plane).

Modelling the soil (soil domain)

The boundary element method involves the integration of an appropriate elementary singular solution for the soil medium over the surface of the problem domain, i.e. the pile-soil interface. With reference to the present problem which involves an unloaded ground surface, the well-established solution of Mindlin [5] for a point load within a homogeneous, isotropic elastic half space has been adopted. The soil deformations at the pile-soil interface are related to the soil tractions via integration of the Mindlin's kernel, yielding:

$$\{u_s\} = [G_s]\{t_s\}$$

where $\{u_s\}$ are the soil displacements, $\{t_s\}$ are the soil tractions and $[G_s]$ is a flexibility matrix of coefficients obtained from Mindlin's solution for the axial and lateral response.

The off-diagonal flexibility coefficients are evaluated by approximating the influence of the continuously distributed loads by discrete point loads applied at the location of the nodes. The singular part of the diagonal terms of the $[G_s]$ matrix is calculated via analytical integration of the Mindlin functions. This is a significant advance over previous work (e.g. PGROUP) where these have been integrated numerically, since these singular integrals require considerable computing resources. Further computational efficiency is achieved by exploiting symmetries and similarities in forming single-pile and interaction flexibility matrices. This reduces the computational time and renders the analysis practical for large groups of piles.

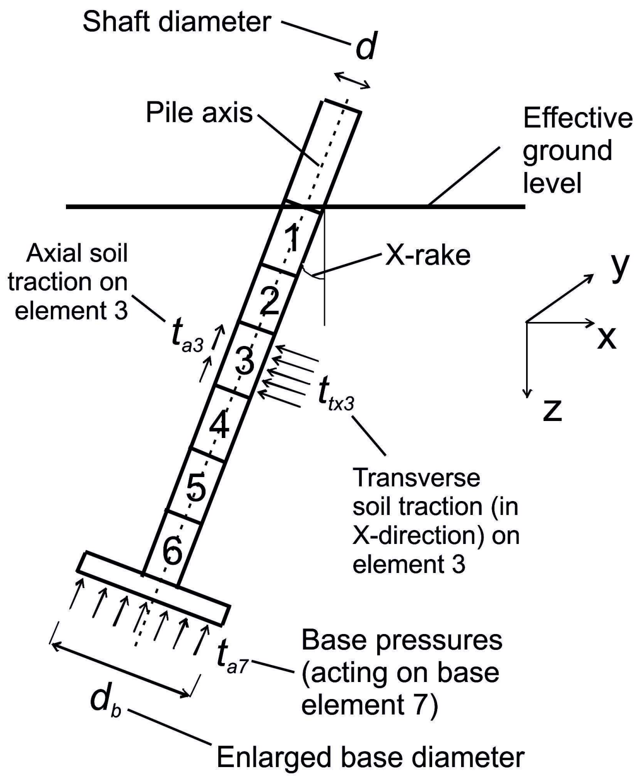


Figure 3. Discretization of the pile-soil interface into N = 6 shaft elements

Treatment of Gibson and multi-layered soil profiles

Mindlin's solution is strictly applicable to homogeneous soil conditions. However, in practice, this limitation is not strictly adhered to, and the influence of soil non-homogeneity is often approximated using some averaging of the soil moduli. PGroupN handles Gibson soils (i.e. soils whose stiffness increases linearly with depth) and generally multi-layered soils according to an averaging procedure first examined by Poulos [6] and widely accepted in practice [7], [8], [9], [10], [11], i.e. in the evaluation of the influence of one loaded element on another, the value of soil modulus is taken as the mean of the values at the two elements.

Poulos's procedure is adequate in most practical cases but becomes less accurate if large differences in soil modulus exist between adjacent elements or if a soil layer is overlain by a much stiffer layer (Poulos [12]). In such cases, the alternative procedure proposed by Yamashita et al. [13] may be adopted for the axial response analysis. For the generic element *i*, this procedure calculates an equivalent value of soil modulus on the basis of weighted average values of soil modulus over 4 elements above and 4 elements below the element *i*. At the pile top, the averaging process is curtailed so as not to include non-existent elements. At the pile base, in order to consider the influence of soil layers below the pile tip, the equivalent value also takes into account the values of soil modulus down to a depth equal to the height of 4 'imaginary' elements below the pile base (Note: these elements are termed 'imaginary' because only the pile-soil interface is discretised into elements, i.e. there are no 'real' elements below the pile base.)

It should be pointed out that the Poulos averaging procedure does not consider the influence of soil layers located below pile tip.

Rigid layer

Mindlin's solution has been used to obtain approximate solutions for a layer of finite thickness by employing the Steinbrenner approximation [14] to allow for the effect of an underlying rigid layer in reducing the soil displacements (Poulos [12]; Poulos and Davis [15]). If a rigid layer is defined, it must be the last (i.e. bottom) layer.

It is assumed that the rigid layer, which is considered to be semi-infinite in extent, cannot be located higher than 110% of the embedded length of the longest pile in the group.

Modelling the piles (pile domain)

If the piles are assumed to act as simple beam-columns which are fixed at their heads to the pile cap, the displacements and tractions over each element can be related to each other via the elementary beam theory, yielding:

$$\left\{u_p\right\} = \left[G_p\right]\left\{t_p\right\} + \{B\}$$

where $\{u_p\}$ are the pile displacements, $\{t_p\}$ are the pile tractions, $\{B\}$ are the pile displacements due to unit boundary displacements and rotations of the pile cap, and $[G_p]$ is a matrix of coefficients obtained from the elementary (Bernoulli-Euler) beam theory.

Solution of the system

Applying the previous two equations via compatibility and equilibrium constraints at the pile-soil interface, leads to the following system of equations:

$$\left\{t_p\right\} = -\left[G_p + G_s\right]^{-1}\{B\}$$

where $[G_p + G_s]$ is the global square matrix of the pile group.

By successively applying unit boundary conditions, i.e. unit vertical displacement, unit horizontal displacements (in the *X*- and *Y*-directions) and unit rotations (in the *XZ*, *YZ*, and *XY* planes) to the pile cap, it is possible to obtain the system of vertical loads, horizontal loads (in the *X*- and *Y*-directions) and moments (in the *XZ*, *YZ*, and *XY* planes) acting on the cap that are necessary to equilibrate the stresses developed in the piles.

Thus, if an external loading system F_z (vertical load), F_x (horizontal load in the *X*-direction), M_y (moment about the *Y*-axis), F_y (horizontal load in the *Y*-direction), M_x (moment about the *X*-axis), M_z (torsional moment about the *Z*-axis) is acting on the cap, the corresponding vertical displacement (u_z), horizontal displacement in the *X*-direction (u_x), rotation about the *Y*-axis (θ_y), horizontal displacement in the *Y*-direction (u_y), rotation about the *X*-axis (θ_x), and rotation about the *Z*-axis (θ_z) of the cap are related via:

$$\begin{Bmatrix} F_z \\ F_x \\ M_y \\ F_y \\ M_x \\ M_z \end{Bmatrix} = [K] \begin{Bmatrix} u_z \\ u_x \\ \theta_y \\ u_y \\ \theta_x \\ \theta_z \end{Bmatrix}$$

where the coefficients of the 6 x 6 [*K*] matrix are the equilibrating forces as discussed above. The [*K*] matrix represents the global stiffness matrix of the pile-soil system which may be used as a boundary condition for the superstructure analysis.

It is reasonable to assume that there is no interaction between the horizontal response in *X* and *Y* directions, i.e. the stiffness coefficients K_{24}, K_{25}, K_{34}, K_{35}, K_{42}, K_{43}, K_{52} and K_{53} are all equal to zero [16]. By inverting the global stiffness matrix [*K*], it is possible to obtain the global flexibility matrix [*F*] of the pile-soil system and hence the pile cap deformations may be obtained for any loading condition:

$$\begin{Bmatrix} u_z \\ u_x \\ \theta_y \\ u_y \\ \theta_x \\ \theta_z \end{Bmatrix} = [F] \begin{Bmatrix} F_z \\ F_x \\ M_y \\ F_y \\ M_x \\ M_z \end{Bmatrix}$$

In order to obtain the tractions acting on the piles for the prescribed loading conditions, the pile tractions due to unit boundary conditions from the equation for $\{t_p\}$ must be scaled using the cap displacements and rotations obtained from the last equation. Finally, integrating the axial, transverse, and torsional tractions acting on the piles, yields the distribution of axial forces, shear forces and moments acting on each pile.

Limiting pile-soil stresses

The foregoing procedure is based on the assumption that the soil behaviour is linear-elastic. However, if soil yielding is considered, it is then necessary to ensure that the stress state at the pile-soil interface does not violate the yield criteria. This can be achieved by specifying the limiting stresses at the pile-soil interface.

Fine (cohesive) soils

For fine soils, a total stress approach is adopted.

The ultimate unit shaft resistance (q_s) is calculated according to BS 8004:2015 as:

$$q_s = \alpha \cdot c_u$$

where c_u is the undrained shear strength of the soil and α is an empirical coefficient (adhesion factor) with a default value of 0.5.

The ultimate unit base resistance (q_b) is calculated according to BS 8004:2015 as:

$$q_b = N_c \cdot c_u$$

where N_c is a bearing capacity factor with a default value of 9.

The ultimate unit transverse resistance (q_{tr}) is calculated as:

$$q_{tr} = N_{c,tr} \cdot c_u$$

where $N_{c,tr}$ is a bearing capacity factor that increases linearly with depth (z) from ground surface, as was originally suggested by Broms [17] and now widely adopted in practice [18], according to:

$$N_{c,tr} = 2 + 7\left(z/3D\right) \leq 9$$

where D is the pile diameter.

Coarse (cohesionless) soils

For coarse soils, an effective stress approach is adopted.

The ultimate unit shaft resistance (q_s) is calculated according to BS 8004:2015 as:

$$q_s = K_s \cdot \tan\delta \cdot \sigma'_v$$

where K_s is the coefficient of horizontal soil stress, δ the angle of friction between pile and soil, and σ'_v the vertical effective stress in the free field.

The ultimate unit base resistance (q_b) is calculated according to BS 8004:2015 as:

$$q_b = N_q \cdot \sigma'_v$$

where N_q is a bearing capacity factor that depends on the soil's angle of shearing resistance (φ), By default, N_q is calculated using the equation established by Berezantzev *et al*. [19] and approximated in Fleming *et al*. [57] by:

$$N_q = 10^{7.5\cdot\left(\frac{\varphi}{100}-0.1\right)}$$

where φ is entered in degrees.

The ultimate unit transverse resistance (q_{tr}) is calculated according to Fleming *et al*. [57] as:

$$q_{tr} = K_p^2 \cdot \sigma'_v = \left(\frac{1+\sin\varphi}{1-\sin\varphi}\right)^2 \sigma'_v$$

where K_p is the passive earth pressure coefficient.

Rocks

For rocks, an approach based on unconfined compressive strength is adopted.

