

AXIAL PILE ANALYSIS · CAPACITY AND T-Z
METHOD

Sol-load

Technical Reference

Capacity methods, load-transfer curves, numerical
solution and verification.

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1 Scope and notation

1.1 Purpose

This reference describes the engineering model and the numerical methods implemented in Sol-load 1.0.0: the idealisation of the pile and the soil, the static methods for axial capacity, the load-transfer (t-z and Q-w) curves, the numerical solution of the axially loaded pile, the interpretation of cone penetration tests, and the verification of the implementation. It is intended to let an engineer judge whether the program is appropriate for a problem and to check its results independently. The use of the program is described in the User Manual (SPL-SL-002).

The capacity methods and the load-transfer curves are empirical relations published in the technical literature and in design guidance. For each of them this reference gives the equations as they are evaluated by Sol-load and the publication on which they are based. Those publications describe the load tests behind each method, its range of validity and its limitations; users should consult them when a method is applied near or beyond the conditions for which it was developed. Different methods can give markedly different capacities for the same pile; the choice of method, of the soil parameters and of the factors of safety or resistance factors remains the responsibility of the engineer.

1.2 Notation

Symbols are defined where they first appear. The principal symbols are listed below. Equations are written in consistent units (kN, m, kPa unless stated otherwise); where a correlation is dimensional, the units are stated with the equation.

Table 1.1 — Principal symbols

Symbol	Meaning	Symbol	Meaning
A, A_b	material area; bearing area at the tip	q_b	unit tip resistance
B, D	pile width, diameter	q_c, q_t	cone resistance, corrected cone resistance
E	Young's modulus of the pile	Q_s, Q_b	shaft and tip capacity
f_s	unit shaft resistance	s_u	undrained shear strength
K, K_0	lateral earth-pressure coefficient	t, t_{max}	mobilised and maximum unit shaft resistance
L	pile length	w, z	tip movement; relative pile–soil movement on the shaft
N_{60}	SPT blow count at 60 % energy	x	distance along the pile from the head
N_q	bearing capacity factor	α	adhesion factor
P	axial force in the pile (compression +)	δ	interface friction angle
p_a	atmospheric pressure, 100 kPa	σ'_v	vertical effective stress
Q, Q_{max}	mobilised and maximum tip force	φ	effective friction angle

1.3 Sign convention and units

Depth z is measured vertically downwards from the ground surface; the distance x is measured along the pile axis from the pile head. Axial force, pile movement and shaft stress are positive in compression and for downward movement. In an uplift analysis the program reports the same quantities with negative signs. Results can be shown in SI units (kN, m, kPa, with diameters and movements in mm) or in US customary units (kip, ft, ksf, with diameters and movements in inches); the analysis itself is carried out in SI units.

2 Pile and soil model

2.1 The pile

The pile is a straight member of total length L , made of sections stacked from the head to the tip. Above the ground surface it may have a free length; it may be inclined from the vertical by a batter angle θ , in which case a distance x along the pile corresponds to the depth $z = (x - L_p) \cos \theta$. Each section has a shape, a material, an axial stiffness and a unit weight:

- **Circular and pipe** sections are defined by the outside diameter and, for a pipe, the wall thickness; area, perimeter and bearing area follow from the dimensions. A circular or pipe section may taper linearly to a smaller diameter at its bottom.
- **Rectangular** sections are defined by width and depth.
- **H and user-defined** sections are defined by their material area, contact perimeter and bearing area.

The axial stiffness EA may vary linearly within a section when a modulus at the bottom is given. The pile tip is closed, or open with a plugged, unplugged or automatically determined plug condition (Section 3.8).

2.2 The soil

The soil profile consists of horizontal layers, each assigned to a soil material of one of the families clay, sand, silt, rock or user-defined. Any parameter of a material can be given a second value at the bottom of the layer; the program then interpolates linearly through the thickness of each layer that uses the material. The undrained shear strength of a clay has its own top and bottom values.

The vertical effective stress follows from the total unit weight γ above the water table and the saturated unit weight γ_{sat} below it:

$$\sigma'_v(z) = \int_0^z \gamma'(\zeta) d\zeta, \quad \gamma' = \gamma \quad (\zeta < z_w), \quad \gamma' = \gamma_{sat} - \gamma_w \quad (\zeta \geq z_w) \quad (2.1)$$

with $\gamma_w = 9.807 \text{ kN/m}^3$ and z_w the depth of the water table. Without a water table, $\gamma' = \gamma$ throughout.

2.3 Friction angle from SPT

When a cohesionless material has no friction angle but an SPT blow count N_{60} , the friction angle is derived at the effective stress of each point as the smaller of two published correlations: a linear relation of the AASHTO type, $\varphi = \varphi_{AASHTO}(N_{60}) - 2.5 \sigma'_v / (20 \text{ psi})$, with $\varphi_{AASHTO} = 30^\circ, 32^\circ, 35^\circ, 40^\circ$ and 43° at $N_{60} = 3, 4, 10, 30$ and 60 , and the relation of Gibbs and Holtz between blow count, effective overburden and friction angle, interpolated between the charts for 0, 20 and 40 psi effective overburden. The overburden is limited to 40 psi (276 kPa). The derived angle is used by all capacity methods that need a friction angle; an explicit friction angle always takes precedence.

