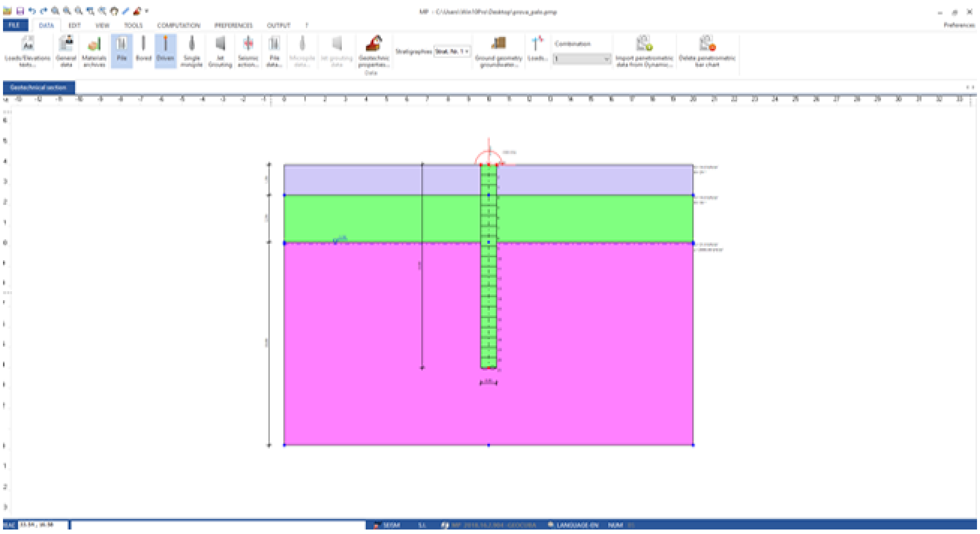
MP

1 Pile and Micropile

Pile and micropile software is intended for calculation of the bearing capacity of the foundation terrain of a pile or micropile (Screw-piles) bearing loads in whatever distribution (moment, normal force, shear). Structural calculation yielding dimensions of longitudinal steel struts, interval and size of rings is also performed.



Pile and micropile software is intended for calculation of the bearing capacity of the foundation terrain of a pile or micropile in whatever load distribution (moment, normal force, shear).

The worksheet, upon which is shown the plan of piles or micropiles inserted in the foundation terrain, may be dimensioned as required.

Geometric characteristics of the paling and the elements connected thereto (loads, location of trial sounding/boring and seismic stress) together with the geotechnic parameters of the terrain may be entered and altered within the worksheet activating the available functions either in the drop down menus, relocatable toolbars or through hot keys.

Pile Types

- Driven and bored
- Summit linked or independent piles
- Calculation of conical trunk piles
- Calculation of marginal point load according to Berezantzev, Hansen, Janbu, Vesic, Terzaghi
- Calculation of lateral load capacity according to Tomlinson
- Horizontal bearing capacity
- Seismic corrections according to Okamoto
- Occurrence of surcharges on terrain
- Presence of aquifer
- Long and short term analysis
- Calculation of horizontal reaction modulus according to Chiarugi-Maia

- Settlements according to Davis-Poulos
- Structural calculation of section and stress diagrams
- Numeric report on marginal load based on length variation
- Graphic on marginal load based on length variation
- Computation of Screw-piles.
- Pressiometric method

Micropile types

- For Tubifix types Mayer or Bustamante e Doix methodology may be applied
- Micropiles in layers having raised mechanic characteristics
- Schneebeli method
- Calculation of horizontal reaction modulus according to Chiarugi-Maia
- Structural calculation of section
- Settlements according to Davis-Poulos

1.1 Conventions

Loads

Use the following conventions for loads acting on the single piles or minipiles.

Horizontal forces (Fo)

Positive figure for force acting right to left.

Vertical forces (Fv)

Positive figure for force acting downwards.

Moments (M)

Positive if acting in clockwise direction.

Displacements

Displacements

Positive if acting towards the right.

Rotations

Positive if acting in clockwise direction.

Forces acting on the foundation (where operating with minipiles)

Horizontal forces (Fo)

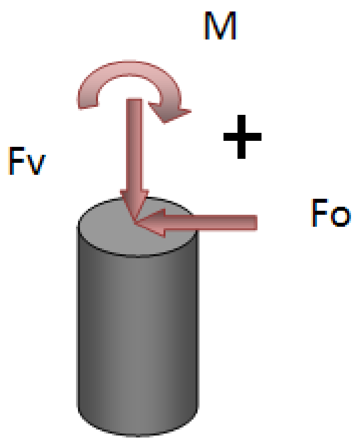
Positive figure for force acting right to left.

Vertical forces (Fv)

Positive figure for force acting downwards.

Couple (M)

Positive if acting in clockwise direction.



2 Pressuremeter

1. Charge limite d'un élément de fondation Qu
Qu = Qpu + Qsu

1.1 Effort mobilisable sous la pointe Qpu

$$Qpu = A \cdot qu \text{ ou } Qpu = \rho_p \cdot A \cdot qu$$

- **A** = section de la pointe
- ρ_p = coefficient réducteur (cas de pieux ouverts, H, palplanches)
- $\mathbf{q_u}$ = contrainte de rupture : $q_u = k_q \cdot P_{le}^*$

- $\mathbf{P_{le}^*}$ = pression limite nette équi.

$$P_{le}^* = \frac{1}{b+3a} \int_{D-b}^{D+3a} P_l^*(z) \cdot dz$$

- **b** = min {a,h}
- **a** = pris égal à la moitié de la largeur B de l'élément de fondation si celle-ci est supérieure à 1,00 m et à 0,50 m dans le cas contraire.
- **h** = désigne la hauteur de l'élément de fondation contenue dans la formation porteuse.
- $\mathbf{p_l^*(z)}$ est obtenue en joignant des segments de droite sur une échelle linéaire les différents pl* mesurées.
- $\mathbf{k_p}$ = facteur de portance donnée en fonction de la catégorie de sol et du type de pieu lorsque la profondeur d'encastrement équivalente De est supérieure à la profondeur critique Dc (De ≥ Dc , Dc ≥ 5B).

Facteur k_p pour (D_e/B ≥ 5)			
Nature de terrains		Eléments mis en œuvre sans refoulement du sol	Eléments mis en œuvre avec refoulement du sol
Argiles – Limons	A	1.1	1.4
	B	1.2	1.5
	C	1.3	1.6
Sables –Graves	A	1.0	4.2
	B	1.1	3.7
	C	1.2	3.2
Cayes	A	1.1	1.6
	B	1.4	2.2
	C	1.8	2.6
Marnes – Marno calcaire		1.8	2.6
Roches altérées (*)		1.1 a 1.8	1.8 a 3.2

Catégories conventionnelles des sols			
Classe de sol			P_l (MPa)
Argiles – Limons	A	Argiles et limons mous	< 0.7
	B	Argiles et limons fermes	1.2-2.0
	C	Argiles très fermes à dures	> 2.5
Sables –Graves	A	Lâches	< 0.5
	B	Moyennement compacts	1,0 – 2,0
	C	Compacts	> 2,5
Cayes	A	Molles	< 0,7
	B	Altérées	2.2
	C	Compactes	> 3,0
Marnes – Marno-calcaire	A	Tendres	1.5-4.0
	B	Compacts	> 4,5
Roches	A	Altérées	2,5 – 4,0
	B	Fragmentées	1.8 a 3.2