The ultimate unit shaft resistance (q_s) is calculated according to BS 8004:2015 as:

$$q_s = k_1 \cdot p_a \cdot \left(q_u / p_a\right)^{k_2}$$

where q_u is the unconfined compressive strength of the intact rock; p_a is atmospheric pressure (100 kPa); and k_1

and k_2 are constants. With the default values k_1 = 0.79 and k_2 = 0.5, this equation reproduces that proposed by Poulos and Bunce [20] (with q_s *and* q_u in MPa):

$$q_s = 0.25 \cdot \sqrt{q_u}$$

The ultimate unit base resistance (q_b) is calculated according to BS 8004:2015 as:

$$q_b = k_3 \cdot p_a \cdot \left(q_u / p_a\right)^{k_4}$$

where k_3 and k_4 are constants. With default values k_3 = 2.5, k_4 = 1.0, and p_a = 1 MPa, this equation reproduces that proposed by Tomlinson [52] and Poulos [21] (where q_b *and* q_u are in MPa):

$$q_b = 2.5 \cdot q_u$$

The ultimate unit transverse resistance (q_{tr}) is calculated according to Poulos [21] as:

$$q_{tr} = \eta \cdot q_b$$

where η is a factor that increases linearly with depth (z) from ground surface, according to:

$$\eta = k_5 \left(1 + z / D\right) \leq 1.0$$

where D is the pile diameter and k_5 = 0.22 by default.

Group "shadowing" effect

Under lateral loads, closely spaced pile groups are subjected to a reduction of lateral capacity. This effect, commonly referred to as "shadowing", is related to the influence of the leading row of piles on the yield zones developed in the soil ahead of the trailing row of piles. Because of this overlapping of failure zones, the front row will be pushing into virgin soil while the trailing row will be pushing into soil which is in the shadow of the front row piles. A consequence of this loss of soil resistance for piles in a trailing row is that the leading piles in a group will carry a higher proportion of the overall applied load than the trailing piles. This effect also results in gap formation behind the closely spaced piles and an increase in group deflection. It has been shown both theoretically and experimentally that the shadowing effect becomes less significant as the spacing between piles increases and is relatively unimportant for centre-to-centre spacing greater than about six pile diameters [22], [23], and [24].

The shadowing effect has been modelled into the PGroupN analysis using the approach outlined by Fleming et al. [57]. Following this approach, it has been assumed that a form of block failure will govern when the shearing resistance of the soil between the piles is less than the limiting resistance of an isolated pile. Referring to **Figure 4**, the limiting lateral resistance for the pile which is in the shadow of the front pile may be calculated from the lesser of the limiting bearing stress for a single pile and $2\frac{s}{d}t_s$, where s is the centre-to-centre pile spacing, d is the pile diameter and t_s is the friction on the sides of the block of soil between the two piles. The value of t_s may be taken as c_u for fine (cohesive) soils and $\sigma'_v \tan\varphi'$ for coarse (cohesionless) soils.

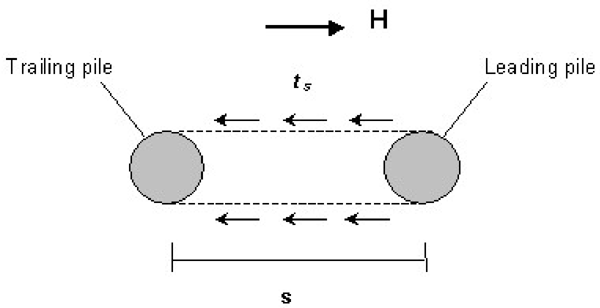


Figure 4. Plan view of block failure under lateral load (after Fleming et al, 1992)

The outlined approach provides a simple yet rational means of estimating the shadowing effect in closely spaced groups, as compared to the purely empirical "p-multiplier" concept which is employed in load-transfer analyses (e.g. in GROUP [25]).

Extension to non-linear soil behaviour

Non-linear soil behaviour has been incorporated by assuming that the soil Young's modulus varies with the stress level at the pile-soil interface. A simple and popular assumption is to adopt a hyperbolic relationship between soil stress and strain, in which case the tangent Young's modulus of the soil E_{tan} is given by (see [12], [26], [27]):

$$E_{\tan} = E_i \left(1 - \frac{R_f t}{t_s}\right)^2$$

where E_i is the initial tangent soil modulus, R_f is the hyperbolic curve-fitting constant, t is the pile-soil stress and t_s is the ultimate unit pile-soil stress obtained from equations for the limiting pile-soil stress. Thus, the boundary element equations described above for the linear response are solved incrementally using the modified values of soil Young's modulus of given above and enforcing the conditions of yield, equilibrium and compatibility at the pile-soil interface. For the pile-soil interface elements that have yielded, no more stress increase is permitted and therefore any increase in load is redistributed between the remaining elastic elements until all elements have failed. It is noted that yielding of an element introduces a discontinuity in the material property and, therefore, the use of Mindlin's solution to determine the remaining elastic coefficients is only approximate. However, previous work indicates that the errors engendered by this approach are slight (e.g. Basile [2], Poulos [12]).

The hyperbolic curve fitting constant R_f defines the degree of curvature of the stress-strain response and can range between 0 (an elastic-perfectly plastic response) and 0.99 (R_f = 1 is representative of an asymptotic hyperbolic response in which the limiting pile-soil stress is never reached). Different values of R_f should be used for the axial response of the shaft and the base, for the shaft lateral response, and for the shaft torsional response.

For the axial response of the shaft, values of R_f in the range 0-0.75 are generally used, while the base axial response is highly non-linear and therefore values of R_f in the range 0.90-0.99 are appropriate (e.g. [12], [28]). For the lateral and torsional response of the shaft, values of R_f in the range 0.50-0.99 generally give a reasonable fit with the observed behaviour.

The best way to determine the values of R_f is by back-fitting the PGroupN load-deformation curve with the measured data from a full-scale pile loading test. In the absence of any test data, the values of R_f can be estimated based on experience and, as a preliminary assessment, the following values may be adopted: R_f = 0.5 (shaft), R_f = 0.99 (base), R_f = 0.9 (lateral), and R_f = 0.99 (torsional).

Finally, it should be noted that, in assessing the lateral response of a pile at high load levels, the assumption of a linear elastic model for the pile material becomes less valid and may lead to an underestimation of pile deflections.

Assumptions about the pile cap

The pile cap is considered to be perfectly rigid and hence does not bend under load.

If a material (concrete) is assigned to the pile cap, then its weight is added to any applied loads acting on the pile group. If no material is assigned, the then pile cap is considered to be weightless.

Raked piles

Forces and bending moments in raked piles are given in local coordinates, i.e relative to an axis running along the centre of the raked pile.

Options

The following options control the way the Boundary Element Analysis is performed.

*Option (*default)*	*Possible values (*default)*	*Controls*
Drainage condition	Drained* Undrained	Use of drained or undrained strength stiffness (undrained values are only available for Soil Layers containing Fine Soils)
Stress-strain model	Linear-elastic Bi-linear* Hyperbolic	Stress-strain curve for all ground materials (ground stength is ignored when the linear-elastic model is selected)
Layer averaging	Poulos* Yamashita	Method used to calculate strength and stiffness values at selected nodes along the pile shaft. Poulos' method uses the spot value at the depth of the node; Yamashita's method uses a weighted average below and above the node.
Degrees of freedom	(read only)	Measure of the complexity of the design situation

Precision

The following options control the precision of the Boundary Element Analysis.

*Option (*default)*	*Possible values (*default)*	*Controls*
Automatic	On*/off	Whether the precision is set automatically or specified manually
Mesh granularity	Fine* Medium Coarse Custom	[Can only be set when the Automatic flag is off] For single piles subject to no vertical load, the number of nodes along the pile shaft (*N*) is set to L/D. For all other pile configurations, *N* is set to L/2D for fine; L/3D for medium granularity; or L/4D for coarse granularity.
Number of elements per pile	(varies)	[Can only be set when the Mesh granularity is Custom] The number of nodes along the pile shaft (*N*) is set to L/D.
Number of load increments	a) 1* b) 25–500, 200*	a) Applies when stress-strain model is linear-elastic a) Applies when stress-strain model is bi-linear or hyperbolic

Boundary element engine

The following options control the output generated by the boundary element engine.

*Option (*default)*	*Possible values (*default)*	*Controls*
Version	(read only)	
Work folder	(read only)	Where temporary work files are created during the analysis
Save results	On/off*	Whether result files are saved or not. Files are saved in the same folder as the RPX file and named: “<Name>^<Calcuation>.<ext>”, <Name> = name of RPX file, <Calculation> = name of calculation, <ext> = file extension
Create XML output	On*/off	Whether the engine produces results in text (XML) format If this flag is off, no results will be displayed in Repute
Create TXT output	On/off*	Whether the engine produces results in text (TXT) format
Create PGW output	On/off*	Whether the engine produces input data in PGroupN-Win (PGW) format. Turn on when advised by Geocentrix Technical Support
Create PGN output	On/off*	Whether the engine produces input data in PGroupN-DOS (PGN) format. Turn on when advised by Geocentrix Technical Support
Create LOG output	On/off*	Whether the engine produces a log (LOG) file Turn on when advised by Geocentrix Technical Support
Use Maths Kernel Library	On*/off	Whether the Intel MKL Library is used to invert the boundary element analysis’ stiffness matrix

Algorithms

The following algorithms can be linked to the Boundary Element Analysis:

- Skin friction limit
- Bearing pressure limit
- Plugging
- Undrained adhesion (alpha = fs/cu)
- Earth pressure coefficient (Ks)
- Wall friction (delta/phi)

- Bearing capacity in soil
- Rock friction (alpha_q = fs/qu)
- Rock bearing (Nqu)
- Rock pressure coefficient (k5)

Fleming's analysis

Fleming's analysis predicts the load vs settlement behaviour of a single pile. The analysis is based on the method described in Fleming's paper *A new method for single pile settlement prediction and analysis* [29].

The total load applied to the pile is given by:

$$P = \left(\frac{U_s \times s}{M_s \times D_s + s} \right) + \left(\frac{D_b \times E_b \times U_b \times s}{0.6 \times U_b + D_b \times E_b \times s} \right)$$

where:

D_b = base diameter
D_s = shaft diameter
E_b = base stiffness (modulus of soil beneath the pile base)
P = axial force applied to the pile
M_s = shaft flexibility factor (0.004 in soft to firm or relatively loose soils; ~0.0005 in very stiff soils or soft rock; 0.001-0.002 in stiff overconsolidated clays)
s = total pile head settlement, assuming the pile is purely rigid
U_b = ultimate base load
U_s = ultimate shaft load

The above equation can be solved to give the total pile head settlement for any applied force:

$$s = \frac{-g \pm \sqrt{g^2 - 4fh}}{2f}$$

$$f = D_b E_b \left(P - U_s - U_b \right)$$

$$g = 0.6 U_b \left(P - U_s \right) + E_b M_s D_b^2 \left(P - U_b \right)$$

$$h = 0.6 M_s D_b U_b P$$

The elastic shortening of the pile shaft under load can be estimated from:

$$P \le U_s, s_e = \frac{4}{\pi} \times \frac{P\left(L_0 + K_E \times L_F \right)}{D_s^2 \times E_c}$$

$$P > U_s, s_e = \frac{4}{\pi} \times \frac{P\left(L_0 + L_F \right) - L_F U_s \left(1 - K_E \right)}{D_s^2 \times E_c}$$

where:

E_c = Young's modulus of elasticity of the pile material in compression
K_E = factor for calculating effective column length (usually ~0.45 in stiff overconsolidated clays)
L_F = length of pile involved in frictional load transfer
L_0 = length of pile which is friction-free or carries low friction
s_e = elastic shortening of pile

Values of the parameters are normally found by a curve-fitting exercise. See Fleming's paper [loc. cit.] for examples. This method is also implemented in the computer program CEMSET, described in that paper.