2.4 Embedded length and tip averaging

The capacity is evaluated as a function of the penetration: each row of a capacity profile represents the same pile with its tip at that depth, so that the profile shows how the capacity develops while the pile is driven or the shaft is installed. The interval between rows is the capacity step (by default the pile length divided by the number of increments of the solver). The shaft capacity is the sum over these intervals of the unit shaft resistance times the contact area; the methods that integrate a continuous law (USACE, FHWA, Revised Lambda) split the integration at layer boundaries, section changes, the water table and the depths where a limiting value is reached. A finer capacity step resolves thin layers more closely.

The unit tip resistance can be averaged over a zone around the tip ($\pm 1.5D$ or $\pm 3D$). The soil profile must then extend at least to the bottom of the averaging zone.

3 Axial capacity

3.1 General form

The nominal axial capacity of the pile with its tip at the embedded length considered is the sum of the shaft and tip capacities, corrected for the weight of the pile when that option is selected:

$$Q_n = Q_s + Q_b - W' \quad (\text{compression}), \quad Q_n = \eta_u Q_s + W' \quad (\text{uplift}) \quad (3.1)$$

$$Q_s = \sum_i f_{s,i} C_i \Delta x_i, \quad Q_b = q_b A_b \quad (3.2)$$

where $f_{s,i}$ is the unit shaft resistance of interval i , C_i its contact perimeter and Δx_i its length along the pile, q_b the unit tip resistance, A_b the bearing area, W' the effective weight of the pile (its weight less the buoyancy of the displaced volume below the water table) and η_u the uplift shaft factor entered by the user (default 0.8; not applied to the CPT-based methods, whose tension relations already reduce the shaft resistance, nor to user-defined resistances). In uplift there is no tip resistance. The buoyancy acts on the gross section for a closed-end pipe and on the material area otherwise.

The shaft resistance can be ignored above a chosen depth (for example through soft or scoured soil). Each material can cap the unit shaft and tip resistance with limiting values; a positive limit replaces the limits built into a method.

3.2 API method

The method of API RP 2GEO (the former RP 2A) for driven piles is applied as follows.

Clay. The unit shaft resistance is $f_s = \alpha s_u$ with

$$\alpha = 0.5 \psi^{-0.5} (\psi \leq 1), \quad \alpha = 0.5 \psi^{-0.25} (\psi > 1), \quad \alpha \leq 1, \quad \psi = s_u / \sigma'_v \quad (3.3)$$

and the unit tip resistance is $q_b = 9 s_u$.

Sand. The unit resistances are

$$f_s = K \sigma'_v \tan \delta \leq f_{lim}, \quad q_b = N_q \sigma'_v \leq q_{lim}, \quad \delta = \phi - 5^\circ \quad (3.4)$$

where K is the lateral coefficient of the material (0.8 when it is not given, the value recommended for open-ended piles; 1.0 is recommended for full-displacement piles, and the program notes when the default is used), and N_q , f_{lim} and q_{lim} follow Table 3.1 by interpolation in δ , unless they are entered for the material.

Table 3.1 — API parameters for cohesionless soil

	δ (°)	15	20	25	30	35
Limiting shaft resistance f_{lim} (kPa)		47.8	66.9	81.3	95.7	114.8
Bearing factor N_q		8	12	20	40	50
Limiting tip resistance q_{lim} (kPa)		1 900	2 900	4 800	9 600	12 000

3.3 USACE method

The method of the U.S. Army Corps of Engineers (EM 1110-2-2906) is applied as follows.

Clay. $f_s = \alpha s_u$ with $\alpha = 1.0$ for $s_u \leq 0.25$ tsf (24 kPa), 0.5 for $s_u \geq 0.75$ tsf (72 kPa) and linear in between. For long piles the option “long-pile adhesion” replaces α in the range 24–72 kPa by $\alpha_1 \alpha_2$ after Semple and Rigden, with α_1 varying from 1.0 to 0.5 as s_u / σ'_v increases from 0.35 to 0.8 and α_2 from 1.0 to 0.7 as L/D increases from 50 to 120. The tip resistance is $9 s_u$.

Sand. The vertical effective stress is limited to its value at the critical depth D_c :

$$f_s = K \bar{\sigma}'_v \tan \delta, \quad q_b = N_q \bar{\sigma}'_v, \quad \bar{\sigma}'_v = \sigma'_v (\min(z, D_c)) \quad (3.5)$$

with $D_c = 10B$ for $\phi < 33^\circ$, $15B$ for $33^\circ \leq \phi < 36^\circ$ and $20B$ otherwise (B = tip diameter), or the depth at which σ'_v reaches 3 500 psf (168 kPa) when that option is chosen, or a critical depth entered for the material. The interface angle is $\delta = 0.75\phi$ for steel, 0.95ϕ for concrete and 0.9ϕ for timber (0.875ϕ for steel H-piles, the mean of steel–sand and sand–sand friction) unless entered. K is the USACE value of the material, else its K_0 , else 0.8. N_q follows the bearing-capacity chart of the manual (for example 23 at 30° , 56 at 36° , 88 at 40°) unless entered. For a battered pile the lateral coefficient becomes $K \cos \theta + \sin \theta$. A silt adds the clay adhesion of the standard curve to the friction term.

3.4 FHWA method

The methods of the FHWA manual for driven piles (FHWA-NHI-16-009) are applied: the Nordlund method in cohesionless soil and the α method of Tomlinson in cohesive soil.