- **De** : hauteur d'encastrement équivalente

$$D_e = \frac{1}{P_{le}^*} \int_d^D p_l^*(z) \cdot dz$$

1.2 Effort limite mobilisable par frottement latéral Qsu

Choix des abaques pour la détermination de qs et courbes												
	Argile - Limon			Sables - Graves			Craie			Marnes		Roches
	A	B	C	A	B	C	A	B	C	A	B	
Foré simpl	Q1	Q1, Q2 (1)	Q2, Q3 (1)	---			Q1	Q3	Q4, Q5 (1)	Q3	Q4, Q5 (1)	Q6
Foré boue	Q1	Q1, Q2 (1)		Q1	Q2, Q1 (2)	Q3, Q2 (2)	Q1	Q3	Q4, Q5 (1)	Q3	Q4, Q5 (1)	Q6
Foré tubé (tube récupéré)	Q1	Q1, Q2 (1)		Q1	Q2, Q1 (2)	Q3, Q2 (2)	Q1	Q2	Q3, Q4 (3)	Q3	Q4	--
Foré tubé (tube perdu	Q1			Q1		Q2	(4)			Q2	Q3	--
Puits (5)	Q1	Q2	Q3	--			Q1	Q2	Q3	Q4	Q5	Q6
Métal battu fermé	Q1	Q2		Q2		Q3	(4)			Q3	Q4	Q4
Battu préfabriqué Béton	Q1	Q2		Q3			(4)			Q3	Q4	Q4
Battu moulé	Q1	Q2		Q2		Q3	Q1	Q2	Q3	Q3	Q4	--
Battu errore	Q1	Q2		Q3		Q4	(4)			Q3	Q4	--
Injecté basse Pression	Q1	Q2		Q3			Q2	Q3	Q4	Q5		--
Injecté haute Pression (6)	--	Q4	Q5	Q5		Q6	--	Q5	Q6	Q6		Q7(7)

$$Q_{su} = P \cdot \int_0^h q_s(z) \cdot dz \quad \text{ou} \quad Q_{su} = \rho_s \cdot P \cdot \int_0^h q_s(z) \cdot dz$$

(1) Réalésage et rainurage en fin de forage
(2) Pieux de grande longueur (supérieure à 30 m)
(3) Forage à sec, tube non louvoyé
(4) Dans le cas des craies, le frottement latéral peut être très faible pour certains type de pieux. Il convient d'effectuer une étude spécifique
(5) Sans tubage, ni virole foncés perdues (paroi rugueuse)
(6) Injection sélective et répétitive à faible débit
(7) Injection sélective et répétitive à faible débit et traitement préalable des massifs fissurés ou fracturés avec obturation des cavités

- **P** = périmètre de l'élément de fondation
- **q_s(z)** = frottement latéral unitaire limite à la cote z,
- **ρ_s** = coefficient réducteur (cas de palplanches)

- Courbes Q_1 à Q_4 (n désignant le numéro de la courbe)

$$si \ \frac{P_l}{P_n} \leq 1 \quad q_s = q_{sn} \cdot \frac{P_l}{P_n} \cdot \left(2 - \frac{P_l}{P_n}\right) \quad \text{simon} \quad q_s = q_{sn}$$

avec

$$q_{sn} = 0.04 \cdot n \ (\text{MPa}) \ P_n = (1 + 0.5 \cdot n) \ (\text{MPa})$$

Ces courbes étant bornées supérieurement par la courbe Q5.

- Courbes Q_5 à Q_7

$$-Q_5: \quad q_s = \min(\frac{p_l - 0.2}{9}; \frac{p_l + 3.3}{32}) \quad \text{pour} \quad p_l \geq 0.2\,\text{MPa}$$

$$-Q_6: \quad q_s = \min(\frac{p_l + 0.4}{10}; \frac{p_l + 4.0}{30}) \quad (\text{en général} \quad p_l \geq 1.0\ \text{MPa})$$

$$-Q_7: \quad q_s = \frac{p_l + 0.4}{10} \quad (\text{en général} \quad p_l \geq 2.5\,\text{MPa})$$

2. Charge de fluage Q
Mise en œuvre sans refoulement

Qc = 0.5·Qpu + 0.7·Qsu

Mise en œuvre avec refoulement

Qc = 0.7·Qpu + 0.7·Qsu

3. Etats limites de mobilisation du sol
E.L.U - C. fondamentales: Qu / 1.40
E.L.U - C. accidentelles: Qu / 1.20
E.L.S - C. rares: Qu / 1.10
E.L.S - C. quasi - permanentes: Qu / 1.40

3 Screw piles

Screw-Piles and Helical Anchors in Soils
This Guide should be used for preliminary calculations only and applies only to the deep installation of Screw-Piles and Helical Anchors in uniform soils. It is only applicable for design when the depth (D) to the top helical plate is greater than 10 times the diameter (B) of the helical plate and the minimum depth of embedment of the helical plate is 5 ft. The methods described in this Guide provide an estimate of the ULTIMATE capacity; the Engineer must apply an appropriate Factor of Safety to obtain the ALLOWABLE capacity.

General Bearing Capacity Equation
At the present time, the design of Screw-Piles and Helical Anchors generally follows the traditional theory of General Bearing Capacity used for compression loading of foundations. Terzaghi's general bearing capacity equation for determining ultimate bearing capacity, as given in most Foundation Engineering textbooks is often stated as:

$$q_{ult} = c'N_c + q'N_q + 0.5\gamma'BN_\gamma$$

where:
qult = Ultimate Unit Bearing Capacity
c' = effective cohesion
q' = effective overburden stress = γ'D
γ' = effective unit weight of soil
D = depth
B = diameter of helix
Nc, Nq, Nγ = bearing capacity factors

Notes on use of Terzaghi's General Bearing Capacity equation:

1. Because B is considered very small for Screw-Piles and Helical Anchors, relative to most concrete footings, some engineers choose to ignore the term $0.5\gamma' B N\gamma$ in design.
2. In saturated clays under compression loading, Skempton's (1951) Bearing Capacity Factor for shallow round helical plates may also be used:

$$NC = 6.0(1 + 0.2D/B) < 9.0$$

3. The unit weight of the soil is the total (wet) unit weight if the helical plate is above the water table and the buoyant unit weight if the helical plate is below the water table.
4. For saturated clay soils with $\varphi' = 0$, $Nq = 1.0$; For sands, Nq is a function of friction angle, φ'
5. In all cases, for both compression and tension loading, the upper limit of capacity is governed by the mechanical strength of the Screw-Pile or Helical Anchor as provided by the manufacturer.

Contribution of Shaft to Capacity

Many Screw-Piles and Helical Anchors are manufactured with square central shafts. For these piles/anchors, the contribution of the shaft to the ultimate capacity is usually ignored and the total capacity is only calculated from the bearing capacity of the helical plate(s). For Screw-Piles and Helical Anchors with round steel central shafts the shaft section between plates for multi-helix elements is ignored, but the shaft above the top plate may be included in design, at least for that section of the shaft in full contact with the soil as discussed in Section 3.

DEEP Single-Helix Screw-Piles and Helical Anchors

Deep installations of Screw-Piles and Helical Anchors are generally more common than shallow installations, provided there is sufficient soil depth to perform the installation. The reason is that higher load capacities are generally developed from a deeper installation in the same soil.