Longitudinal ULS

Longitudinal ULS checks the ultimate limit state of a single pile under vertical loading.

The design effect of actions F_d is given by:

$$F_d = \sum \gamma_G F_{G,k} + \sum \psi \gamma_Q F_{Q,k}$$

where:

γ_G = partial factor on permanent actions (≥ 1.0)
$F_{G,k}$ = characteristic permanent action
ψ = combination factor (≤ 1.0)
γ_Q = partial factor on variable actions (≥ 1.0)
$F_{Q,k}$ = characteristic variable action

The design resistance R_d is given by:

$$R_d = \left(\frac{\int_{z=0}^{z=L} f_s A_s dz}{\gamma_s \times \gamma_{Rd}} \right) + \left(\frac{q_b A_b}{\gamma_b \times \gamma_{Rd}} \right)$$

where:

f_s = skin friction against the pile shaft
A_s = circumferential area of pile shaft (per unit length)
z = depth below ground surface
L = length of pile shaft
q_b = unit end-bearing resistance of pile base
A_b = area of pile base
γ_s = partial factor on shaft resistance
γ_b = partial factor on base resistance
γ_{Rd} = model factor on pile resistance

In undrained horizons

The skin friction f_s in undrained horizons is given by:

$$f_s = \alpha \times c_{u,d}$$

where:

α = adhesion factor (= 0.5 by default)
$c_{u,d}$ = design value of the undrained strength along the pile shaft

The end bearing resistance q_b in undrained horizons is given by:

$$q_b = N_c c_{u,b,d} + \sigma_{v,b}$$

where:

N_c = end-bearing coefficient (= 9 by default)
$c_{u,b,d}$ = design value of the undrained strength below the pile base
$\sigma_{v,b}$ = vertical total stress below the pile base

The $\sigma_{v,b}$ term is only included in q_b when the self-weight of the pile is treated as an action. Otherwise it is ignored.

In drained horizons

The skin friction f_s in drained horizons is given by:

$$f_s = K_s \sigma'_v \tan \delta$$

where:

K_s = lateral earth pressure coefficient against the shaft (= 0.7 by default)
σ'_v = vertical effective stress in the free-field at the relevant level along the pile shaft
δ = angle of interface (wall) friction (= 0.5 x soil's angle of shearing resistance, by default)

The end bearing resistance q_b in drained horizons is given by:

$$q_b = N_q \sigma'_{v,b} + \sigma_{v,b}$$

where:

N_q = end-bearing coefficient (= Terzaghi's algorithm, by default)
$\sigma'_{v,b}$ = vertical effective stress below the pile base
$\sigma_{v,b}$ = vertical total stress below the pile base

The $\sigma_{v,b}$ term is only included in q_b when the self-weight of the pile is treated as an action. Otherwise it is ignored.

Randolph's analysis

Randolph's analysis predicts the settlement of a single pile. The analysis is based on the method described in the book *Piling engineering* by Fleming et al. [30].

The load/settlement ratio of the pile head is given by:

$$\frac{P}{G_l r_0 s} = \frac{\left[\dfrac{4\eta}{(1-\nu)\xi}\right] + \left[\dfrac{2\pi\rho}{\zeta} \times \dfrac{\tanh(\mu l)}{\mu l} \times \dfrac{l}{r_0}\right]}{1 + \left[\dfrac{4\eta}{\pi\lambda(1-\nu)\xi} \times \dfrac{\tanh(\mu l)}{\mu l} \times \dfrac{l}{r_0}\right]}$$

where:

P = axial force applied to the pile
s = total pile head settlement
$\eta = r_b/r_0$ = ratio of under-ream for under-reamed piles
$\xi = G_l/G_b$ = ratio of end-bearing for end-bearing piles
$\rho = \overline{G}/G_l$ = variation of soil modulus with depth
$\lambda = E_p/G_l$ = pile/soil stiffness ratio
$\zeta = \ln(r_m/r_0)$ measure of radius of influence of pile
$\mu l = \sqrt{(2/\lambda\zeta)} \times (l/r_0)$ measure of pile compressibility

See Fleming et al.'s [57] book for examples.

Validation

Validation checks that single piles and pile groups are properly specified.

The following conditions are flagged as errors (and subsequent calculations are aborted):

- Ground is missing
- Borehole is missing
- Borehole has no layers
- Layer weight density is not specified
- Groundwater is above ground level
- Standing water is below ground level
- Pile foundation is missing
- Toe of the longest pile is below the bottom of the borehole
- Two or more piles are at the same (x, y) position on plan
- Two or more pile heads have different depths (i.e. they lie on the same horizontal plane)
- Actions are missing

Warnings are given if any of the following conditions arise:

- Water table is missing
- Two or more piles are raked towards each other
- Design standard is missing

In addition, when a boundary element analysis is performed, the following conditions are flagged as errors (and the subsequent analysis is aborted):

- Number of piles exceeds 350
- Number of layers exceeds 50
- Number of pile elements is less than 3 or greater than 50
- Number of load increments is greater than 500
- Torque is applied to the pile group and one or more piles have an asymmetrical rake: if the pile is double-raked (in both the X and Y directions), then it must be symmetrically raked, i.e. the absolute values of the angles of rake in the X and Y directions must be the same (only when torsion loading is present)
- Piles are too close together (i.e. the smallest spacing to diameter ratio is less than 2.5)
- Piles are too stubby (i.e. the smallest slenderness ratio is less than 5)
- Layer stiffness is not specified (large-strain stiffness is checked for linear-elastic and linear-elastic/perfectly-plastic analyses; small-strain stiffness for a non-linear analysis)

Chapter 3
Design standards

Repute supports the following design standards:

- British Standard BS 8004: 1986 [31], now superseded by BS 8004: 2015
- Draft Eurocode 7 ENV 1997-1: 1994 [32], now superseded by Eurocode 7
- Eurocode 7 EN 1997-1: 2004 [33], with no National Annex
- British Standard BS EN 1997-1:2004+A1:2013 [34], Eurocode 7 with the UK National Annex
- Irish Standard IS EN 1997-1: 2007 [35], Eurocode 7 with the Irish National Annex
- NTC08 (Italian National Building Code) 2008 [36]
- Singapore Standard SS EN 1997-1: 2010 [37], Eurocode 7 with the Singapore National Annex
- British Standard BS 8004: 2015 [38]
- Custom Working Stress, user-defined (with default values from BS 8004:1986)
- Custom Eurocode 7, user-defined (with default values from Eurocode 7)

Partial (safety) factors

The following symbols are used in this chapter to represent partial (and other safety) factors that are employed in pile design.

γ_G	partial factor on (unfavourable) permanent action
γ_Q	partial factor on (unfavourable) variable action
γ_A	partial factor on (unfavourable) accidental action
$\gamma_{G,fav}$	partial factor on favourable permanent action
γ_b	partial factor on base resistance of pile
γ_s	partial factor on shaft resistance of pile
γ_t	partial factor on total (i.e. shaft + base) resistance of pile
γ_{st}	partial factor on shaft resistance of pile in tension
γ_{Rd}	model factor on pile resistance
γ_φ	partial factor on coefficient of shearing resistance of soil
γ_c	partial factor on effective cohesion of soil
γ_{cu}	partial factor on undrained strength of soil
γ_{qu}	partial factor on unconfined compressive strength of rock

The values of these factors are given in the following table.

Design standard (Cases A-C) (Design Approaches 1-3)		Partial factors on ...								
		Actions/effects				Resistance ($\gamma_\varphi = \gamma_c = \gamma_{cu} = \gamma_{qu} = 1.0$)				
		γ_G	γ_Q	γ_A	$\gamma_{G,fav}$	γ_b	γ_s	γ_t	γ_{st}	γ_{Rd}
BS 8004:1986		1.0	1.0	1.0	1.0	3.0	1.0	2.0	2.0	1.0
ENV 1997-1	A	1.0	1.5	1.0	0.95	1.3[d] 1.6[b] 1.45[c]	1.3	1.3[d] 1.5[b] 1.4[c]	1.6	1.5
	B	1.35	1.5	1.0	1.0					
	C	1.0	1.3	1.0	1.0					
EN 1997-1	1[1]	1.35	1.5	1.0	1.0	1.0[d] 1.25[b] 1.1[c]	1.0	1.0[d] 1.15[b] 1.1[c]	1.25	-

Design standard (Cases A-C) (Design Approaches 1-3)		Partial factors on ...								
		Actions/effects				Resistance ($\gamma_{\varphi} = \gamma_{c} = \gamma_{cu} = \gamma_{qu} = 1.0$)				
		γ_{G}	γ_{Q}	γ_{A}	$\gamma_{G,fav}$	γ_{b}	γ_{s}	γ_{t}	γ_{st}	γ_{Rd}
	1[2]	1.0	1.3	1.0	1.0	1.3[d] 1.6[b] 1.45[c]	1.3	1.3[d] 1.5[b] 1.4[c]	1.6	-
	2	1.35	1.5	1.0	1.0	1.1	1.1	1.1	1.15	-
	3	1.35 1.0	1.5 1.3	1.0	1.0	γ_{R} = 1.0 but with $\gamma_{\varphi} = \gamma_{c}$ = 1.25, $\gamma_{cu} = \gamma_{qu}$ = 1.4				-
Eurocode 7 with UK National Annex	1[1]	1.35	1.5	1.0	1.0	1.0	1.0	1.0	1.0	-
	1[2†]	1.0	1.3	1.0	1.0	2.0[r] 1.7[d]	1.6[r] 1.5[d]	2.0[r] 1.7[d]	2.0[r] 1.7[d]	1.4
	1[2‡]					1.7[r] 1.5[d]	1.4[r] 1.3[d]	1.7[r] 1.5[d]	1.7[r] 1.5[d]	
	1[2¶]									1.2
Eurocode 7 with Irish NA		Same as Eurocode 7								1.75
NTC08	1[1]	1.35	1.5	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	1[2]	1.0	1.3	1.0	1.0	1.45[d] 1.7[b] 1.6[c]	1.45	1.45[d] 1.6[b] 1.55[c]	1.6	1.0
	2	1.35	1.5	1.0	1.0	1.15[d] 1.35[b] 1.3[c]	1.15	1.15[d] 1.2[b] 1.25[c]	1.25	1.0
Eurocode 7 with Singapore NA		Same as Eurocode 7 with UK National Annex								
BS 8004:2015		Same as Eurocode 7 with UK National Annex								
b = bored piles; c = continuous flight auger (CFA) piles; d = driven piles; r = replacement piles (bored and CFA) †with no pile tests; ‡with control tests on 1% of piles;¶with investigation tests										

Chapter 4
Actions

Repute® implements the following actions:

- Combinations of actions
- Forces
- Moments

Sign convention

The sign convention adopted in Repute is illustrated in **Figure 5**. The symbols F_x, F_y, and F_z represent positive forces along the x, y, and z axes respectively; s_x, s_y, and s_z are displacements in the corresponding directions; M_x, M_y, and M_z represent clockwise moments about those axes; and θ_x, θ_y, and θ_z are the corresponding clockwise rotations.