Cohesionless soil (Nordlund).

$$f_s = K_\delta C_f \sigma'_v \frac{\sin(\delta + \omega)}{\cos \omega} \quad (3.6)$$

where ω is the taper angle of the pile (zero for a straight pile), K_δ the coefficient of lateral earth pressure from the Nordlund charts as a function of φ , ω and the displaced volume per unit length, C_f the correction factor for K_δ when $\delta \neq \varphi$, and δ/φ follows the chart for the pile type (pipe, timber, precast concrete, Raymond step-taper and uniform taper, H-pile, tapered monotube; chosen automatically from the section or entered). The unit tip resistance is

$$q_b = \alpha_t N'_q \min(\sigma'_v, 150 \text{ kPa}) \leq q_L \quad (3.7)$$

with α_t and N'_q from the charts as functions of φ and the embedment ratio, and the limiting tip resistance q_L as a function of φ . Any of these chart values can be entered for a material instead.

Cohesive soil (Tomlinson). $f_s = \alpha s_u$ with α from the three Tomlinson curves (A: sand or sandy gravel overlying stiff clay; B: soft clay overlying stiff clay; C: stiff clay without different overlying strata) as a function of s_u and of the penetration ratio into the stiff clay; the curve is chosen automatically from the layer sequence, or set for the material, or α is entered. The tip resistance is $9 s_u$.

3.5 Revised Lambda method

The λ method for clay, in the revised form of Kraft, Focht and Amerasinghe, gives the average unit shaft resistance along the embedded length as

$$\bar{f}_s = \lambda (\bar{\sigma}'_v + 2 \bar{s}_u) \quad (3.8)$$

where $\bar{\sigma}'_v$ and $\bar{s}_u$ are mean values over the embedded length (in cohesionless layers $K_0 \sigma'_v \tan \varphi$ takes the place of s_u). The coefficient depends on the flexibility of the pile:

$$\lambda = \max(0.14, a - b \ln \Pi_3), \quad \Pi_3 = \frac{\pi D \bar{s}_u L^2}{EA \Delta}, \quad \Delta = 2.54 \text{ mm} \quad (3.9)$$

with $a = 0.232$, $b = 0.032$ when $\bar{s}_u > 0.4 \bar{\sigma}'_v$ (overconsolidated) and $a = 0.178$, $b = 0.016$ otherwise. The tip and the plug follow the API method. Because λ depends on the whole embedded length, the shaft capacity profile is computed for each penetration; where the formula would give a smaller shaft capacity than at a shallower penetration, the shaft capacity is continued with the gradient of the last increment, and the program notes it.

3.6 User-defined unit resistances

With the method "User defined", the unit shaft and tip resistances entered for each material (constant or varying through the layer) are integrated directly. The limits of the material do not cap them.

3.7 CPT-based methods

The methods NGI-05, ICP-05, UWA-05 and Fugro-05 (Clausen, Aas and Karlsrud; Jardine et al.; Lehane et al.; Kolk et al.; all 2005), recommended for offshore driven piles in API RP 2GEO, evaluate the unit resistances from the cone resistance q_c . They apply to vertical circular and pipe piles. Without CPT readings, q_c is estimated from the SPT blow count of the layer, $q_c = 2.8 N_{60} p_a$ (corrected for overburden), and the program notes that the cone resistance was estimated. The principal relations in sand are, with h the distance from the tip, R^* the equivalent radius and δ_{cv} the constant-volume interface angle ($\varphi - 5^\circ$ when not entered):

Table 3.2 — Shaft relations of the CPT-based methods in sand

Method	Unit shaft resistance
NGI	$f_s = (z/z_{tip}) p_a F_{Dr} (\sigma'_v/p_a)^{0.25} F_{sig} F_{tip} F_{load} F_{mat} \geq 0.1 \sigma'_v, F_{Dr} = 2.1 (D_r - 0.1)^{1.7}$
ICP	$f_s = (\sigma'_{rc} + \Delta\sigma'_{rd}) \tan \delta_{cv}, \sigma'_{rc} = 0.029 q_c (\sigma'_v/p_a)^{0.13} \max(h/R^*, 8)^{-0.38}$
UWA	$f_s = (\sigma'_{rc} + \Delta\sigma'_{rd}) \tan \delta_{cv}, \sigma'_{rc} = 0.03 q_c A_{r,eff}^{0.3} \max(h/D, 2)^{-0.5}$
Fugro	$f_s = 0.08 q_c (\sigma'_v/p_a)^{0.05} (h/R^*)^{-0.9}$ in compression; $0.045 q_c (\sigma'_v/p_a)^{0.15} \max(h/R^*, 4)^{-0.85}$ in tension

The dilatancy term $\Delta\sigma'_{rd} = 4G\Delta r/D$ uses the interface dilation Δr of the material and the shear modulus derived from q_c . In tension and for open piles the methods apply their published reduction factors. In clay, NGI uses its $\alpha(\psi, I_p)$ relation, and ICP, UWA and Fugro use the ICP clay relation $f_s = 0.8 K_c \sigma'_v \tan \delta_f$ with $K_c = (2 - 0.625 \log S_t) \text{YSR}^{0.42} \max(h/R^*, 8)^{-0.2}$, which needs the

remoulded strength (sensitivity S_r) and the yield stress ratio. The tip resistances are fractions of q_c averaged around the tip as published for each method, with an undrained ($0.8 q_c$ closed, $0.4 q_c$ plugged) or drained ($1.3 / 0.65 q_c$) choice for clay at the tip.

3.8 Open-ended piles and plugging

For a pipe pile with an open tip the user chooses the plug condition:

- **Plugged:** the tip resistance acts on the gross area of the tip; there is no internal shaft resistance.
- **Unplugged:** the tip resistance acts on the steel annulus and the shaft resistance acts also inside the pile.
- **Automatic plug:** the soil column that enters the pile during installation is followed for each penetration; the pile is plugged where the internal shaft resistance of the column exceeds the bearing resistance of the gross area less the annulus, and the capacity is the smaller of the two mechanisms. The plug length and the internal resistance are reported. In clay the internal shaft resistance uses the remoulded strength of the material.