Compression Loading of Screw-Piles in CLAY

Under both compression and tension loading of deep Screw-Piles and Helical Anchors in clay, the ultimate capacity is obtained using the Total Stress Analysis (TSA) and undrained shear strength. In saturated clays with $\varphi' = 0$ and $c = su$ the bearing capacity equation is often give as:

$$QH = AH(Nc)su$$

where:

QH = Ultimate Bearing Capacity in Compression
su = undrained shear strength
Nc = Bearing Capacity Factor for clays with $\varphi' = 0$; for round plates $NC = 6.0(1 + 0.2D/B) < 9$
AH = Effective area of the helical plate For deep installations, NC = 9, which gives: QH = AH(9)(su)

For deep installations, Nc = 9, which gives:

QH = AH(9)(su)

Compression Loading of Screw-Piles in SAND

For deep installations of single-helix Screw-Piles and Helical Anchors in sand the ultimate capacity is obtained using the Effective Stress Analysis (ESA) from:

$$QH = AH(\sigma'_{vo}\,Nq + 0.5\gamma' B N\gamma)$$

where:

σ'_{vo} = vertical effective stress at the depth (D) of the helix = $\gamma'D$

Nq and Nγ = bearing capacity factors

B = Diameter of the helical plate

γ' = effective unit weight of the soil

The bearing capacity factor Nq is usually obtained from values used for determining the end bearing capacity for deep pile foundations. There have been a number of different recommendations for estimating Nq which are available in most foundation engineering textbooks, e.g., Fang & Winterkorn 1983:

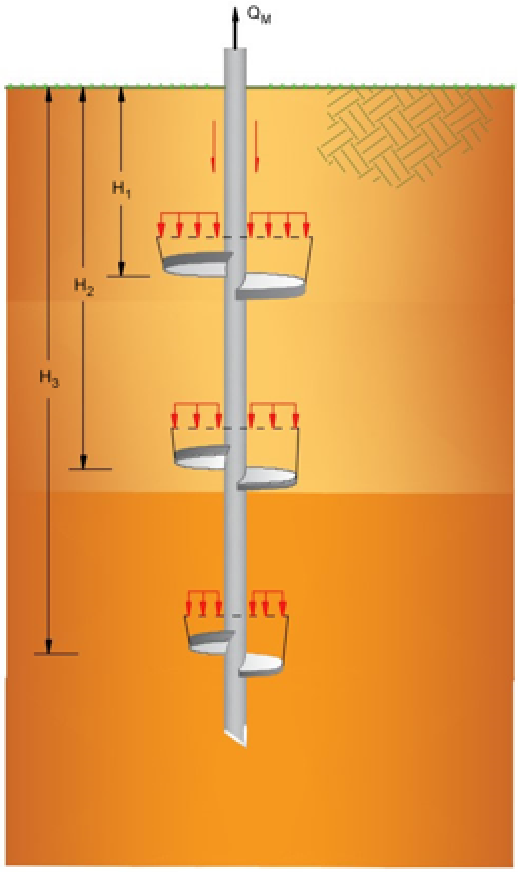
$$Nq = 0.5\,(12 \times \varphi')^{(\varphi'/54)}$$

Because the area of the plate is usually small, the contribution of the “width” term (0.5γ'BNγ) to ultimate capacity is also very small and the width term is often ignored. This reduces to

$$QH = AH(\sigma'_{vo}\,Nq)$$

DEEP Multi-Helix Screw-Piles and Helical Anchors

The ultimate capacity of deep multi-helix Screw-Piles and Helical Anchors depends on the geometry of the helical section, namely the size and number of helical plates and the spacing between the plates. In the U.S. most manufacturers of Screw-Piles and Helical Anchors produce elements with a helix spacing of 3 times the helix diameter. This spacing is assumed to allow individual plates to develop full capacity with no interaction between plates and the total capacity is often taken as the sum of the capacities from each plate as shown in Figure.



Development of Capacity for Multi-Helix Screw-Piles and Helical Anchors with S/D >3.

Compression and Tension Loading of Multi-Helix Screw-Piles

Ultimate capacity of multi-helix Screw-Piles in compression and Helical Anchors in tension with a helix spacing/diameter ration > 3 is often taken as the summation of the capacities of the individual plates:

$$QM = \Sigma QH$$

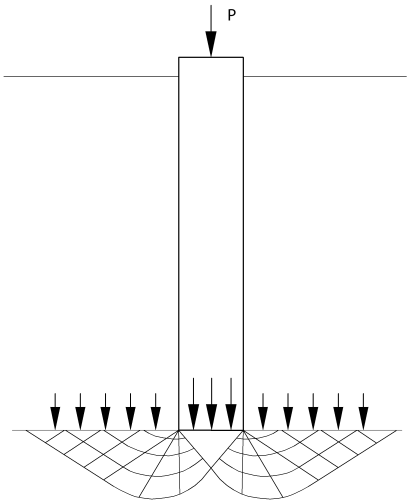
where:
QM = Total Capacity of a Multi-Helix Screw-Pile/Helical Anchor
QH = Capacity of an Individual Helix

Reference
Dr. Alan J. Lutenegger, P.E., F. ASCE for International Society for Helical Foundations (ISHF)

4 Technical Notes

4.1 Pile Point Resistance

The formula for bearing capacity proposed by **Terzaghi**, reproduced below, assumes that that the soil existing up to the point of embedment, can be substituted by a surcharge equal to the effective vertical tension (ignoring the fact that the interaction between pile and soil may modify this value) and thus reduces the problem to the analysis of bearing capacity of surface foundation.



Terzaghi formula

This may be written:

$$Q_p = c \cdot Nc \cdot s_c + \gamma \cdot L \cdot N_q + 0.5 \cdot \gamma \cdot D \cdot N_\gamma \cdot s_\gamma$$

where:

$$N_q = \frac{a^2}{2\cos^2\left(45 + \frac{\phi}{2}\right)}$$

$$a = e^{(0.75\pi - \phi/2)\tan\phi}$$

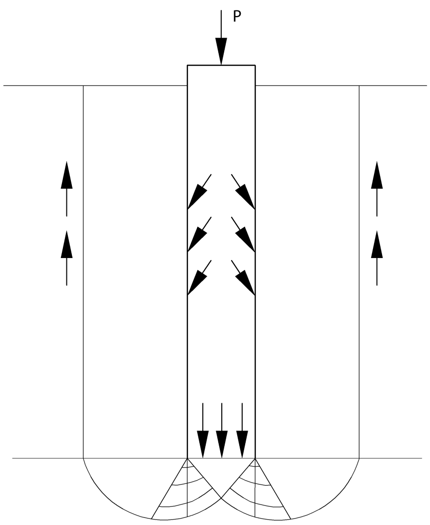
$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = \frac{\tan\phi}{2}\left(\frac{K_{p\gamma}}{\cos^2\phi} - 1\right)$$

ϕ, gradi	N_c	N_q	N_γ	$K_{p\gamma}$
0	5.7	1.0	0.0	10.8
5	7.3	1.6	0.5	12.2
10	9.6	2.7	1.2	14.7
15	12.9	4.4	2.5	18.6
20	17.7	7.4	5.0	25.0
25	25.1	12.7	9.7	35.0
30	37.2	22.5	19.7	52.0
34	52.6	36.5	36.0	
35	57.8	41.4	42.4	82.0
40	95.7	81.3	100.4	141.0
45	172.3	173.3	297.5	298.0
48	258.3	287.9	780.1	
50	347.5	415.1	1153.2	800.0

Berezantzev's method

Fundamentally **Berezantzev** refers to a slip surface of Terzaghi's model which terminates at the base level (Pile point/tip) he however considers that the cylinder of soil with the same axis as the pile, whose diameter is the extent of the section of the slip surface, be in some measure sustained by tangential action, by the rest of the soil along the lateral surface. From this arises a decreased value of pressure on the inferior base as this silo effect increases, i.e. for every increase in the ratio D/B, which is accounted for by the coefficient N_q .