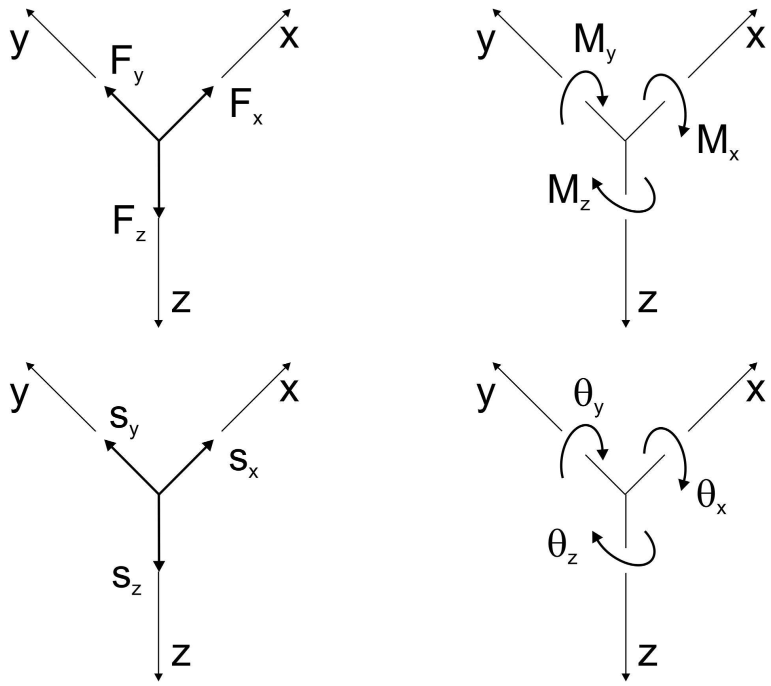


Figure 5. Sign convention used in Repute 2 for (top-left) forces, (top-right) moments, (bottom-left) displacements, (bottom-right) rotations

In cross-section view, the y-axis goes into the screen/paper; in elevation view, the x-axis comes out of the screen/paper; and, on plan view, the z-axis goes into the screen/paper.

The sign convention adopted by Repute 2.x differs from that used in Repute 1.x (which was based on the old PGROUP convention, illustrated in **Figure 6**). The symbols H_x, H_y, and V represented forces along the x, y, and z axes respectively (H for horizontal force, V for vertical); and M_x and M_y were clockwise moments along the x- and y-axes, respectively. Since torque was not supported, there was no symbol for the moment about the z-axis.

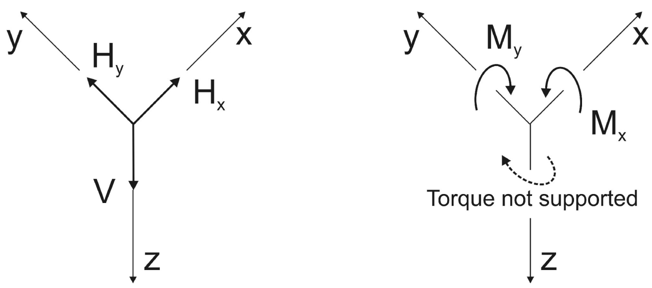


Figure 6. Sign convention used in Repute 1.x for (left) forces and (right) moments

Combinations of actions

A combination of actions may include any number of forces and any number of moments.

The components of the combination's resultant force are given by:

$$F_{x,c} = \sum_{i=1}^{n_{fx}} F_{x,i}, F_{y,c} = \sum_{i=1}^{n_{fy}} F_{y,i}, F_{z,c} = \sum_{i=1}^{n_{fz}} F_{z,i}$$

where the summations are made over each- force i in the combination (assuming that the number of forces F_x, F_y, and F_z are n_{fx}, n_{fy}, and n_{fz} respectively; and the number of moments M_x, M_y, and M_z are n_{mx}, n_{my}, and n_{mz} respectively).

The components of the combination's resultant moment are given by:

$$M_{x,c} = \sum_{i=1}^{n_{mx}} M_{x,i} + \sum_{i=1}^{n_{fy}} F_{y,i}\left(z_i - z_c\right) - \sum_{i=1}^{n_{fz}} F_{z,i}\left(y_i - y_c\right)$$

$$M_{y,c} = -\sum_{i=1}^{n_{fx}} F_{x,i}\left(z_i - z_c\right) + \sum_{i=1}^{n_{my}} M_{y,i} + \sum_{i=1}^{n_{fz}} F_{z,i}\left(x_i - x_c\right)$$

$$M_{z,c} = \sum_{i=1}^{n_{fx}} F_{x,i}\left(y_i - y_c\right) - \sum_{i=1}^{n_{fy}} F_{y,i}\left(x_i - x_c\right) + \sum_{i=1}^{n_{mz}} M_{z,i}$$

where the summations are made over each moment i in the combination (assuming the same number of individual forces and moments given above).

Forces

A force is fully specified by its components F_x, F_y, and F_z along the x, y, and z axes, respectively. The resultant force is given by:

$$F = \sqrt{F_x^2 + F_y^2 + F_z^2}$$

A component of force is considered positive when its acts in the axis's positive direction.

Moments

A moment is fully specified by its components M_x, M_y, and M_z around the x, y, and z axes, respectively. The resultant moment is given by:

$$M = \sqrt{M_x^2 + M_y^2 + M_z^2}$$

A component of moment is considered positive when it rotates clockwise about the respective axis, when looking in the axis's positive direction.

Chapter 5
Material and section properties

Repute® allows you to specify properties for the following materials:

- Soils, rocks, and fill
- Concretes
- Steels

Repute also allows you to specify properties for the following sections:

- Bearing piles
- Circular section
- Custom section
- Rectangular section

Soils, rocks, and fill

Repute implements the following soils:

- Gravel, Sand, Coarse Silt, Granular Fill, and Custom Granular Soil
- Silt, Clay, Cohesive Fill, Organic Soil, River Soil, and Custom Cohesive Soil
- Chalk, Rock

These soils are further described according to the Re/x Soil Classification System, which is based on the terms defined in EN ISOs 14688 [39] and 14689 [40].

The following table lists the soils that are included in the Re/x Soil Classification System and give the corresponding group symbols from each of the established systems listed above (where they are available).

Soil	*Symbol*	*Class*	*Possible states*
Gravel	Gr CGr MGr FGr siGr clGr	GRAVEL* Coarse GRAVEL Medium GRAVEL Fine GRAVEL silty GRAVEL clayey GRAVEL*	Unspecified (-) Very loose (V. loose)¶ Loose Medium dense (Med. dense) Dense Very dense (V. dense)
Sand	Sa CSa MSa FSa siSa clSa	SAND* Coarse SAND Medium SAND Fine SAND silty SAND clayey SAND*	*Same as GRAVEL*
Coarse silt	CSi	Coarse SILT	*Same as GRAVEL*
Silt	Si saSi clSi	SILT*† sandy SILT*† clayey SILT*†	*Same as CLAY*
Clay	Cl grCl saCl siCl Lam	CLAY*†$ gravelly CLAY*† sandy CLAY*† silty CLAY*† Laminated CLAY*†	Unspecified (-)*$ Extremely low strength (Extr. low) Very low strength (V. low) Low strength Medium strength*$ High strength*$ Very high strength (V. high)*$ Extremely high strength (Extr. high)*$

Soil	*Symbol*	*Class*	*Possible states*
Organic	Or siOr clOr Peat Loam	ORGANIC SOIL† siltyl ORGANIC SOIL† clayey ORGANIC SOIL† PEAT† LOAM†	*Same as CLAY*
Granular fill	Mg rock-Mg slag-Mg grMg saMg chalk-Mg brick-Mg ash-Mg pfa-Mg	MADE GROUND MADE GROUND (rock) MADE GROUND (slag) gravelly MADE GROUND sandy MADE GROUND MADE GROUND (chalk) MADE GROUND (brick) MADE GROUND (ash) MADE GROUND (PFA)	Unspecified (-) Poorly-compacted (PC) Well-compacted (WC)
Cohesive fill	clMg siMg	clayey MADE GROUND† silty MADE GROUND†	*Same as CLAY*
Chalk	Chk Chk1 Chk2 Chk3 Chk4 Chk5 Chk6	Unclassified* Grade I* Grade II* Grade III* Grade IV* Grade V Grade VI	Unspecified (-)
Rock	Rock Marl	Weathered rock* Marl*	Unspecified (-)
River soil	River mud Dock silt Alluvium	River mud† Dock silt† Alluvium†	Extremely low strength (Extr. low) Very low strength (V. low) Low strength
Custom granular	-	Unclassified*$	*Same as GRAVEL*
Custom cohesive	-	Unclassified*†$	*Same as CLAY*

*may have effective cohesion (if symbol appears next to Class & State)
†may be undrained
$may be fissured (if symbol appears next to Class & State)
¶potential for liquefaction

Database of soil properties

Repute uses a database of soil properties to check that any parameters you enter for a soil are compatible with that soil's engineering description. The program's checking system is based on the concept that there are *normal* and *extreme* ranges for each soil parameter. An error message is issued when:

- The soil is marked for 'strict validation' and you enter a value that is outside the *normal* range for a particular soil parameter
- You enter a value that is outside the *extreme* range, regardless of whether the soil is marked for strict validation

Default parameters are provided for all soil types. These are provided to assist in initial design studies only and should not be used as a substitute for measured parameters. As in all forms of geotechnical design, parameters should be chosen on the basis of adequate site investigation, including suitable laboratory and field measurements.

The publications that have been referred to in compiling the database include:

- Terzaghi & Peck [41]
- NAVFAC DM-7 [42]
- Peck, Hanson, and Thornburn
- Winterkorn and Fang [43]
- Canadian Foundation Engineering Manual [44]
- Reynolds and Steedman [45]

- Bell [46]
- Mitchell [47]
- TradeARBED's *Spundwand-Handbuch Teil 1, Grundlagen* [48]
- Bolton [49]
- Clayton and Militiski [50]
- Clayton [51]
- Tomlinson [52]
- British Steel's *Piling Handbook* [53]

Invaluable advice regarding the properties of various soils was provided by J.B. Burland, the late P.R. Vaughan, D.W. Hight, and G. Sills.