A driving shoe (a bottom section with a smaller bore) is taken into account: only the soil inside the shoe length contributes internal friction.

3.9 Design capacity

Three design bases are available:

$$Q_d = \frac{Q_s + Q_b}{F_s} \mp W' \text{ (ASD)}, \quad Q_d = \phi_s Q_s + \phi_b Q_b \mp W' \text{ (LRFD)}, \quad Q_d = Q_n \text{ (nominal)} \quad (3.10)$$

with the upper sign in compression and the lower sign in uplift; the defaults are $F_s = 2.5$, $\phi_s = 0.55$ and $\phi_b = 0.5$.

In LRFD each layer may carry its own shaft and tip resistance factors (zero = the global factors); the shaft factor of each increment is that of its layer, and the tip factor that of the layer at the tip. In uplift the weight of the pile is multiplied by the tip resistance factor of the layer at the tip. The factors act on the capacity only; the load-transfer curves are always unfactored.

4 Load-transfer curves

4.1 Shaft and tip springs

For the response of the pile to axial load, the soil is represented by independent nonlinear springs: along the shaft a t - z curve gives the mobilised unit shaft resistance t as a function of the relative movement z between pile and soil, and at the tip a Q - w curve gives the mobilised tip force Q as a function of the tip movement w (Figure 4.1). Every curve passes through the origin, is piecewise linear between its points, and keeps its last value beyond its last point. Shaft curves act in both directions (the same curve with opposite sign); the tip spring carries compression only.

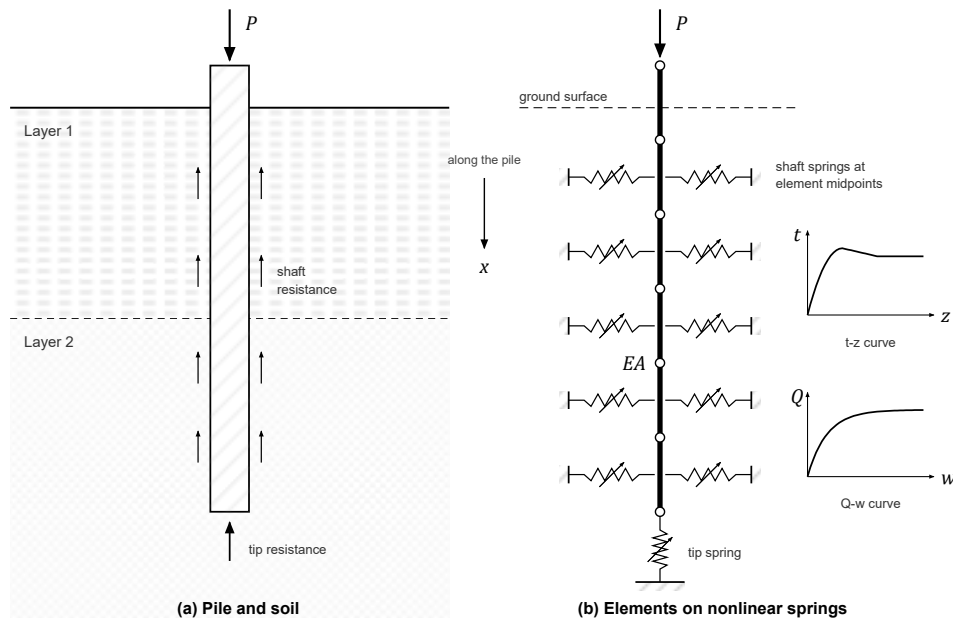


Figure 4.1 — Idealisation of the axially loaded pile: elements, shaft springs (t - z) and tip spring (Q - w)

4.2 Maximum resistances

The curves are normalised shapes multiplied by a maximum resistance:

- in the **Settlement** mode, the maximum unit shaft resistance $t_{\max}$ and the maximum unit tip resistance are the limiting values entered for each soil material; the tip force is the unit value times the bearing area (the annulus for an unplugged open pile);
- in the **Capacity + Settlement** mode, they are taken from the capacity method chosen as reference, so that the load-settlement curve approaches the computed nominal capacity. For Revised Lambda, whose shaft resistance is defined for the whole pile, the local value is the increase of the shaft capacity with penetration divided by the added shaft surface.

In uplift, $t_{\max}$ is reduced by the uplift factor of the capacity (Chapter 3), except for the CPT-based methods.

4.3 Shaft (t - z) curves

The t - z law of each layer is chosen in the material (**t - z law**) or, when it is “Automatic”, by the project: Reese & O’Neill for a drilled shaft; Coyle & Reese in clay and Mosher in sand when the USACE load transfer is selected; otherwise the clay and sand models of the project. Table 4.1 gives the normalised shapes, and Figure 4.2 plots them.