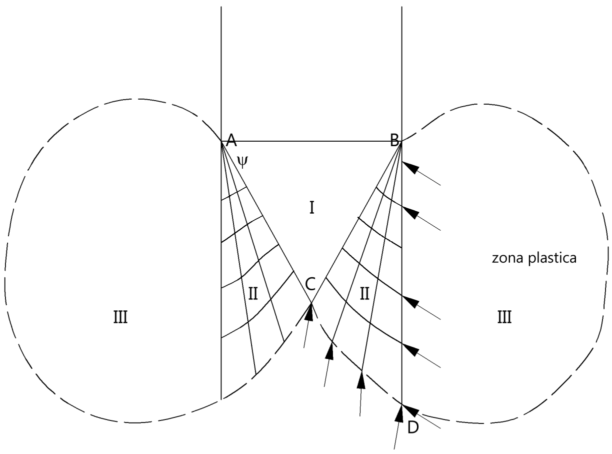
For soil with friction (φ) and cohesion (c), unit resistance Q_p at the point is given by the expression:

$$Q_p = c' N_c + \gamma' L' N_q$$

where:
γ = is unit weight of the soil;
L = length of pile;
N_c and N_q = are the bearing capacity factors including of the effect due to the shape (circular).

Vesic's method
Vesic likens the problem of failure at the pile tip to that of the expansion of a cylindrical cavity immersed in a elastoplastic medium, so that even the compressibility of the medium is taken in account.
According to Vesic the bearing capacity coefficients N_q e N_c may be evaluated as shown:

$$N_q = \frac{3}{3 - \sin\phi}\left\{\exp\left[\left(\frac{\pi}{2} - \phi\right)\tan\phi\right]\tan^2\left(45 + \frac{\phi}{2}\right) I_{rr}^{(4\sin\phi)/[3(1+\sin\phi)]}\right\}$$



Reduced rigidity index I_{rr} in the preceding expression is evaluated based on volumetric deformation ε_v.

$$I_{rr} = \frac{I_r}{1 + I_r \cdot \varepsilon_v}$$

Rigidity index is evaluated using tangential elasticity modulus G' and the soil's shear resistance. When soil is undrained or soil is in dense state, the term ev may be assumed to be zero thus rendering I_{rr}=I_r.
I_r may be taken from the following table:

Terreno	I_r
Sand	75-150

Silt	50-75
Clay	150-250

The coefficient Nc is evaluated as:

$$N_c = (N_q - 1)\cot\phi \qquad (a)$$

When φ = 0 (undrained condition):

$$N_c = \frac{4}{3}(\ln I_{rr} + 1) + \frac{\pi}{2} + 1$$

Janbu's method

Janbu evaluates N_q as follows: (The angle y is expressed in radians)

$$N_q = \left(\tan\phi + \sqrt{1+\tan^2\phi}\right)^2 \exp(2\psi\tan\phi)$$

N_c may be obtained from (a) when $\varphi > 0$. Where $\varphi = 0$ use $N_c = 5.74$.

Hansen's formula

Hansen's formula is valid for all ratios D/B and so both for surface and deep foundations. However the author introduced a number of coefficients, in order to better reflect the actual behaviour of foundations. Without these there would be a too great increase in bearing capacity with increase in depth.

For values L/D < 1 :

$$d_c = 1 + 0.4\tan^{-1}\frac{L}{D}$$

$$d_q = 1 + 2\tan\phi(1-\sin\phi)^2\tan^{-1}\frac{L}{D}$$

Where φ = 0:

D/B	0	1	1.1	2	5	10	20	100
d'c	0	0.40	0.33	0.44	0.55	0.59	0.61	0.62

In the following, the expressions with (') are applicable when φ =0.

Form (shape) factor:

$$s'_c = 0.2\frac{D}{L}$$

$$s_c = 1 + \frac{N_q}{N_c}\frac{D}{L}$$

$$s_q = 1 + \frac{D}{L}\tan\phi$$

$$s_r = 1 - 0.4\frac{D}{L}$$

Depth factor:

$$d'_c = 0.4k$$

$$d_c = 1 + 0.4k$$

$$d_q = 1 + 2\tan\phi(1 - \sin\phi)k$$

$$d_r = 1 \qquad \text{per qualsiasi k}$$

$$k = \tan^{-1}\frac{L}{D} \qquad \text{se } \frac{L}{D} > 1$$

4.1.1 Lateral limit load

Lateral bearing capacity is calculated using method A proposed by **Tomlinson** (1971) according to the following:

$$Q_l = (\alpha c + \sigma K \tan\delta) \cdot Al \cdot f_w$$

Al = Lateral pile surface
f_w = Correction factor connected to the conic form of the pile. i.e. the percentage diminution of the pile diameter.
c = Average cohesion value (or shear resistance in undrained conditions).
s = Effective vertical pressure of the terrain.
K= Coefficient of horizontal thrust. This depends on the technique of the pile and on the previous compaction state and is calculated as :

- **For driven piles**

  $k = 1 - \tan^2\phi$

or may be selected, in concrete cases, from the following table:

Palo		K values	
	Loose Soils		Dense Soils
Steel	0.5		1
Prescast. Concrete	1		2
Wood	1		3

- **For drilled piles**

$k = 1 - \text{sen}\phi$

δ = Friction between pile and soil. A function of the roughness of the pile surface:

- For driven piles

$$\delta = \frac{3}{4}\tan\phi$$

- For drilled piles

$$\delta = \tan\phi$$

α = Adhesion coefficient obtained from the following guidelines

- **For drilled piles**

- *Caquot – Kerisel*

$$\alpha = \frac{100 + c^2}{100 + 7c^2}$$

- *Meyerhof – Murdock (1963)*

$\alpha = 1 - 0.1c$ per $c < 5\, t/m^2$

$\alpha = 0.525 - 0.005 \cdot c$ per $c \geq 5\, t/m^2$

- *Whitaker – Cooke (1966)*

$\alpha = 0.9$ per $c < 2.5\, t/m^2$

$\alpha = 0.8$ per $2.5 \leq c < 5\, t/m^2$

$\alpha = 0.6$ per $5 \leq c \leq 7.5\, t/m^2$

$\alpha = 0.9$ per $c > 7.5\, t/m^2$

- *Woodward (1961)*

$\alpha = 0.9$ per $c < 4\ t/m^2$

$\alpha = 0.6$ per $4 \leq c < 8\ t/m^2$

$\alpha = 0.5$ per $8 \leq c \leq 12\ t/m^2$

$\alpha = 0.4$ per $12 \leq c \leq 20\ t/m^2$

$\alpha = 0.20$ per $c > 20\ t/m^2$

- **For driven piles**

$2.5 \leq c < 5\ t/m^2$	$\alpha = 1.00$
$5 \leq c < 10$	$\alpha = 0.70$
$10 \leq c < 15$	$\alpha = 0.50$
$15 \leq c < 20$	$\alpha = 0.40$
$c \geq 20$	$\alpha = 0.30$

4.2 Negative Skin Friction

When a pile is driven through a layer of compressible material, before consolidation is complete, the soil will move downwards relative to the pile inducing friction forces between pile and soil that give rise to the phenomenon of so called negative skin friction. The effect of negative skin friction is to raise the axial load on the pile with consequent increase in settlement due to the elastic shortening of the pile under the increased load. The force arising fromthe negative skin friction effect is assessed as the frictional component of lateral resistance (see Trunk Resistance) along the side surface in contact with the layer in which the phenomenon arises, but of opposite sense to positive friction. The resultant force so determined is not subtracted from limit load but from the operational load.