Mass/weight densities

The following table gives Re/x database values for dry density (ρ_d).

Soil classification		Dry density ρ_d (kg/m^3)				
		Default	Strict validation		Relaxed validation	
Class	State		Minimum	Maximum	Minimum	Maximum
(All) Gr	-	2050	1400	2200	1200	2500
	V. loose	1500	1300	1600	1200	1800
	Loose	1650	1400	1800	1300	2000
	Med. dense	1850	1500	2000	1400	2200
	Dense	2050	1700	2200	1500	2400
	V. dense	2250	2000	2400	1700	2500
(All) Sa	-	1675	1275	1800	1200	2200
	V. loose	1450	1225	1550	1200	1750
	Loose	1500	1275	1600	1225	1850
	Med. dense	1575	1350	1700	1275	1950
	Dense	1675	1450	1800	1350	2050
	V. dense	1800	1575	1900	1450	2200
(All) Si	All	1850	1275	2150	1100	2200
(All) Cl	-	2050	1500	2200	1200	2500
	Ext. low	1650	1400	1800	1200	2000
	V. low	1650	1400	1800	1200	2000
	Low	1750	1500	1900	1300	2100
	Med	1900	1650	2050	1450	2250
	High	2050	1800	2200	1600	2400
	V. high	2200	1950	2350	1750	2450
	Ext. high	2300	2100	2400	1900	2500
Or	All	1500	1000	2050	800	2250
siOr		1500	1250	1600	1000	1750
clOr		1500	1250	1600	1000	1750
Peat		1200	1000	1300	800	1400
Loam		1900	1650	2050	1450	2250
Mg	All	1600	1225	1800	600	2500
rock-Mg		1900	1500	2100	1400	2200
slag-Mg		1450	1200	1600	1000	1800
grMg		1950	1400	2200	1200	2500
saMg		1600	1225	1800	1200	2200
chalk-Mg		1350	1300	1400	1250	1450
brick-Mg		1600	1300	1750	1100	1900
ash-Mg		1000	650	1000	600	1200
pfa-Mg		1350	1000	1500	900	1700
clMg/siMg	All	1550	1100	1750	950	1900

Soil classification		Dry density ρ_d (kg/m^3)				
		Default	Strict validation		Relaxed validation	
Class	State		Minimum	Maximum	Minimum	Maximum
Chk Chk1 Chk2 Chk3 Chk4 Chk5 Chk6	All	1450 2050 1575 1450 1375 1350 1350	1275 1650 1400 1325 1300 1275 1275	2250 2250 1650 1500 1425 1400 1400	1255 1525 1350 1275 1250 1225 1225	2500 2500 1725 1550 1475 1450 1450
(All) Rock	All	2250	2100	2300	2050	2500
(All) River soil	Extr./v. low Low	1600 1650	1250 1400	1800 1800	1200 1200	2000 2000
Custom	-	2000	1200	2400	600	2500

The following table gives Re/x database values for wet (saturated) density (ρ_s).

Soil classification		Wet (saturated) density ρ_s (kg/m^3)				
		Default	Strict validation		Relaxed validation	
Class	State		Minimum	Maximum	Minimum	Maximum
(All) Gr	- V. loose Loose Med. dense Dense V. dense	2200 1850 2000 2100 2200 2250	1800 1700 1800 1900 2000 2200	2300 1900 2100 2200 2300 2400	1500 1500 1700 1800 1900 2000	2500 2100 2200 2300 2400 2500
(All) Sa	- V. loose Loose Med. dense Dense V. dense	2075 1900 1950 1975 2075 2175	1800 1750 1800 1850 1950 2050	2150 1975 2000 2050 2150 2250	1600 1600 1750 1800 1850 1950	2400 2000 2050 2150 2250 2400
(All) Si	All	2050	1800	2150	1500	2400
(All) Cl		Same as dry density				
Or siOr clOr Peat Loam	All	1650 1650 1650 1250 1900	1050 1500 1500 950 1650	2050 1750 1750 1400 2050	850 1400 1400 850 1450	2250 1950 1950 1500 2250
Mg rock-Mg slag-Mg grMg saMg chalk-Mg brick-Mg ash-Mg pfa-Mg	All	2000 2100 1850 2150 2050 1825 1850 1450 1750	1650 1900 1700 1800 1800 1750 1650 1300 1500	2150 2200 1900 2300 2150 1850 1950 1500 1800	1200 1750 1400 1500 1600 1700 1400 1200 1350	2500 2300 2000 2500 2400 1900 2100 1800 2000
clMg/siMg	All	1850	1500	2050	1300	2250
Chk Chk1 Chk2 Chk3 Chk4 Chk5 Chk6	All	1900 2300 1975 1900 1850 1825 1825	1750 2025 1850 1800 1775 1750 1750	2450 2450 2025 1925 1875 1850 1850	1725 1925 1800 1750 1750 1725 1725	2600 2600 2075 1950 1900 1900 1900
(All) Rock	All	Same as dry density				
(All) River soil	All	Same as dry density				

Soil classification		Wet (saturated) density ρ_s (kg/m^3)				
		Default	Strict validation		Relaxed validation	
Class	State		Minimum	Maximum	Minimum	Maximum
Custom	-	2000	1200	2400	850	2600

Drained strength

The following table gives Re/x database values for peak angle of shearing resistance (φ′) and effective cohesion (c′).

Soil classification		Peak angle of shearing resistance φ′ (°)/effective cohesion c′ (kPa)				
		Default	Strict validation		Relaxed validation	
Class	State		Minimum	Maximum	Minimum	Maximum
(All) Gr	-	37°/0	35°/0	50°/0	28°/0	60°/10
	V. loose	34°/0	32°/0	38°/0	28°/0	40°/10
	Loose	37°/0	35°/0	40°/0	30°/0	45°/10
	Med. dense	42°/0	40°/0	45°/0	35°/0	50°/10
	Dense	47°/0	45°/0	50°/0	40°/0	55°/10
	V. dense	52°/0	50°/0	55°/0	45°/0	60°/10
(All) Sa	-	32°/0	30°/0	40°/0	20°/0	55°/10
	V. loose	26°†/0	25°†/0	28°†/0	20°†/0	30°†/10
	Loose	32°/0	30°/0	35°/0	26°/0	40°/10
	Med. dense	34°/0	33°/0	37°/0	29°/0	45°/10
	Dense	37°/0	36°/0	40°/0	33°/0	50°/10
	V. dense	42°/0	40°/0	45°/0	37°/0	55°/10
CSi	-	28°/0	27°/0	33°/5	20°/0	45°/10
	V. loose	26°†/0	25°†/0	28°†/5	20°†/0	30°†/10
	Loose	28°/0	27°/0	31°/5	23°/0	35°/10
	Med. dense	29°/0	28°/0	32°/5	25°/0	37°/10
	Dense	30°/0	29°/0	33°/5	27°/0	40°/10
	V. dense	33°/0	32°/0	36°/5	30°/0	45°/10
Si	All	28°/0	25°/0	35°/5*	17°/0	45°/10*
saSi		28°/0	25°/0	35°/5*	17°/0	40°/10*
clSi		23°/0	20°/0	30°/5*	17°/0	35°/10*
Cl	All	20°/0	20°/0	33°/10*	15°/0	39°/15*
grCl/saCl		24°/2	20°/0	33°/10*	18°/0	39°/15*
siCl		27°/2	24°/0	33°/10*	20°/0	39°/15*
Lam		19°/2	16°/0	25°/10*	15°/0	39°/15*
Or	All	23°/0	20°/0	30°/0	18°/0	39°/0
siOr/clOr		23°/0	20°/0	30°/0	18°/0	37°/0
Peat		23°/0	20°/0	30°/0	18°/0	37°/0
Loam		27°/0	24°/0	33°/0	20°/0	39°/0
Mg	All	35°/0	30°/0	45°/0	23°/0	60°/0
rock-Mg		43°/0	40°/0	50°/0	35°/0	60°/0
slag-Mg		33°/0	30°/0	40°/0	25°/0	50°/0
grMg		40°/0	35°/0	50°/0	28°/0	60°/0
saMg		32°/0	30°/0	35°/0	23°/0	40°/0
chalk-Mg		32°/0	30°/0	37°/0	25°/0	43°/0
brick-Mg		42°/0	40°/0	45°/0	35°/0	50°/0
ash-Mg		37°/0	35°/0	40°/0	30°/0	45°/0
pfa-Mg		32°/0	30°/0	37°/0	27°/0	40°/0
clMg/siMg	All	21°/0	17°/0	30°/0	15°/0	35°/0
Chk	-	35°/0	30°/0	45°/20	25°/0	55°/100
Chk1		35°/10	30°/0	45°/20	25°/0	55°/100
Chk2		34°/5	30°/0	43°/20	25°/0	52°/50
Chk3		34°/5	30°/0	41°/20	25°/0	49°/50
Chk4		33°/2	30°/0	39°/10	25°/0	46°/20
Chk5		32°/0	30°/0	37°/0	25°/0	43°/0
Chk6		32°/0	30°/0	35°/0	25°/0	40°/0
(All) Rock	-	33°/5	30°/0	38°/10	27°/0	42°/20

Soil classification		Peak angle of shearing resistance φ′ (°)/effective cohesion c′ (kPa)				
		Default	Strict validation		Relaxed validation	
Class	State		Minimum	Maximum	Minimum	Maximum
(All) River soil	All	22°/0	16°/0	33°/0	15°/0	39°/0
Custom	-	30°/0	20°/0	50°/10	10°/0	60°/100

†φ′ reduced to allow for potential liquefaction
*c′ = 0kPa when state is set to extremely low, very low, or low strength

The following table gives Re/x database values for constant volume (i.e. critical state) angle of shearing resistance (φ'_{cv}) and effective cohesion (c'_{cv}).

