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5. Analysis of foundation

5.1 Loading, load cases

This chapter describes an input of loading and load cases, respectively into programs „**Spread footing**“ and „**Piles**“. An arbitrary number of load cases can be created in the „**Loading**“ dialog window, see **Fig. 5.1**.

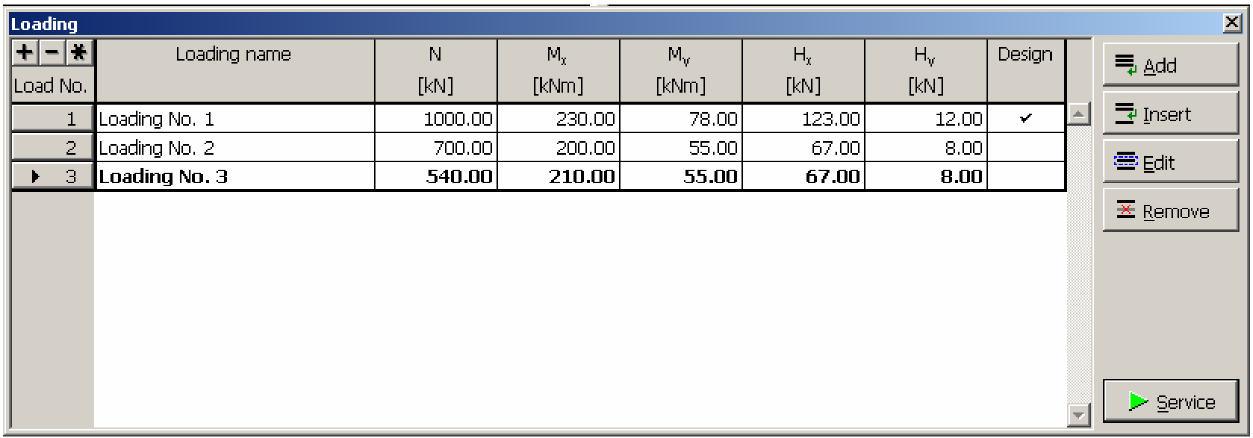


Fig. 5.1 List of load cases

A special attention should be paid to sign convention and to the orientation of Cartesian axes (**Fig. 5.2**). The program „**Spread footing**“ further distinguishes between the „**Design**“ and „**Service**“ loading (**Fig. 5.2**). The design loading is selected when the analysis is carried out according to the first limit state (1LS) (bearing capacity, dimensioning), whereas the service loading enters the analysis based on the second limit state (2LS) (settlement, rotation).
The analysis dialog windows also allow selection of a number of load cases in the analysis; either all (i.e., to determine the largest deformation for individual verifications) or only one load case can be selected (**Fig. 5.3**).

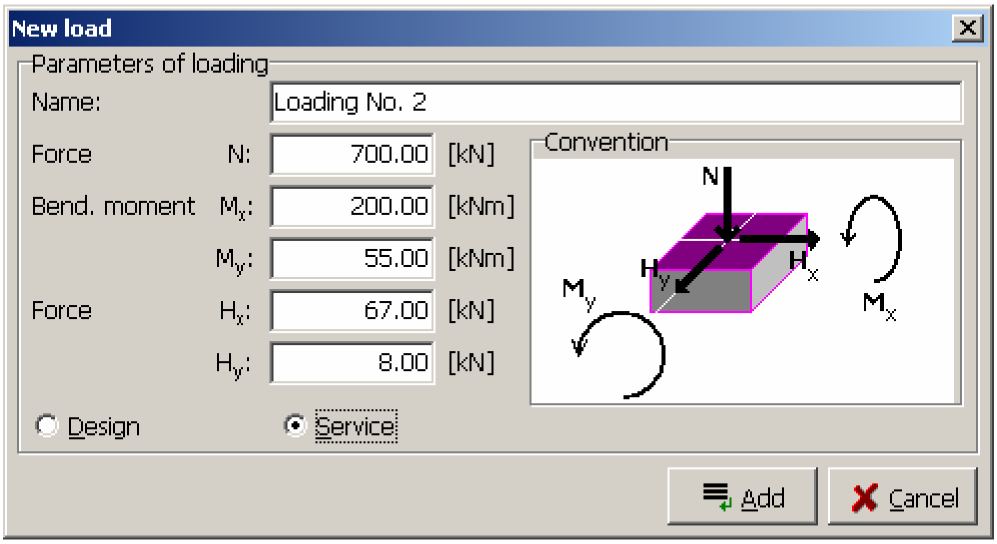


Fig. 5.2 Sign convention

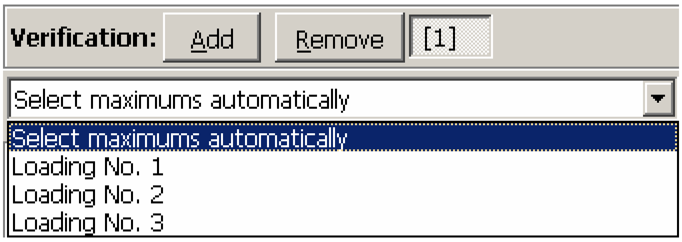


Fig. 5.3 Selection of load cases

5.2 Geostatic stress, uplift pressure

The program further inserts fictitious layers at the locations where the stress and lateral pressure (GWT, points of construction, etc.) change. The normal stress in the i[th] layer is computed according to:

$$\sigma_{zi} = \sum h_i \gamma_i$$

where:

h_i - thickness of the i[th] layer [m],
σ_{zi} - normal stress in the i[th] layer
γ_i - bulk weight of soil [kN/m^3]