Table 4.1 — Normalised t - z curves

Law	Abscissa	Points (z ; $t/t_{\max}$)
API (clay)	z/z_{peak} , $z_{\text{peak}} = 0.01D$	0.16 ; 0.30 — 0.31 ; 0.50 — 0.57 ; 0.75 — 0.80 ; 0.90 — 1.0 ; 1.0 — 2.0 ; t_{res}
API (sand)	z/z_{peak} , $z_{\text{peak}} = 0.01D$	as clay, without softening ($t_{\text{res}} = 1$)
Coyle & Reese (clay)	z (in)	0.01 ; 0.18 — 0.02 ; 0.38 — 0.04 ; 0.79 — 0.06 ; 0.97 — 0.08 ; 1.0 — 0.12 ; 0.97 — 0.16 ; 0.93
Mosher (sand)	z	hyperbola $t = z / (1/E_s + z/t_{\max})$
Reese & O’Neill, clay	z/D (%)	0.1 ; 0.51 — 0.2 ; 0.72 — 0.3 ; 0.90 — 0.5 ; 0.925 — 0.8 ; 0.955 — 1.2 ; 0.925 — 1.6 ; 0.90 — 10 ; 0.835

Law	Abscissa	Points (z ; $t/t_{\max}$)
Reese & O'Neill, sand	z/D (%)	0.1 ; 0.335 — 0.2 ; 0.58 — 0.3 ; 0.72 — 0.5 ; 0.86 — 0.8 ; 0.95 — 1.2 ; 0.965 — 10 ; 0.95

The residual ratio t_{res} of the API clay curve is entered for the material (0.9 when blank) and the peak movement z_{peak} can be entered too. The initial modulus E_s of the Mosher curve depends on the friction angle: it increases from 6 000 to 10 000 psf/in for ϕ between 28° and 31°, from 10 000 to 14 000 psf/in between 32° and 35°, and from 14 000 to 18 000 psf/in between 35° and 38°. When the sand has an SPT blow count but no friction angle, the angle used for the transfer curves follows the zero-overburden chart of Gibbs and Holtz (32°, 34°, 36°, 38°, 42° and 45° at $N = 2, 4, 6, 10, 15$ and 20).

Manual t-z curves are entered as tables at chosen depths; between two depths the curve is interpolated point by point, and above the first or below the last depth the nearest table applies.

4.4 Tip (Q-w) curves

Table 4.2 — Q-w curves

Law	Definition
API	$w/D = 0.002$; $Q/Q_{\max} = 0.25 — 0.013$; $0.50 — 0.042$; $0.75 — 0.073$; $0.90 — 0.10$; 1.0
USACE	$Q = Q_{\max} \min(1, (w/w_c)^{0.5})$, $w_c = 0.05D$ in sand (or the Z_c of the material), $0.04D$ in clay
Coyle & Reese (clay, Skempton)	$Q = Q_{\max} \min(1, 0.5 (w / 2\varepsilon_{50}D)^{0.5})$, full mobilisation at $w = 8\varepsilon_{50}D$
Mosher (sand)	$Q = Q_{\max} \min(1, (w/z_c)^n)$, $z_c = 0.25$ in; $n = 1/2, 1/3, 1/4$ for $\phi \leq 31^\circ, \leq 34^\circ, > 34^\circ$
Vijayvergiya (sand)	$Q = Q_{\max} (w/z_c)^n$ up to $w = 1$ in, $z_c = 0.05D$; $n = 1/2, 1/3, 1/4$ for $\phi \leq 31^\circ, \leq 38^\circ, > 38^\circ$
Reese & O'Neill, clay	w/D (%) = 0.5 ; 0.395 — 1 ; 0.56 — 1.5 ; 0.70 — 2.5 ; 0.865 — 3 ; 0.90 — 4 ; 0.95 — 5 ; 0.97 — 10 ; 0.99
Reese & O'Neill, sand	w/D (%) = 0.5 ; 0.20 — 1 ; 0.345 — 2.5 ; 0.64 — 4 ; 0.83 — 5 ; 1.015 — 7 ; 1.195 — 10 ; 1.46

The Vijayvergiya curve and the Reese & O'Neill sand curve continue to rise above $Q_{\max}$ at large movements, as published; the other curves are limited to $Q_{\max}$.

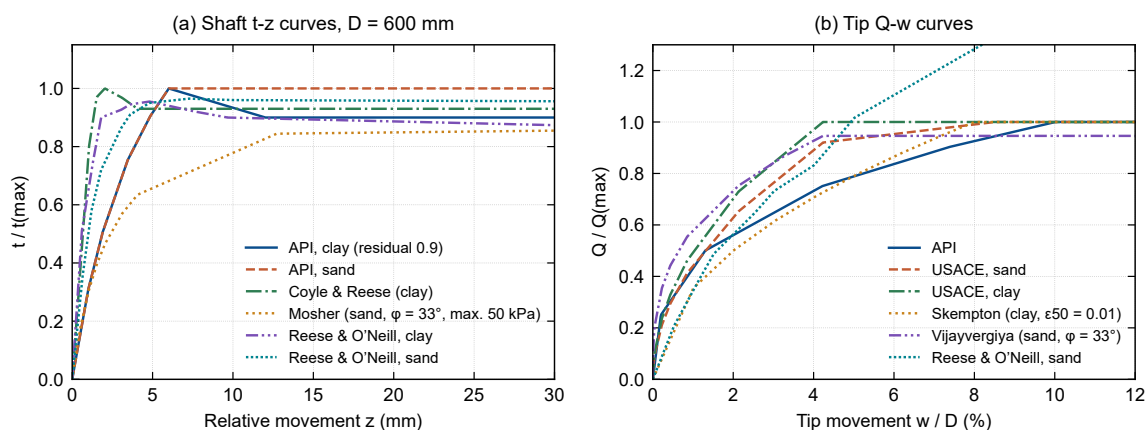


Figure 4.2 — Normalised t-z and Q-w curves as generated by Sol-load for a pile of 600 mm diameter

4.5 Modification factors

A table of modification factors against depth multiplies the resistance (t and Q) and the movement (z and w) of the curves, with a separate factor for the tip movement. The factors scale the curves only; they do not change the capacity profiles.