Soil classification		Const. vol. angle of shearing resistance φ'_{cv} (°)/effective cohesion c'_{cv} (kPa)				
		Default	Strict validation		Relaxed validation	
Class	State		Minimum	Maximum	Minimum	Maximum
- Gr siGr/clGr (other) Gr	All	37°/0 37°/0 37°/0	35°/0 37°/0 37°/0	40°/0 40°/0 40°/0	28°/0 28°/0 28°/0	45°/5 45°/5 45°/0
- Sa siSa/clSa (other) Sa	All	32°/0 32°/0 32°/0	30°/0 30°/0 30°/0	35°/0 35°/0 35°/0	23°/0 23°/0 23°/0	40°/5 40°/5 40°/0
CSi	-	28°/0	27°/0	31°/0	20°/0	35°/5
Si saSi clSi	All	25°/0 25°/0 19°/0	22°/0 22°/0 18°/0	30°/0 30°/0 22°/0	17°/0 20°/0 17°/0	32°/5* 32°/5* 25°/5*
Cl grCl/saCl siCl Lam	All	23°/0 24°/0 23°/0 16°/0	20°/0 20°/0 20°/0 12°/0	33°/0 33°/0 28°/0 20°/0	8°/0 18°/0 18°/0 8°/0	39°/5* 39°/5* 30°/5* 22°/5*
Or siOr/clOr Peat Loam	All	23°/0 23°/0 23°/0 27°/0	20°/0 20°/0 20°/0 24°/0	30°/0 30°/0 30°/0 33°/0	18°/0 18°/0 18°/0 20°/0	39°/0 37°/0 37°/0 39°/0
Mg rock-Mg slag-Mg grMg saMg chalk-Mg brick-Mg ash-Mg pfa-Mg	All	32°/0 37°/0 32°/0 37°/0 32°/0 32°/0 32°/0 33°/0 32°/0	30°/0 35°/0 30°/0 35°/0 30°/0 30°/0 30°/0 30°/0 30°/0	35°/0 40°/0 35°/0 40°/0 35°/0 35°/0 35°/0 38°/0 35°/0	25°/0 30°/0 25°/0 28°/0 23°/0 25°/0 25°/0 27°/0 27°/0	45°/0 45°/0 45°/0 45°/0 40°/0 40°/0 40°/0 42°/0 40°/0
clMg/siMg	All	21°/0	17°/0	28°/0	15°/0	30°/0
(All) Chk	-	32°/0	30°/0	35°/0	25°/0	40°/5
(All) Rock	-	33°/0	30°/0	38°/0	27°/0	42°/5
(All) River soil	All	22°/0	16°/0	33°/0	15°/0	39°/0
Custom	-	25°/0	20°/0	35°/0	8°/0	45°/5

†φ′ reduced to allow for potential liquefaction
*c′ = 0kPa when state is set to extremely low, very low, or low strength

Undrained strength

The following table gives Re/x database values for undrained strength (c_u) and rate of increase in undrained strength with depth (Δc_u).

Soil classification		Undrained strength c_u (kPa)/increase with depth Δc_u (kPa/m)						
		Default	Strict validation		Relaxed validation	
Class	Stength		Minimum	Maximum	Minimum	Maximum
(All) Si	-	50/0	20/-10	150/8	1/-100	1000/100
(All) Cl	Extr. low	7/0	2/-10	10/8	1/-100	15/100
clMg/siMg	Very low	15/0	10/-10	20/8	7/-100	25/100
Custom	Low	25/0	20/-10	40/8	15/-100	55/100
cohesive	Medium	50/0	40/-10	75/8	25/-100	100/100
	High	100/0	75/-10	150/8	50/-100	200/100
	Very high	200/0	150/-10	300/8	100/-100	400/100
	Extr. high	375/0	300/-10	500/8	200/-100	1000/100
(All) River soil	Extr. low	7/0	5/-10	10/8	1/-100	15/100
	Very low	15/0	10/-10	20/8	7/-100	25/100
	Low	25/0	20/-10	40/8	15/-100	55/100

Drained and undrained stiffnesses

Soil stiffness may be specified for drained and, if appropriate, undrained conditions in terms of shear modulus (G), Young's modulus (E), and Poisson's ratio (ν), where:

$$E = 2G(1+\nu)$$

Values of G and E may be specified as increasing with depth, by entering values for the increase (dG or dE) and the distance over which that increase occurs (dz), The 'gradient' is then calculated as dG/dz and dE/dz.

Different values of G and E may be specified in the horizontal and vertical directions. These values are linked by the 'anistoropy' parameter, defined as:

$$anisotropy = G_h / G_v = E_h / E_v$$

The anisotropy parameter for soils is limited in value between 0 and 2.

Large strain stiffness values should be smaller than small strain values.

Concretes

Mass/weight densities

According to EN 206-1 [54], normal weight concrete has weight density between 2000 and 2600 kg/m^3.

Strength

The compressive strength of concrete measured in a cylinder test is approximately 80% of the concrete's strength when measured in a cube test.

For concrete grades C8/10, C12/15, C16/20, C20/25, C25/30, C30/37, C35/45, C40/50, C45/55, and C50/60, the first number signifies the concrete's cylinder strength and the second number its cube strength (both in MPa) in accordance with Eurocode 2.

For concrete grades C25, C30, C35, C40, C45, and C50, the number signifies the concrete's cube strength (in MPa) in accordance with BS 8110.

According to Arya [55], the strength of concrete varies from 12 to 60 MPa.

Stiffness

According to EN 1992-1-1 [56], the Young's modulus of elasticity for concrete is between 27 and 44 GPa. Fleming [57] quotes values between 5 and 40 GPa for foundation concrete.

Different values of Young's modulus may be specified in the horizontal and vertical directions. These values are linked by the 'anistoropy' parameter, defined as:

$$anisotropy = E_h / E_v$$

The anisotropy parameter for concrete is limited in value between 0 and 1.

Steels

Mass/weight densities

According to EN 1993-1-1 [58] §3.2.6, structural steel has a weight density of 7850 kg/m^3.

Strength

For structural steel grades S235, S275, S355, and S450, the number signifies the steel's yield strength.

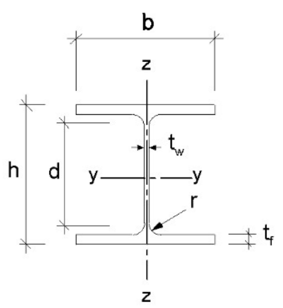
For Corus's Advance range of steels (Advance 275 and Advance 355), the number also signifies the steel's yield strength.

Stiffness

According to EN 1993-1-1 [59] §3.2.6, the Young's modulus of elasticity for structural steel is 210 GPa and its Poisson's ratio is 0.3.

Bearing piles

The properties of Corus's UKBP range of bearing piles are provided in the folder `[R]\Sections\Bearing Piles`, each in a separate XML file (e.g. `UKBP 203x203x45.xml`).



The figure (right) shows the key dimensions of an I-section, with notation taken from EN 1993-1-1:

- Width (b)
- Depth (h)
- Web thickness (t_w)
- Flange thickness (t_f)
- Depth between fillets (d)
- Root radius (r)

The section's strong (y-y) and weak (z-z) axes are also shown. The x-x axis runs along the length of the bearing pile (perpendicular to the plane of the paper).

Circular section

The section area (A) of a circular section is calculated from its diameter (D) as follows:

$$A = \frac{\pi D^2}{4}$$

Custom section

The custom section allows you to enter the following custom section properties:

- Circumference
- Section area (A)
- Polar moment of area (J)
- Separately about strong (y-y) and weak (z-z) axes:
    - Depth (h)
    - 1st moment of area (Q)
    - 2nd moment of area (I)

Rectangular section

The section area (A) of a rectangular section is calculated from its breadth (B) and depth (D) as follows:

$$A = B \times D$$

Chapter 6
Algorithms

Algorithms allow you to change the way calculations are performed. Repute® implements the following algorithms:

- Alpha algorithm
- Bearing capacity algorithm
- Bearing pressure limit
- Beta algorithm
- Lateral earth pressure coefficient
- Plugging algorithm
- No contact algorithm
- Skin friction limit
- Wall friction algorithm

Alpha algorithm

The alpha algorithm determines shaft friction (f_s) along the pile in undrained soil horizons, as a proportion of the soil's undrained strength (c_u):

$$f_s = \alpha \times c_u$$

The options for determining α are summarized below.

*Algorithm (*default)*	*Equation*
Custom alpha	α = any value > 0 and ≤ 1
Skempton's alpha [60]	α = 0.45
Alpha = 0.5*	α = 0.5
Alpha for London Clay	α = 0.6
Randolph & Murphy's alpha [61]	$c_u/\sigma'_v \geq 1: \alpha = 0.5 \times (c_u/\sigma'_v)^{-0.75}$ $c_u/\sigma'_v < 1: \alpha = 0.5 \times (c_u/\sigma'_v)^{-0.5} \leq 1.0$
Semple & Rigden's alpha [62]	$\alpha = (0.5 \leq \alpha_1 \leq 1) \times (0.7 \leq \alpha_2 \leq 1)$ $\log(\alpha_1) = \log(0.5) \times \frac{\log(c_u/\sigma'_v) - \log(0.35)}{\log(0.8) - \log(0.35)}$ $\log(\alpha_2) = \log(0.7) \times \frac{\log(L/D) - \log(50)}{\log(120) - \log(50)}$
Bowles' alpha [63]	$c_u < 75kPa: \alpha = 1.1 - 0.3 \times (c_u/75kPa)$ $75 \leq c_u < 200kPa: \alpha = 0.98 - 0.3 \times (c_u/125kPa)$ $c_u \geq 200kPa: \alpha = 0.5$
Sladen's alpha [64]	$\alpha = C_1 \times (c_u/\sigma'_v)^{-0.45}$ C_1 = 0.4-0.5 for bored piles; C_1 > 0.5 for driven piles (C_1 = 0.5 assumed)

*Algorithm (*default)*	*Equation*
O'Neill & Reese's alpha [65]	$c_u / p_a \leq 1.5 : \alpha = 0.55$ $1.5 < c_u / p_a \leq 2.5 : \alpha = 0.55 - 0.1\left[\left(c_u / p_a\right) - 1.5\right]$ $c_u / p_a > 2.5 : \alpha = 0.45$
US Army Corps of Engineers' alpha [66]	$0.5 \leq \alpha = 1.0 - 0.5 \times \frac{c_u - t_1}{t_2 - t_1} \leq 1.0$ $t_1 = 0.25 tsf\,(US) \approx 24 kPa$ $t_2 = 0.75 tsf\,(US) \approx 72 kPa$
Key: c_u = undrained shear strength; σ'_v = vertical effective stress; L = pile length; D = pile diameter; p_a = atmospheric pressure (= 100 kPa)	

Bearing capacity algorithm

The bearing capacity algorithm determines the earth pressure coefficients (N_q, N_γ, and N_c) that are used to calculate the base resistance of the pile in soil. The coefficients are mainly related to the soil's angle of shearing resistance (φ):

$$N_q = e^{\pi \times \tan\varphi} \times \tan^2\left(\pi/4 + \varphi/2\right)$$

$$N_\gamma = varies$$

$$N_c = \left(N_q - 1\right)\cot\varphi$$

The options available for determining N_q, N_γ, and N_c are summarized below (where not stated explicitly, the equations for N_q and N_c are as given above).