4.3 Minipile in Operation

Analysis of pile or minipile in normal operation is effected using **Finite Elements Method** (FEM).
Finite Elements Method models realistically foundation piles/minipiles subjected to transverse loads considering both displacements and rotation at nodes, to define the elastic line of the pile and thus is the most realistic and effective method available to analize this type of structure.
Below are recalled the broad lines of the method.
P is the matrix of external nodal forces. F is the matrix of internal forces.
A is the matrix of influence factors which due to the equilibrium of internal

and external forces binds the first two according to the well known relation:

$$P = AF$$

Internal displacements e (translation and rotation) of the element in the generic node are linked to the external displacements X (translation and rotation) applied to the nodes by the relation:

$$e = BX$$

where the matrix X is matrix A transposed.
On the other hand the internal forces F are linked to internal displacements by:

$$F = Se$$

Which by substitution gives:

$$F = SA^TX$$

and therefore:

$$P = AF = A\ SA^TX$$

Calculating the inverse of matrix A SA^T one obtains the expression for external displacements:

$$X = (A\ SA_T)^{-1}P$$

When displacements X are known, it is possible to deduce the internal forces F required for the project structure.
The matrix A SA^T is known as the global rigidity matrix in that it links nodal displacements and external forces. The method further has the advantage of permitting known rotations and displacements to be taken into account as boundary conditions.
The nodal reactions of the springs that represent the terrain are considered as global forces related to the modulus of subgrade reaction and to the area of influence of the node.In the pile/minipile solution by finite elements subjected to transverse loads, subgrade reaction modulus is considered in the form:

$$k_s = A_s + B_sZ^n$$

or alternatively, where it is desired to contain the growth of the modulus with increase in depth:

$$k_s = A_s + B_s\tan^{-1}(Z/B)$$

in which Z is the depth and B the diameter of pile/minipile.
The values of A_s e B_sZ^n are obtained from the expression for bearing capacity (Bowles) with correction factorss$_i$, d_i, & i_i set to 1:

$$k_s = q_{ult}/DH = C(cN_c + 0.5\gamma BN_g)$$
$$BsZn = C(\gamma N_qZ^1)$$

Where C=40 is obtained in relation to a maximum settlement of 25 mm.

4.4 Minipile Critical Load

Due to their decided slenderness, it is opportune to verify the stability of elastic equilibrium of Tubifix minipiles embedded in the terrain.In the interests of safety, computation assumes that the trunk head be hinged/pinned into the foundation while the bulb be embedded in an elastic medium. Critical load is then determined by the following relationship:

$$P_k = \frac{\pi^2 \cdot E \cdot J}{L^2} \times \left(m^2 + \frac{\beta \cdot L^4}{m^2 \cdot \pi^4 \cdot E \cdot J} \right)$$

where:
P_k = Critical load
E = Steel elastic modulus
J = Moment of inertia of the reacting section
L = Length between the extremities of the minipile that are assumed to be bound
b = Terrain reaction modulus per unit of lateral displacement
m = Integer number of halfwaves of trunk flexion

$$\beta = K \cdot D_p$$

D_p= Diametro di perforazione
K= Modulo di Winkler

When L reaches very high values, the assumption of single deformation (m=1) brings P_k to unlikely values. The minimum value of P_k is obtained for m >1.
By intoducing the dimension l = L / m (halfwave length):

$$P_k = \pi^2 \cdot E \cdot J \cdot \left(\frac{1}{\lambda^2} + \frac{\beta \cdot \lambda^2}{\pi^4 \cdot E \cdot J} \right)$$

To obtain the value of P_k from the above lambda (l) may be considered as a continuous variable in whose relation P_k may be derived:

$$\frac{dP_k}{d\lambda} = \pi^2 \cdot E \cdot J \cdot \left(-\frac{2}{\lambda^3} + \frac{2 \cdot \beta \cdot \lambda}{\pi^4 \cdot E \cdot J} \right) = 0$$

$$\lambda = \pi \cdot \sqrt[4]{\frac{E \cdot J}{\beta}}$$

$$P_k = 2 \cdot \sqrt{\beta \cdot E \cdot J}$$

$$J = \frac{\pi}{64} \cdot \left(D_e^{\,4} - D_i^{\,4}\right) + \frac{\pi}{64} \cdot \frac{1}{n} \cdot D_i^{\,4} + \frac{\pi}{64} \cdot \frac{k_i}{n} \cdot \left(D_p^{\,4} - D_e^{\,4}\right)$$

D_i = Internal tube diameter
D_e = External tube diameter
D_p = Drill dimeter
n = Homogeneity modulus steel concrete
K_i = Coefficient between 0 and 1 that indcates the degree of steel concrete participation.

4.5 Seismic Correction

Seismic corrections to the angle of friction of layer bearing the foundations, are only relevant in well compacted, cohesionless soils. It is incorrect to apply them in loose or averagely compacted soils, as in these, seismic vibrations produce an effect opposite to expansion causing increase in compaction and angle of friction.
Correction factor applicable in computation of bearing capacity in seismic conditions, that account for the phenomenon of expansion may be Menu data between the authors below:

Vesic
According to this author it is sufficient to reduce the angle of friction in the foundation layers, by 2º. The problem with this proposal is that it takes no account of the intensity of the specific seismic stress (see max. seismic acceleration parameter). On the other hand this correction seems to be confirmed by actual observations in a number of seismic events.

Sano
The author proposes a reduction in the angle of friction in the bearing layers according to the following:

$$D_p = \operatorname{arctg}\left(\frac{A_{max}}{\sqrt{2}}\right)$$

where Amax is the maximum seismic acceleration.
This method in contrast to Vesic does take account of the intensity of seismic stress. Experience however seems to demonstrate that an uncritical application of the method leads to excessively conservative values of Qlim

Okamoto
The author proposes a reduction in the angle of friction in the bearing layers according to the following:

$$D_p = 1 - A_{max}$$

where A_{max} is the maximum seismic acceleration.

4.6 Settlements (Pile)

Vertical settlements are calculated using the Davis-Poulos method, according to which the pile is considered as rigid (undeformable) embedded in an elastic medium, semispace, or layer of finite thickness. The hypothesis considers that the interaction between pile and soil is constant for each (n) cylindrical segments in which the pile side surface is subdivided.
The settlement of the i th surface due to the load transmitted by the pile to the soil along the j th surface may be expressed as:

$$W_{i,j} = \left(\frac{\tau_j}{E}\right) \cdot B \cdot I_{i,j}$$

where:

- τ_j = Increment in tension at the mid point of the segment.
- E = Elastic modulus of the terrain.
- B= Diameter of the pile.
- $I_{i,j}$ = Coefficient of influece.