5 Solution of the axially loaded pile

5.1 Governing equation

Equilibrium and compatibility of a pile element of length Δx give

$$\frac{dP}{dx} = -C t(u - s) + W'_x, \quad \frac{du}{dx} = -\frac{P}{EA} \quad (5.1)$$

where u is the downward movement of the pile, s the settlement of the soil (zero without downdrag), C the perimeter and W'_x the effective weight per unit length (when included). At the tip, $P(L) = Q(w)$ with $w = u(L)$.

5.2 Element equations

The pile is divided into the number of increments chosen in the solver settings, with nodes added at the ground surface, at section joints and, when the effective weight is included, at the water table. The option “Align increments to soil boundaries and movement changes” adds nodes at layer boundaries and at the depths of the modification-factor and soil-settlement tables. Each element carries one shaft spring at its midpoint. Starting at the tip with a trial movement w , the force $P_b = Q(w)$ and the movement at the bottom of each element are known, and the relative movement z_m at the midpoint satisfies

$$z_m - c t(z_m) = w_b + \frac{P_b \Delta x}{2EA} - \frac{W'_x \Delta x^2}{4EA} - s_m, \quad c = \frac{C \Delta x^2}{4EA} \quad (5.2)$$

Because t is piecewise linear, this equation is solved exactly in the segment that contains the root; the element length is halved automatically until c times the steepest slope of the curve is below 0.5, which makes the left-hand side increase monotonically and the root unique. The force and the movement at the top of the element follow as

$$P_t = P_b + t(z_m) C \Delta x - W'_x \Delta x, \quad u_t = w_b + \frac{(P_b + P_t) \Delta x}{2EA} \quad (5.3)$$

Repeating this element by element up to the head gives the head load and head movement that correspond exactly to the trial tip movement. Equilibrium of the whole pile (head load plus weight equals the tip force plus the sum of the shaft forces) is checked for every solution to a relative accuracy of 10^{-8} .

5.3 Load control

With **tip-movement** control the solution above is direct. With **head-load** control the tip movement is found for which the head load equals the target: the tip movement is increased along a logarithmic sequence from 10^{-10} m to a limit that covers the curves, until the head load passes the target on the first loading branch; the crossing is then refined with Brent's method. A solution is accepted only when

$$|P_{head} - P_{target}| \leq \varepsilon \max(1 \text{ kN}, |P_{target}|) \quad (5.4)$$

with ε the convergence tolerance of the solver settings. When the target exceeds the largest head load that the pile reaches on the first loading branch, no result is published for that case; the message gives the largest response reached and the tip movement at which it occurs.

The load-settlement curve is computed, not interpolated: it consists of solutions at the tip movements of the load cases, at 51 tip movements between zero and the largest of them, and, when tip-movement output steps reach further, at additional geometric and linear steps up to the last step.

5.4 Downdrag

With the downdrag option, a profile of soil settlement $s(z)$ is entered. The shaft springs then act on the movement of the pile relative to the soil (5.2): where the soil settles more than the pile, the shaft resistance acts downwards (negative skin friction). The neutral depth, where the relative movement changes sign, is reported for each load case.

5.5 FHWA neutral-plane analysis

The neutral-plane analysis of the FHWA guidance (FHWA-NHI-16-009; Siegel et al.) uses the unfactored FHWA capacity profile down to a chosen toe penetration L . With the permanent load Q_d , the cumulative shaft resistance $S(z)$ and the tip resistance R_t , the neutral plane is the depth where the load curve $Q_d + S(z)$ meets the resistance curve $k R_t + S(L) - S(z)$, that is where

$$S(z) = \frac{1}{2} (k R_t + S(L) - Q_d) \quad (5.5)$$

for the three degrees of tip mobilisation $k = 1, 0.5$ and 0 . The depth is interpolated linearly between capacity rows. The maximum axial load is $Q_d + S(z)$ and the drag force is $S(z)$. The program reports when the curves do not intersect or intersect more than once.

6 Cone penetration test data

6.1 Import

CPT soundings are read from text files with one record per line. The user chooses the column and the unit of depth, cone resistance q_c , sleeve friction f_s and, optionally, pore pressure u_2 , the first and last row, the decimal character and the delimiters. Records are sorted by depth, a repeated depth keeps its first record, and lines without the requested numbers are skipped. A negative friction value (an instrument code for a missing reading) is stored as zero; such a record is skipped by the interpretation, and the CPT-based capacity methods read only its cone resistance.

6.2 Interpretation

Each valid reading ($q_c > 0$, $f_s > 0$) is interpreted with the relations of Robertson and Cabal (CPT Guide). The corrected cone resistance and the unit weight are

$$q_t = q_c + u_2 (1 - a), \quad \frac{\gamma}{\gamma_w} = 0.27 \log R_f + 0.36 \log \frac{q_t}{p_a} + 1.236 \quad (6.1)$$

with the net area ratio a (0.78 by default), $R_f = 100 f_s / q_t$ in per cent and $p_a = 101.325$ kPa. The total vertical stress is the trapezoidal integral of the unit weight from the ground surface; the hydrostatic pore pressure u_0 acts below the water table. The normalised parameters are

$$Q_{t1} = \frac{q_t - \sigma_v}{\sigma_v}, \quad F = \frac{100 f_s}{q_t - \sigma_v}, \quad Q_{tn} = \frac{q_t - \sigma_v}{p_a} \left(\frac{p_a}{\sigma_v} \right)^n, \quad B_q = \frac{u_2 - u_0}{q_c} \quad (6.2)$$

with the stress exponent $n = 0.381 I_{cn} + 0.05 \sigma'_v / p_a - 0.15$, iterated from $n = 1$ until it changes by less than 0.001 and limited to 1. The soil behaviour type indices are

$$I_c = \sqrt{(3.47 - \log Q_{t1})^2 + (\log F + 1.22)^2}, \quad I_{cn} = \sqrt{(3.47 - \log Q_{tn})^2 + (\log F + 1.22)^2} \quad (6.3)$$

and a reading is sand-like for $I_{cn} < 2.6$ and clay-like otherwise. The derived parameters are:

- **Friction angle** (sand-like readings) by one of three selectable correlations: Robertson and Campanella (1983), $\tan \varphi = (\log(q_c / \sigma'_v) + 0.29) / 2.68$; Kulhawy and Mayne (1990), $\varphi = 17.6^\circ + 11^\circ \log Q_{tn}$; or Mayne (2006), $\varphi = 29.5^\circ B_q^{0.121} (0.256 + 0.336 B_q + \log Q_{t1})$. The alternative correlations are also evaluated in clay-like readings.
- **Undrained shear strength** (clay-like readings), $s_u = (q_t - \sigma_v) / N_{kt}$, with $N_{kt} = 10.5 + 7 \log F$ or a fixed value.
- **SPT blow count**, $N_{60} = (q_t / p_a) / 10^{1.1268 - 0.2817 I_c}$.
- **Behaviour type**, bands of I_{cn} at 1.31, 2.05, 2.6, 2.95 and 3.6.

The soundings can be plotted on the soil behaviour type charts of Robertson (1986 and 2010) for inspection; the charts do not assign soil properties.

6.3 Filling layer parameters

The command that fills layer parameters from a sounding uses, for each layer, the first valid reading at or below the top of the layer and the first valid reading at or below its bottom, and projects the line between them to the top and the bottom of the layer. Unit weight, friction angle, undrained strength, N_{60} and q_c are filled this way. These are end-point values, not averages; the engineer should review them against the full sounding before they are used.

7 Verification

7.1 Approach

The implementation is verified at three levels: against closed-form solutions of the governing equations, against hand calculations of the capacity methods for profiles where the integrals can be evaluated exactly, and by checks of equilibrium that do not depend on the empirical laws. The capacity methods and the load-transfer curves themselves are empirical; their validity rests on the load tests reported in the publications cited in Chapter 3 and Chapter 4. All results in this chapter were computed with Sol-load 1.0.0 and are reproducible with the script supplied with the documentation.

7.2 Elastic pile on linear springs

For a pile of constant axial stiffness EA with a linear shaft spring $t = k z$ and a linear tip spring $Q = K_b w$, the governing equations (5.1) reduce to $d^2u/dx^2 = \lambda^2 u$, with the solution, written from the tip ($\xi = L - x$),

$$u = u_b (\cosh \lambda \xi + \Omega \sinh \lambda \xi), \quad P = EA \lambda u_b (\sinh \lambda \xi + \Omega \cosh \lambda \xi), \quad \lambda = \sqrt{\frac{k C}{EA}}, \quad \Omega = \frac{K_b}{EA \lambda} \quad (7.1)$$

The test pile is a 20 m concrete pile of diameter 0.6 m with $E = 30$ GPa ($EA = 8482300$ kN), $k = 20\,000$ kPa/m and $K_b = 200\,000$ kN/m, giving $\lambda L = 1.333$ and $\Omega = 0.354$. The springs are entered as manual t-z and Q-w curves, and a head load of 1 000 kN is applied with a convergence tolerance of 10^{-9} .

Table 7.1 — Elastic pile: comparison with the closed-form solution

Increments N	h (m)	u_0 Sol-load (mm)	Error (%)	P_b Sol-load (kN)	Error (%)
10	2.00	1.88925	2.63e-02	142.166	2.11e-01
20	1.00	1.88962	6.56e-03	142.392	5.28e-02
50	0.40	1.88973	1.05e-03	142.455	8.44e-03
100	0.20	1.88974	2.62e-04	142.464	2.11e-03
200	0.10	1.88975	6.56e-05	142.467	5.28e-04
400	0.05	1.88975	1.64e-05	142.467	1.32e-04

The closed-form values are $u_0 = 1.88975$ mm and $P_b = 142.467$ kN. The error decreases with the square of the element length, as expected for the midpoint scheme of Section 5.2 (Figure 7.1); with the default of 100 increments it is far below any engineering significance. The equilibrium residual of every solution is below 10^{-12} kN.

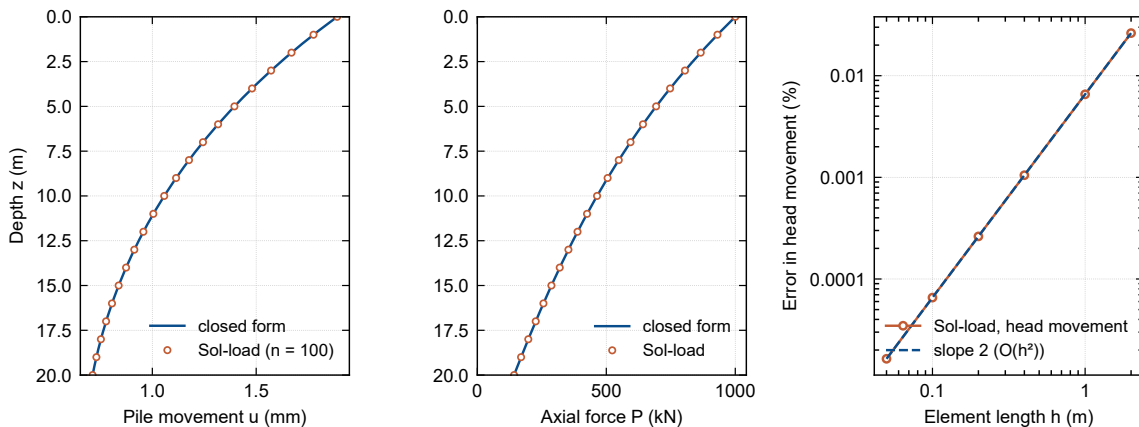


Figure 7.1 — Elastic pile: movement and axial force profiles, and convergence of the head movement with the element length