*Algorithm (*default)*	*Equation*
Custom	N_q = any value ≥ 1 and ≤ 318 N_γ = any value ≥ 0 and ≤ 1000 N_c = any value ≥ π +2 and ≤ 266
Terzaghi [67]	$N_q = 0.5 \times e^{(1.5\pi - \varphi)\times\tan\varphi} \times \sec^2\left(\pi/4 + \varphi/2\right) \geq \pi + 2$ $N_\gamma = 0.5 \times \left(K_{p\gamma} \sec^2\varphi - 1\right) \times \tan\varphi$ Terzaghi obtained $K_{p\gamma}$ by a graphical technique; Repute uses numerical values given by Kumhojkar (1993)
Meyerhof [68]	$N_\gamma = \left(N_q - 1\right) \times \tan\left(1.4\varphi\right)$
Brinch-Hansen [69]	$N_\gamma = 1.5\left(N_q - 1\right) \times \tan\varphi$
Vesic [70]	$N_\gamma = 2\left(N_q + 1\right) \times \tan\varphi$
Berezantzev [71]	$N_q = B_k \times \alpha_t$ $B_k = 0.21 \times e^{30.6\varphi/\pi}$ $\alpha_t = f\left(\varphi, L/D\right)$
Spangler and Handy [72]	$N_\gamma = 1.1\left(N_q - 1\right) \times \tan\left(1.3\varphi\right)$
API RP2A [73]	same as Vesic

*Algorithm (*default)*	*Equation*
Eurocode 7 [74]*	$N_\gamma = 2\left(N_q - 1\right) \times \tan\varphi$
Zhu et al. [75]	$N_\gamma = 2\left(N_q + 1\right) \times \left(\tan\varphi\right)^{1.45}$, based on Case 3 (minimum N_γ)
Key: φ = soil's angle of shearing resistance; L = pile length; D = pile diameter	

Bearing pressure limit

The bearing pressure limit determines the maximum unit bearing resistance ($q_{b,max}$) that is available at the pile toe. The options available for determining $q_{b,max}$ are summarized below.

*Algorithm (*default)*	*Equation*
Custom limit	$q_{b,max}$ = any value > 0 MPa
No limit*	$q_{b,max} = \infty$
North Sea limit	$q_{b,max}$ = 15 MPa
API limit	$q_{b,max}$ = 100 kipf/ft^2 ≈ 4.79 MPa

Beta algorithm

The beta algorithm determines shaft friction (f_s) along the pile in drained soil horizons, as a proportion of the vertical effective stress (σ'_v):

$$f_s = \beta \times \sigma'_v$$

The options available for determining β are summarized below.

*Algorithm (*default)*	*Equation*
Custom beta	β = any value ≥ 0.1 and ≤ 3
O'Neill and Reese (for sand) [76]*	$0.25 \leq \beta = 1.5 - 0.245\sqrt{z} \leq 1.2$
Rollins et al. (for gravel) [77]	$\beta = 3.4e^{-0.085z}$
Rollins et al. (for gravelly sand) [78]	$0.25 \leq \beta = 2.0 - 0.15z^{0.75} \leq 1.8$
Bhushan (for sand) [79]	$\beta = 0.18 + 0.65I_D$
Key: z = depth below ground surface; I_D = soil's density index (relative density)	

Earth pressure coefficient

The earth pressure coefficient (K_s) determines the horizontal effective stress (σ'_h) along the pile shaft in drained soil horizons, as a proportion of the vertical effective stress (σ'_v):

$$\sigma'_h = K_s \times \sigma'_v$$

Different values of K_s are used for piles in compression and in tension.

The horizontal effective stress is then used, in conjunction with the wall friction algorithm, to determine the skin friction (f_s) along the pile shaft:

$$f_s = \sigma_h' \times \tan\delta = K_s \times \sigma_v' \times \tan\delta$$

The options for determining K_s are summarized below.

*Algorithm (*default)*	*Equation*
Custom coefficient	K_s = any value ≥ 0.5 and ≤ 4.5
API coefficient [80]*	K_s = 1.0 when the pile is in compression; 0.8 in tension
North Sea coefficient	K_s = 0.7 when the pile is in compression; 0.5 in tension

No contact algorithm

The no contact algorithm determines the depth (d_s) above which shaft resistance is ignored, owing, for example, to shrinkage of fine soil or socket-holing of coarse soil. The no contact depth d_s is normally related to the soil's plasticity index (I_p). The options for determining d_s are summarized below.

*Algorithm (*default)*	*Equation*
Custom depth	d_s = any value > 0 m and ≤ 12 m
NHBC (1992) [81]*	$I_p > 0.4$: d_s = 1.0 m $0.2 < I_p \le 0.4$: d_s = 0.9 m $I_p \le 0.2$: d_s = 0.75 m
Key: I_p = soil's plasticity index	

Plugging algorithm

The plugging algorithm determines the proportion (ρ) of the pile's gross base area that is considered to offer base resistance. The options for determining K_s are summarized below.

*Algorithm (*default)*	*Equation*
Custom plug	ρ = any value between 0 and 100 %
No plug	ρ = 0 %
Half plug	ρ = 50 %
Full plug*	ρ = 100 %

Rock bearing algorithm

The rock bearing algorithm determines the unit bearing resistance (q_b) of a pile founded in rock, based on the following formula, given in BS 8005:2015:

$$q_b = k_3 p_{ref} \left(\frac{q_u}{p_{ref}} \right)^{k_4}$$

The options available for determining k_3, k_4, and p_{ref} are summarized below.

*Algorithm (*default)*	*Constant k_3*	*Power k_4*	*Reference pressure p_{ref}*
Custom rock bearing	≥ 1 and ≤ 15 (default 15)	≥ 0.5 and ≤ 1 (default 0.5)	100 kPa (0.1 MPa)
Poulos and Davis	1	1	1 MPa
Rowe and Armitage	3	1	1 MPa

*Algorithm (*default)*	*Constant k_3*	*Power k_4*	*Reference pressure p_{ref}*
Piling Engineering	10	1	100 kPa (0.1 MPa)
Tomlinson	2.5	1	1 MPa
Zhang and Einstein	15	0.5	100 kPa (0.1 MPa)
Poulos*	2.5	1	1 MPa

Rock friction algorithm

The rock friction algorithm determines the unit shaft resistance (q_s) of a pile founded in rock, based on the following formula, given in BS 8005:2015:

$$q_s = k_1 p_{ref} \left(\frac{q_u}{p_{ref}} \right)^{k_2} = \alpha_q \left(q_u \right)^{\beta}$$

The options available for determining k_1, k_2, and p_{ref} are summarized below.

*Algorithm (*default)*	*Constant k_1*	*Power k_2 (= β)*	*α_q*	*Reference pressure p_{ref}*
Custom rock friction	≥ 0.15 and ≤ 2.1 (default 0.79)	≥ 0.36 and ≤ 1 (default 0.5)	0.03-2.1 (0.25)	100 kPa (0.1 MPa)
Rosenberg and Journeaux	1.05	0.51	0.34	1 MPa
Horvath	1.04	0.5	0.33	1 MPa
Horvarth and Kenney	0.66	0.5	0.21	10 kPa (0.01 MPa)
Meigh and Wolski	0.55	0.6	0.22	1 MPa
Reynolds and Kaderbeck	0.3	1	0.3	1 MPa
Gupton and Logan	0.2	1	0.2	1 MPa
Rowe and Armitage	1.08	0.57	0.4	1 MPa
Carter and Kulhawy	0.63	0.5	0.2	1 MPa
Toh et al.	0.25	1	0.25	1 MPa
Piling Engineering	1.3	0.5	0.41	100 kPa (0.1 MPa)
Kulhawy and Phoon (lower)	0.71	0.5	0.22	100 kPa (0.1 MPa)
Kulhawy and Phoon (mean)	1.41	0.5	0.45	100 kPa (0.1 MPa)
Kulhawy and Phoon (upper)	2.12	0.5	0.67	100 kPa (0.1 MPa)
Reese and O'Neill	0.15	1	0.15	1 MPa
Poulos and Bunce*	0.79	0.5	0.25	1 MPa

Rock pressure coefficient

The rock pressure algorithm determines the unit transverse resistance (q_{tr}) of a pile founded in rock, based on the following formula:

$$q_{tr} = \eta q_b = k_5 \left(1 + z / D \right) q_b \le q_b$$

where η is a factor that increases linearly with depth (z) from ground surface, D is the pile diameter, and k_5 is a constant.

The options available for determining k_5 are summarized below.

*Algorithm (*default)*	*Constant k_5*
Custom rock pressure coefficient	any value $\geq$ 0.2 and $\leq$ 0.3 (default 0.22)
Poulos*	0.22

Skin friction limit

The skin friction pressure limit determines the maximum unit shaft resistance ($q_{s,max}$) that is available along the pile shaft. The options available for determining $q_{s,max}$ are summarized below.

*Algorithm (*default)*	*Equation*
Custom limit	$q_{s,max}$ = any value > 0 kPa
No limit*	$q_{s,max} = \infty$
North Sea limit	$q_{s,max}$ = 100 kPa
API limit	$q_{s,max}$ = 1700 lbf/ft^2 $\approx$ 81.4 kPa

Wall friction algorithm

The wall friction algorithm determines the skin friction (f_s) along the pile shaft, as a proportion of the horizontal effective stress (σ'_h):

$$f_s = \sigma'_h \times \tan\delta$$

The wall friction δ is often calculated as a proportion of the soil's angle of shearing resistance (φ).

The horizontal effective stress is obtained from the lateral earth pressure coefficient (K_s) and depends the vertical effective stress (σ'_v):

$$\sigma'_h = K_s \times \sigma'_v \Rightarrow f_s = K_s \times \sigma'_v \times \tan\delta$$

The options for determining δ are summarized below.

*Algorithm (*default)*	*Equation*
Custom friction	δ = any value > 0° and ≤ 35°
No friction*	$\delta = 0°$
One-third friction	$\delta = \varphi/3$
One-half friction	$\delta = \varphi/2$
Two-thirds friction	$\delta = 2\varphi/3$
Three-quarters friction	$\delta = 3\varphi/4$
Full friction	$\delta = \varphi$
Five degrees less than the angle of shearing	$\delta = \varphi - 5°$
Key: φ = soil's angle of shearing resistance	

Chapter 7
References

The following pages list the papers referred to throughout the main text of this manual.

[1] Butterfield, R. and Banerjee, P. K. (1971), “The elastic analysis of compressible piles and pile groups”, *Géotechnique* 21, No. 1, 43-60.

[2] Basile, F. (1999), “Non-linear analysis of pile groups”, *Proc Instn Civ Engng, Geotech Engng* 137, No. 2, April, 105-115.

[3] Basile, F. (2003), “Analysis and design of pile groups”, in *Numerical Analysis and Modelling in Geomechanics* (ed. J. W. Bull), Spon Press (Taylor & Francis Group Ltd), Oxford, Chapter 10, 278-315.

[4] Basile, F. (2010), “Torsional response of pile groups”, *Proc. 11th DFI/EFFC Int. Conf. on Geotechnical Challenges in Urban Regeneration*, London, 26-28 May 2010, 13pp.

[5] Mindlin, R. D. (1936), “Force at a point in the interior of a semi-infinite solid”, *Physics* 7, 195-202.

[6] Poulos, H. G. (1979), “Settlement of single piles in nonhomogeneous soil”, *J. Geotech. Engng, Am. Soc. Civ. Engrs* 105, No. GT5, 627-641.

[7] Poulos, H. G. (1990), *User's guide to program DEFPIG Deformation Analysis of Pile Groups*, Revision 6, School of Civil Engineering, University of Sydney.

[8] Leung, C. F. & Chow, Y. K. (1987), “Response of pile groups subjected to lateral loads”, *Int. J. Numer. Anal. Meth. Geomechs* 11, No. 3, 307-314.