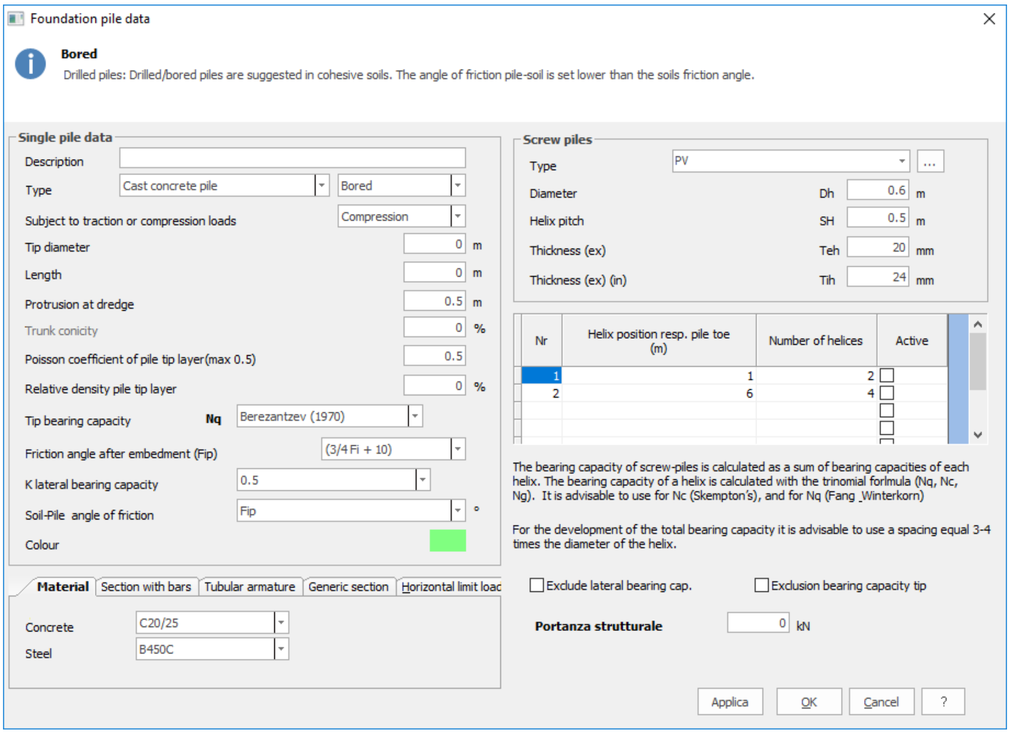
Total settlement is obtained by the sum of $W_{i,j}$ for all j areas.

5 Pile/Minipiles

Pile and Minipile Grids is a program for the calculation of the bearing capacity of the foundation for a single Pile, single minipile or a grid of Minipiles, carrying any combination of loads (moment, normal force and shear). The program also performs structural calculation for dimensions of longitudinal armature and tie binders.
The pile or minipiles, embedded in their foundation terrain, are displayed in a worksheet window, sized as required.

The geometric and physical (loads, materials) characteristics of the pile and or minipiles, and geotechnic properties of the terrain can all be specified and edited within the workspace.

GENERAL CHARACTERISTICS



Pile types

Drilled and driven;

onic trunk pile calculation;

Point limit load calculation according to: *Berezantzev, Hansen, Janbu, Vesic, Terzaghi;*

Calculation of lateral bearing capacity according to Tomlinson;

Seismic corrections according to: *Sano, Okamoto and Vesic;*

Occurrence of water table;

Short and long term analysis;

Calculation of horizontal reaction according to *Chiarugi-Maia, & Bowles;*

Settlements according to Davis-Poulos;

Analysis of stress in non linear Finite Elements. Boundary conditions and nodal actions may be assigned;

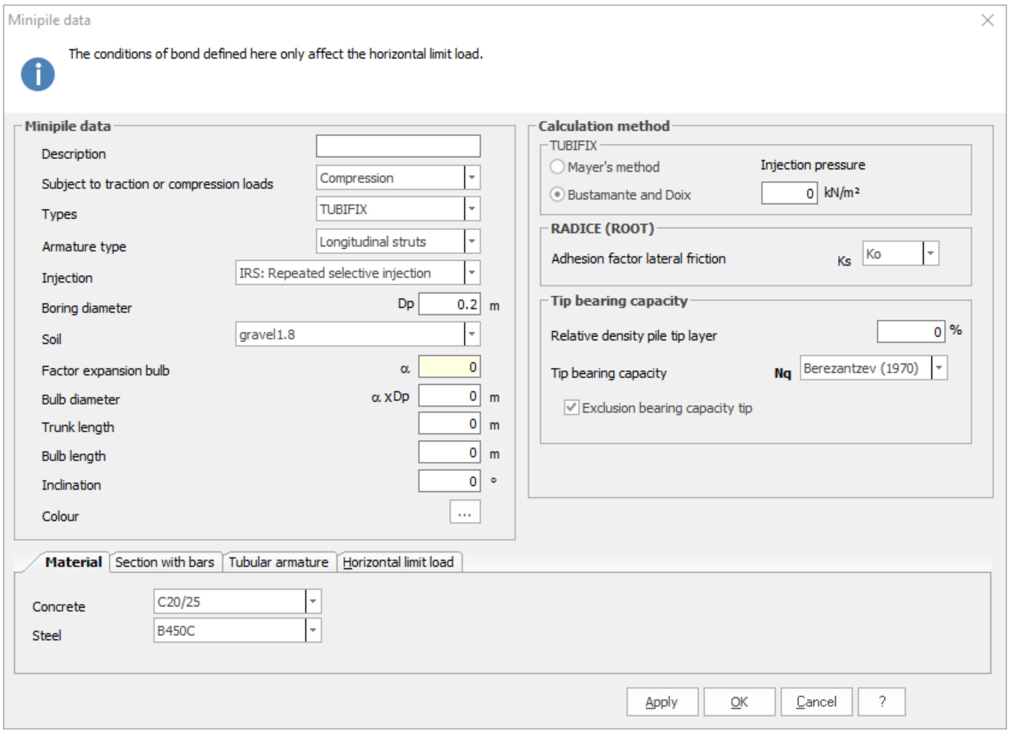
Display of bending moment, shear and deformation diagrams;

Structural calculation of permissible tension and ultimate limit state according to EC2 (& D.M.96-NTC2008-NTC2018 Italy)

Calculation of limit horizontal load;

Evaluation of failure moment for the section;

- Calculation of limit load for multiple combinations of diameter and length.



Minipile types *(Single and grouped minpiles can be calculated)*
Tubifix e Radice;
Short and long term analysis;
Calculation of horizontal reaction according to Chiarugi-Maia;
Settlements according to Davis-Poulos
Analysis of stress in non linear Finite Elements;

- Structural calculation of permissible tension and ultimate limit state according to EC2.

6 Pile Data

This section resides within the Foundation Pile Window and is devoted to the properties of the pile itself and contains the following entry boxes:

Description
Give textual description for this type of pile.

Type
Select from drop down list either driven or bored type of pile. See also guide tip which appears when the box is active..

Point/Tip diameter (m)
Enter pile point (lower) diameter.