7.3 Capacity in uniform soil

Three capacity checks use a 0.6 m concrete pile in a single soil layer, for which the integrals of Chapter 3 can be evaluated by hand.

API, sand. $\varphi = 30^\circ$ ($\delta = 25^\circ$), $K = 0.8$, $\gamma_{\text{sat}} = 20$ kN/m³ with the water table at the surface ($\gamma' = 10.1934$ kN/m³), embedded length 15 m. The unit shaft resistance at the tip is 57.04 kPa and the unit tip resistance 3058.0 kPa, both below the limits of Table 3.1, so that

$$Q_s = C K \tan \delta \gamma' \frac{L^2}{2}, \quad Q_b = N_q \gamma' L A_b \quad (7.2)$$

USACE, sand. The same soil and pile; $\delta = 0.95$ $\varphi = 28.5^\circ$ for concrete, $K = 0.8$, $D_c = 10B = 6$ m and $N_q = 23$, so that

$$Q_s = C K \tan \delta \gamma' \left(\frac{D_c^2}{2} + D_c (L - D_c) \right), \quad Q_b = N_q \gamma' D_c A_b \quad (7.3)$$

API, clay. $s_u = 200$ kPa, $\gamma = 18$ kN/m³ without a water table, embedded length 20 m. Above $z_1 = s_u/\gamma = 11.11$ m, $\psi > 1$ and $f_s = 0.5 s_u^{0.75} (\gamma z)^{0.25}$; below it, $f_s = 0.5 (s_u \gamma z)^{0.5}$. Integrating both branches,

$$Q_s = C \left[\frac{0.5 s_u^{0.75} \gamma^{0.25} z_1^{1.25}}{1.25} + \frac{0.5 (s_u \gamma)^{0.5} (L^{1.5} - z_1^{1.5})}{1.5} \right], \quad Q_b = 9 s_u A_b \quad (7.4)$$

Table 7.2 — Capacity checks against hand calculation

Case	Hand (kN)	Sol-load (kN)	Difference (%)
API sand, shaft	806.37	806.37	2.2e-14
API sand, tip	864.63	864.63	-6.7e-14
USACE sand, shaft	600.90	600.90	-1.6e-13
USACE sand, tip	397.73	397.73	-1.1e-14
API clay, shaft, step 0.2 m	3651.16	3653.67	0.069
API clay, shaft, step 0.1 m	3651.16	3652.21	0.029
API clay, shaft, step 0.05 m	3651.16	3651.60	0.012
API clay, tip	508.94	508.94	0.0e+00

The sand cases agree to round-off, because the unit resistance is linear or piecewise linear in depth and the integration is exact for it. In the clay case the unit resistance varies with $z^{0.25}$ and $z^{0.5}$; the difference is the trapezoidal integration error of the capacity step and decreases as the step is reduced.

7.4 Response at large movement

In the Capacity + Settlement mode the shaft and tip curves are scaled to the reference capacity method. For the API sand case above, a tip movement of $0.25D = 150$ mm mobilises the full shaft and tip resistance, and the head load must equal the nominal capacity. Sol-load gives 1670.998 kN against a nominal capacity of 1670.998 kN (shaft 806.368 kN, tip 864.631 kN): the load-transfer analysis and the capacity analysis use the same resistances.

7.5 Neutral plane

For the downdrag example of the User Manual (FHWA method, permanent load $Q_d = 500$ kN, tip resistance 565.5 kN, shaft resistance 1643.0 kN), the cumulative shaft resistance at the computed neutral depth was compared with the right-hand side of (5.5):

Table 7.3 — Neutral-plane check

k	Neutral-plane penetration (m)	$S(z)$ (kN)	$\frac{1}{2}(k R_t + S(L) - Q_d)$ (kN)	Max. axial load (kN)
1.0	13.026	854.250	854.250	1354.3
0.5	11.776	712.879	712.879	1212.9
0.0	10.526	571.507	571.507	1071.5

7.6 Equilibrium of nonlinear solutions

For nonlinear curves no closed-form solution exists, but every converged solution must satisfy global equilibrium: the head load plus the pile weight equals the tip force plus the sum of the shaft forces. Sol-load checks this for every solution and rejects one with a relative residual above 10^{-8} . For the examples of the User Manual the residuals are:

Table 7.4 — Equilibrium residuals of the User Manual examples

Example and load case	Head load (kN)	Shaft force (kN)	Tip force (kN)	Residual (kN)
1 — driven pile, Service	800.00	653.67	146.33	1.1e-13
1 — driven pile, Test	1200.01	1021.58	178.43	2.3e-13
2 — drilled shaft, Test	1200.00	1162.11	37.89	0.0e+00
3 — downdrag, Test	1200.00	961.46	238.54	8.0e-13

In the downdrag example the shaft force is the net of the negative skin friction above the neutral depth and the positive resistance below it.

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