[9] Chow, Y. K. (1986), “Analysis of vertically loaded pile groups”, *Int. J. Numer. Anal. Meth. Geomechs* 10, No. 1, 59-72.

[10] Chow, Y. K. (1987a), “Three-dimensional analysis of pile groups”, *J. Geotech. Engng, Am. Soc. Civ. Engrs* 113, No. 6, 637-651.

[11] Chow, Y. K. (1987), “Axial and lateral response of pile groups embedded in nonhomogeneous soils”, *Int. J. Numer. Anal. Meth. Geomechs* 11, 621-638.

[12] Poulos, H. G. (1989), “Pile behaviour - theory and application”, 29th Rankine Lecture, *Géotechnique* 39, No. 3, 365-415.

[13] Yamashita, K., Tomono, M. & Kakurai, M. (1987), “A method for estimating immediate settlement of piles and pile groups”, *Soils and Fdns* 27, No. 1, 61-76.

[14] Steinbrenner, W. (1934), *Tafeln zur Setzungberechnung. Strasse* 1, 221.

[15] Poulos, H. G. and Davis, E. H. (1980), *Pile foundation analysis and design*, New York: Wiley.

[16] Randolph, M. F. (1987), *PIGLET, a computer program for the analysis and design of pile groups*.

[17] Broms, B. B. (1964), “Lateral resistance of piles in cohesive soils”, *J. Soil Mechs Fdn Division, Am. Soc. Civ. Engrs* 90, No. SM2, 27-63.

[18] Fleming, W. G. K., Weltman, A. J., Randolph, M. F., and Elson, W. K. (1992), *Piling Engineering* (2nd edn), Glasgow: Blackie Academic and Professional.

[19] Berezantzev, V. G., Khristoforov, V., and Golubkov, V. (1961), “Load bearing capacity and deformation of piled foundations”, *Proc. 5th Int. Conf. Soil Mech. Fdn Engng*, Paris 2, 11-15.

[20] Poulos, H.G., and Bunce, G. (2008), "Foundation design for the Burj Dubai - The World's Tallest Building", *Proc. 6th Int. Conf. on Case Histories in Geotechnical Engineering and Symposium in Honor of Professor James K. Mitchell*, Arlington, VA (USA), Paper No. 1.47, pp. 1-16.

[21] Poulos, H.G. (2016), "Tall building foundations: design methods and applications", *Innov. Infrastruct. Solut.* 1: 10. doi:10.1007/s41062-016-0010-2, 51p.

[22] Cox, W. R., Dixon, D. A., and Murphy, B. S. (1984), “Lateral load tests on 25.4-mm (1-in.) diameter piles in very soft clay in side-by-side and in-line groups”, *Laterally loaded deep foundations: analysis and performance*, ASTM STP 835, J. A. Langer, E. T. Mosley, and C. D. Thompson (ed), American Society for Testing and Materials, 122-139.

[23] Brown, D. A. and Shie, C. F. (1990). “Numerical experiments into group effects on the response of piles to lateral loading”, *Computers and Geotechnics* 10, No. 4, 211-230.

[24] Ng, C. W. W., Zhang, L., and Nip, D. C. N. (2001), “Response of laterally loaded large-diameter bored pile groups”, *J. Geotech. and Geoenv. Engng, Am. Soc. Civ. Engrs* 127, No. 8, 658-669.

[25] Reese, L. C., Wang, S. T., Arrellaga, J. A., and Hendrix, J. (1996). *Computer program GROUP for Windows user's manual*, version 4.0. Ensoft, Inc., Austin, Texas.

[26] Duncan, J. M. and Chang, C. Y. (1970), “Non-linear analysis of stress and strain in soils”, *J. Soil Mechs Fdn Division, Am. Soc. Civ. Engrs* 96, No. SM5, 1629-1681.

[27] Randolph, M. F. (1994), “Design methods for pile groups and piled rafts”, *Proc. 13th Int. Conf. Soil Mech. Fdn Engng*, New Delhi 5, 61-82.

[28] Poulos, H. G. (1994), “Settlement prediction for driven piles and pile groups”, *Proc. Conf. on vertical and horizontal deformations of foundations and embankments*, College Station, Texas, Geotechnical Special Publication, Vol. 2, No. 40, 1629-1649.

[29] Fleming’s paper *A new method for single pile settlement prediction and analysis* (1992), Géotechnique, vol. 42, no. 3, pp 411-425.

[30] Fleming, Weltman, Randolph, and Elson (1992), *Piling engineering* (2nd edition), Glasgow: Blackie Academic and Professional, pp. 122-140

[31] BS 8004: 1986, Code of practice for foundations, British Standards Institution, London.

[32] ENV 1997-1: 1994 (pre-standard), Eurocode 7: Geotechnical design – Part 1: General rules, European Committee for Standardization, Brussels.

[33] EN 1997-1: 2004, Eurocode 7: Geotechnical design – Part 1: General rules, European Committee for Standardization, Brussels.

[34] UK National Annex to BS EN 1997-1: 2007, British Standards Institution, London.

[35] Irish National Annex to IS EN 1997-1: 2007, National Standards Authority of Ireland, Dublin.

[36] Norme tecniche per le costruzioni, DM 14/01/2008, Il Ministro delle Infrastrutture.

[37] Singapore National Annex to SS EN 1997-1: 2010, SPRING, Singapore.

[38] BS 8004: 1986, Code of practice for foundations, British Standards Institution, London.

[39] EN ISO 14688, Geotechnical investigation and testing — Identification and classification of soil, European Committee for Standardization, Brussels.

[40] EN ISO 14689, Geotechnical investigation and testing — Identification and classification of rock, European Committee for Standardization, Brussels.

[41] Terzaghi, K. and Peck, R. B. (1967), Soil mechanics in engineering practice (2nd edition), John Wiley & Sons, Inc, 729pp.

[42] NAVFAC DM-7 (1971), Naval Facilities Engineering Command,Virginia, USA.

[43] Winterkorn H.F. and Fang F.Y. (1975), Foundation Engineering Handbook, Springer.

[44] Canadian Foundation Engineering Manual (1978), Canadian Geotechnical Society.

[45] Reynolds C.E. and Steedman J. (1981), Reinforced Concrete Designers' Manual.

[46] Bell F.G. (1983), *Engineering properties of soils and rocks* (2nd edition), Butterworths, London, 149pp.

[47] Mitchell R.J. (1983), *Earth structures engineering*, Allen & Unwin Inc., Boston.

[48] TradeARBED's *Spundwand-Handbuch Teil 1, Grundlagen* (1986)

[49] Bolton, M. D. (1986) A guide to soil mechanics.

[50] Clayton C.R.I. and Militiski J. (1986), *Earth pressure and earth-retaining structures*, Surrey University Press, Glasgow and London, 300pp.

[51] Clayton C.R.I. (1989), *The mechanical properties of the Chalk*, Proc. Int. Chalk Symp., Brighton.

[52] Tomlinson M.J. (1995), *Foundation design and construction* (6th edition), Longman Scientific & Technical, Harlow, Essex, 536pp.

[53] British Steel's *Piling Handbook* (1997)

[54] EN 206-1, Concrete. Specification, performance, production and conformity, European Committee for Standardization, Brussels.

[55] Arya C. (1994), *Design of structural elements*, E &FN Spon.

[56] EN 1992-1-1, Eurocode 2: Design of concrete structures – Part 1-1 General rules and rules for buildings, European Committee for Standardization, Brussels.

[57] Fleming, Weltman, Randolph, and Elson (1992) *Piling engineering* (2nd edition), Glasgow: Blackie Academic and Professional, 390pp.

[58] EN 1993-1-1, Eurocode 3: Design of steel structures – Part 1-1 General rules and rules for buildings, European Committee for Standardization, Brussels.

[59] EN 1993-1-1 (loc. cit.).

[60] Skempton A.W. (1959), "Cast in-situ bored piles in London Clay", *Géotechnique*, Vol. 9, pp153-173.

[61] Randolph, M.F., and Murphy, B.J. (1985), "Shaft capacity of driven piles in clay", *Proc. 17th Offshore Technology Conf.*, Houston, Texas, Vol. 1, pp371-378 (Paper OTC 4883).

[62] Semple, R.M., and Rigden, W.J. (1984), "Shaft capacity of driven pile in clay", *Analysis and design of pile foundations*, ASCE, October, pp59-79.

[63] Bowles, J.E. (1997), *Foundation analysis and design* (5th edition), New York: McGraw Hill, 1175pp.

[64] Sladen J.A. (1992), "The adhesion factor: applications and limitations", *Can. Geotech. J.* Vol. 29, pp322-326.

[65] O'Neill, M.W, and Reese, L.C. (1999), *Drilled shafts: construction procedures and design methods*, Report No. FHWA-IF-99-025, Washington: US Dept of Transportation, Federal Highway Administration, 790pp.

[66] US Army Corps of Engineers (1991), *Design of pile foundations*, Engineer Manual No 1100-2-2906, Honolulu: University Press of the Pacific, c171pp.

[67] Terzaghi, K. (1943), Theoretical soil mechanics, New York: Wiley.

[68] Meyerhof, G. G. (1963) 'Some recent research on the bearing capacity of foundations', Can. Geotech. J., 1(1), pp. 16–26.

[69] Brinch-Hansen, J. (1970) A revised and extended formula for bearing capacity, Danish Geotechnical Institute, Bulletin No. 28, 6pp.

[70] Vesic, A. S. (1973) ‘Analysis of ultimate loads of shallow foundations’, J. Soil Mech. Found. Div., Am. Soc. Civ. Engrs, 99(1), pp. 45–73.

[71] Berezantsev V.G., Kristoforov V., and Golubkov V. (1961), “Load bearing capacity and deformation of pile foundations”, *Proc. 5th Int. Conf. Soil Mech. And Fdn Engng*, Vol 2, pp11-15.

[72] Spangler M.G. and Handy R.L. (1982), Soil Engineering, HarperCollins.

[73] API RP2A, Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Working Stress Design.

[74] EN 1997-1, (loc. cit.).

[75] Zhu, F., Clark, J. I., and Phillips, R. (2001), “Scale effect of strip and circular Footings resting on a dense sand”, *J. Geotech. Geoenviron. Eng.*, 127(7), 613–621.

[76] O’Neill M. and Reese L.C. (1999), *Drilled shafts: Construction procedures and design methods*, Federal Highways Administration.

[77] Rollins K.M., Clayton R.J., and Mikesell R.C. (1997), “Ultimate side friction of drilled shafts in gravels”, *Foundation Drilling*, Vol. 36, No 5, Association of Drilled Shaft Engineers, Dallas, Texas, USA.

[78] Rollins et al. (loc. cit.).

[79] Bhushan K. (1982), Discussion of “New design correlations for piles in sand”, *J. Geotec. Engng. Div.,* ASCE, Vol. 108, GT11, pp1500-1501.

[80] API RP2A, (loc. cit.).

[81] NHBC (1992), National House-Building Council Standards: Vol 1.