Length (m)

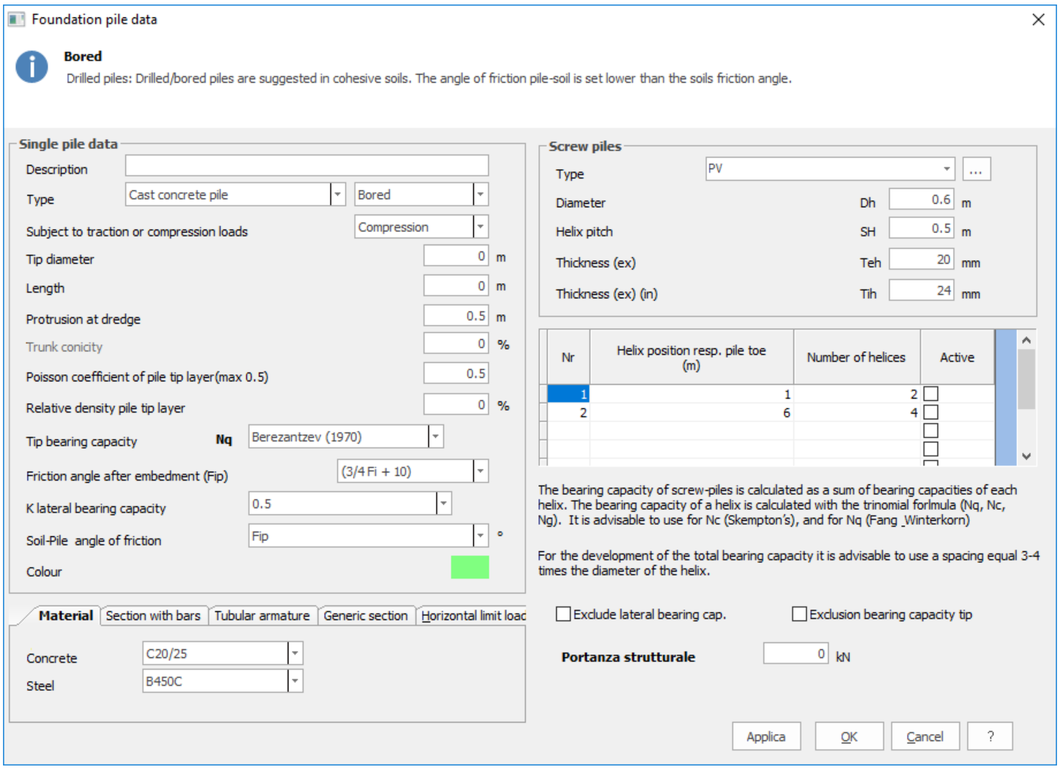
Enter pile length (depth) of the pile measured from excavation dredge line. Please note that at least this depth must have been specified in the stratigraphy window such that the pile does not extend below the lowest layer boundary.

Protrusion at dredge (m)
Height of protrusion of top of pile over the level of excavation dredge line.

Trunk conicity %
The data to enter defines whether the pile is a conic section as opposed to cylindrical and with what angle, its diameter decreases. The value is only relevant for driven piles. Where relevant enter a percentage figure that indicated the diminution percent for each meter length. (10% indicates 10 cm./ meter). See also guide tip which appears when the box is active.

Poisson coeff. of pile tip layer(max 0.5)
Enter value of Poisson's coefficient for the layer at which the pile tip is immersed. This is required for settlement calculation. See also guide tip which appears when the box is active.

Relative density of pile tip layer (%)
Enter value of relative density for the layer at which the pile tip is immersed. This is required for calculation of tip bearing capacity using Vesic's method.



Tip bearing capacity (Nq)
Select from drop down list the author (Berezantev, Terzaghi, Janbu, Hansen e Vesic or User) of the method for computation of tip bearing capacity. (See also technical notes). If 'user' is selected, an additional box opens beside the list, to enable the user determined value to be entered.
Friction angle after embedment (Fip):Select the angle of friction to be used for bearing capacity when the pile is in place. For driven piles it is

suggested to use ¾ the soil value increased by 10; For bored piles it is normal to diminish the soil value by 3º. See also guide tip which appears when the box is active.

K lateral bearing capacity
Select from drop down list the value to assign to coefficient K for the computation of the lateral bearing capacity of the pile. For bored piles the usual value is =1-sin(Fip); for driven piles the usual value is= $1\text{-}\tan^2(Fip)$; Value = 0.5 is common for steel piles and = 1 for prefabricated concrete or wooden piles. If 'user' is selected, an additional box opens beside the list, to enable the user determined value to be entered. See also guide tip which appears when the box is active.

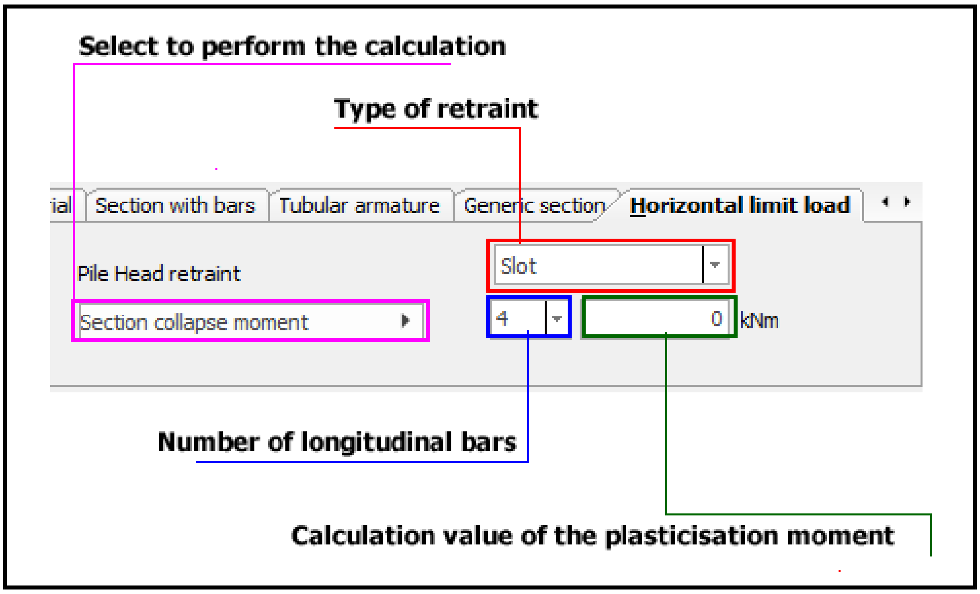
Soil-Pile angle of friction
Select from drop down list, the value to use for delta in computation of lateral bearing capacity of the trunk of the pile. For bored piles the usual value is Fip (See above) while for driven prefabricated concrete piles, the usual value is ¾ Fip. For steel piles the usual value is 25º. See also guide tip which appears when the box is active.

Pile Head restraint
Select from list either slot or hinge as relevant. This selection is only active if either 'Structural computation? or 'Limit horizontal load' check boxes in General Data window have been selected. The specification is needed for the calculation of limit horizontal load and the plastification moment of the section.

Section collapse moment (Kg/m)
This selection is only active if either 'Structural computation? or 'Limit horizontal load' check boxes in General Data window have been selected. Insert the value of plastification moment of the section. See also guide tip which appears when the box is active. Where the material properties have already been entered (Material properties in Data menu), the program is able to calculate this figure. Click in the box and then on the blue underlined text below the tip text box which reads: 'Compute section collapse moment'. This asks for the actual number of rod armatures and then fills the result in the box.

Colour
Opens a colour palette from which the colour of the pile on the diagram may be selected.

The calculation of the horizontal limit load is subject to the calculation of the moment of rupture of the section, access the label Horizontal limit load, select the type of retraint of pile head, define the number of longitudinal bars (for sections in reinforced concrete) and click on the button Section collapse moment.



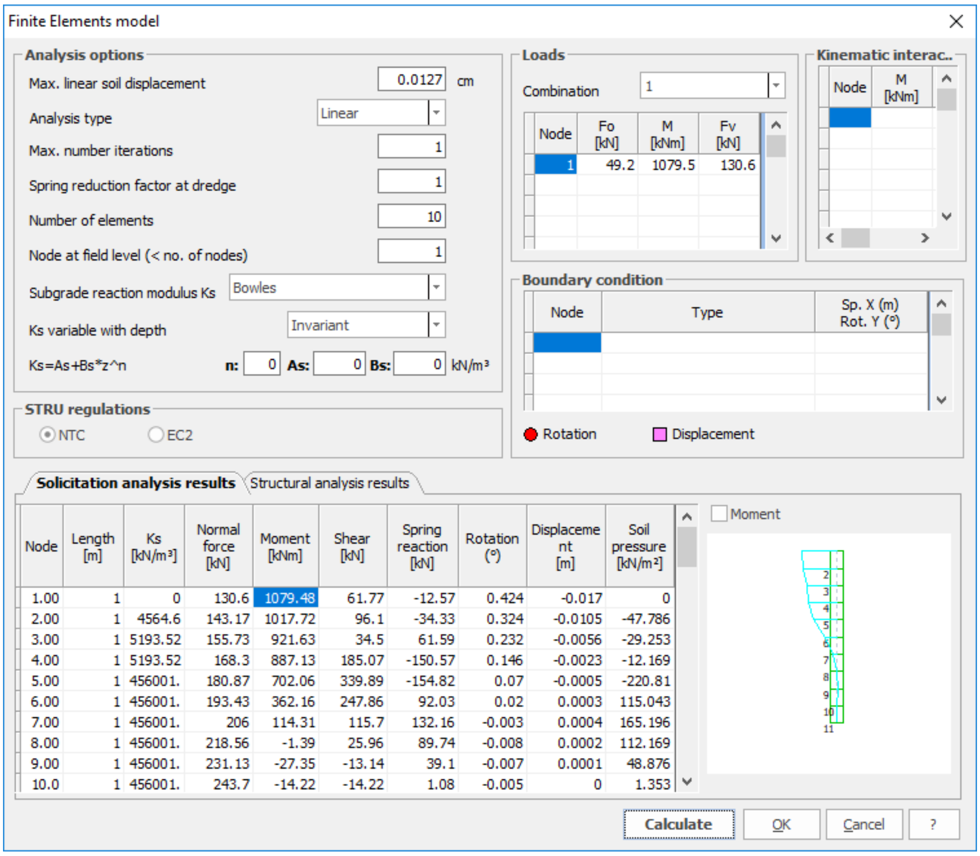
7 Fem data

This window enables entry of parameters required for Finite Elements Method in structural analysis. It is only available if in Structural computation was selected in the Global Data window. The window is made up by three segments: 'Analysis Options', 'Loads', and 'Boundary conditions'. In Analysis Options the following variables should be determined:

Max. linear soil displacement (m)
Enter maximum linear soil displacement in metres.This value embodies a boundary condition for nodal reactions of the springs by which the terrain is represented.

Analysis Type
Select linear or non linear from the dropdown list. The two types refer to the terrain deformation in the area selected by the user.

Max. number iterations
Terminator value for iteration process in determination of displacement matrix and stresses.



Spring reductn. factor at dredge line

Specify factor by which spring rigidity at the bottom of excavation should be reduced. This is useful to take account of soil recast during the insertion of pile or minipile, If no account is to be given to this effect, enter the value 1.

Number of elements

Enter number of elements into which the pile (or minipile) is to be subdivided for the determination of stress and deformation.

Node at field lvl. (< no. of nodes)

Give node ordinal number for node at excavation dredge line. (Nodes are numbered from top downwards). This number will necessarily be less than the number of elements above.

Subgrade reaction modulus Ks

Select from the two alternatives *(Chiarugi-Maia/ Bowles)* in the drop down list the method for the computation of the subgrade reaction modulus. See also Minipile in Operation.

Ks variable with depth

Select either whether subgrade reaction modulus should be varied with depth or held constant. In Loads a table is presented in which load(s) may be declared. It contains the following columns:

Node

Node number to which the load is applied. (Nodes are numbered from top downwards).

Fo
Enter Horizontal force value.

M
Enter couple value.

Fv
Enter Vertical force value.
In Boundary conditions a table is presented in which node condition(s) may be declared. It contains the following columns:

Node
Node number to which the condition applies. (Nodes are numbered from top downwards).

Type
Select from list either Displacement or Rotation to be applied.

Sp. X(m) Rot Y (º)
Enter either displacement in metres or rotation angle depending on type chosen in previous cell.

Note: *Conventions, recalled in tip text which appears when cursor is in one of the columns, whereby Horizontal forces (Fo) are given as +ve when acting right to left; Vertical forces (Fv) are given as +ve when acting downwards; Couple (M) or rotations are given as +ve when clockwise.*

8 Diagrams

Piles & Minipile program produces a full report of the parameters entered and of the result calculations that can be viewed (and edited) from the file menu and the export menu. When the analysis is requested some summary results are produced directly on screen. The result summaries are tailored to the type of support under analisis (Pile/Minipile).

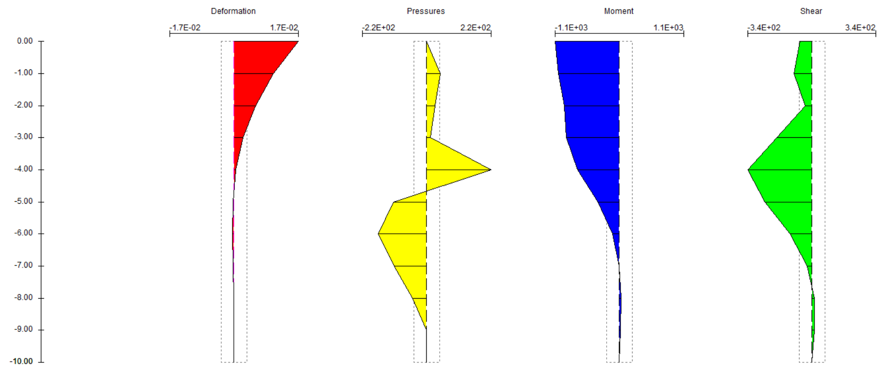
For Minipiles, a summary of Limit load and settlements display, and a Structural analysis table for each minipile, are produced.

For Piles, a summary of Limit load and settlements table, and a Structural analysis table, are produced.

These table may be exported or printed from a floating menu (Right mouse).

The Load & Settlements table for piles also enable (Right mouse) a graphic of loads for variations of pile depth/ diameter to be produced for each step within the variations suggested to the primary pile dimensions, in Foundation Pile.

For Piles, additionally, directly from the Computation menu, graphics of moments, shear, and pressures lines, can be invoked and appear superimposed on the diagram of the pile. They can be printed by invoking graphic print from the File menu.



9 Geoapp

Geoapp: the largest web suite for online calculations

The applications present in Geostru Geoapp were created to support the worker for the solution of multiple professional cases.
Geoapp includes over 40 applications for: Engineering, Geology, Geophysics, Hydrology and Hydraulics.

Most of the applications are free, others require a monthly or annual subscription.

Having a subscription means:

- access to the apps from everywhere and every device;
- saving files in cloud and locally;
- reopening files for further elaborations;
- generating prints and graphics;
- notifications about new apps and their inclusion in your subscription;
- access to the newest versions and features;
- support service throught Tickets. Enter topic text here.

9.1 Geoapp Section

General and Engineering, Geotechnics and Geology

Among the applications present, a wide range can be used for **MP**. For this purpose, the following applications are recommended:

- Horizontal reaction coefficient of foundation piles
- Calculation
- Poles and micropoles
- Load test
- Soil classification
- Newmark

10 References

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Vesic A.S., 1970. "Tests on Instrumented Piles, Ogeehee River Site," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 96, No. SM2, pp. 561-584.

11 Contact

	Phone
	(+39) 0690 289 085 (+40) 737 28 38 54
	E-mail
	info@geostru.eu office@geostru.eu
	Working hours
	Monday – Friday 9 – 17 (GMT + 2)
	Customer support
	For customer support please open a ticket.