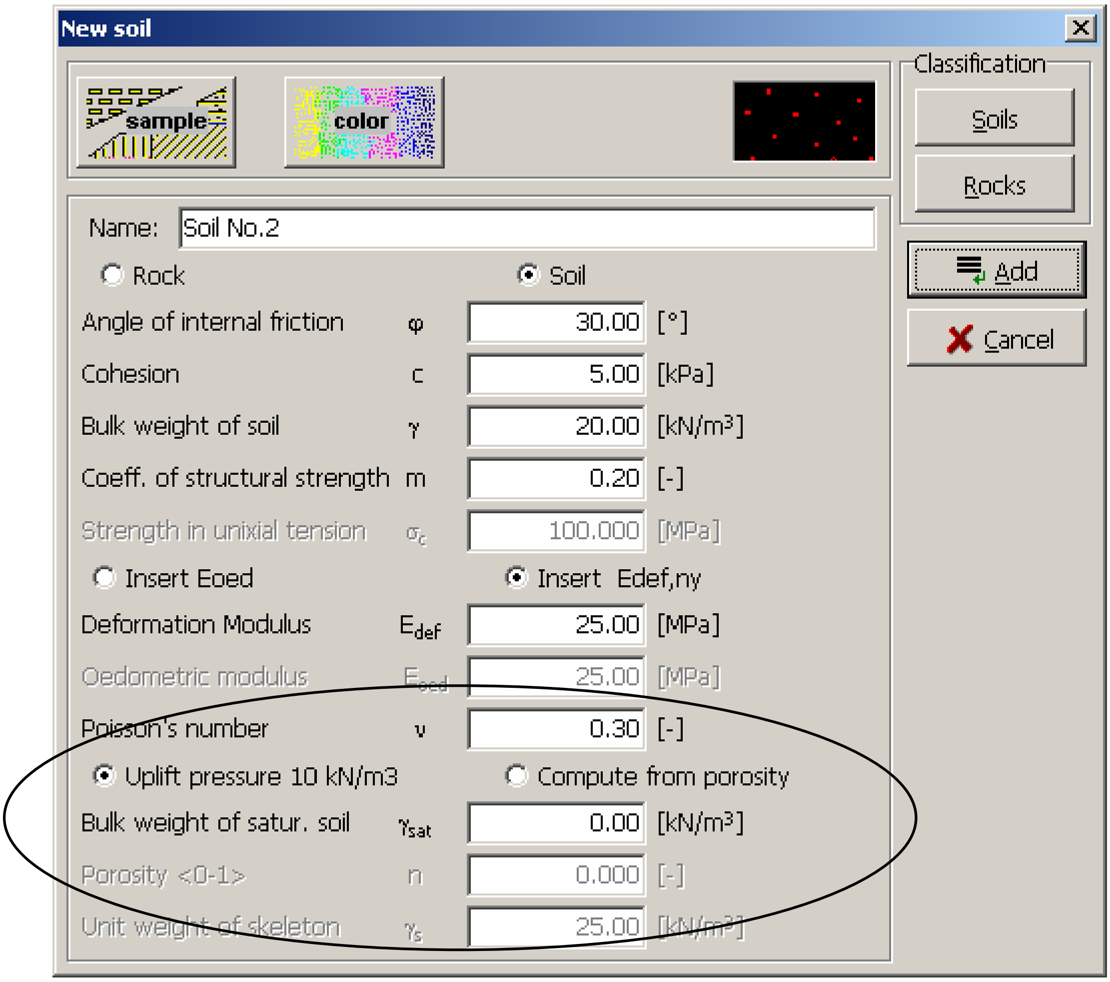


Fig. 5.4 Input of soil parameters to compute γ_{su}

If the layer is found below the ground water table, the bulk weight of a soil under the water must be defined with respect to given parameters of soil (see **Fig. 5.4**) as follows:

- option "Uplift pressure 10 kN/m3":

$$\gamma_{su} = \gamma_{sat} - 10$$

- option "Compute from porosity":

$$\gamma_{su} = \frac{1-n}{\gamma_s - 10}$$

5.3 Program „Spread footing"

5.3.1 Analysis 1st LS - Vertical bearing capacity

The vertical bearing capacity of foundation soil is verified using the following formula

$$\sigma_{de} < R_d$$

where:

σ_{de} - extreme design contact stress in foundation soil [kPa],
R_d - design bearing capacity of foundation soil [kPa].

Design stress at the footing bottom is given by

$$\sigma_{de} = \frac{Q}{A_{ef}}$$

where:

$$Q = N + \gamma_{z1} G + \gamma_{z2} Z$$

$$A_{ef} = b_{ef} l_{ef}$$

$$b_{ef} = b - abs\left(2\frac{M_x + H_y t + N p_x}{Q}\right)$$

$$l_{ef} = l - abs\left(2\frac{-M_y + H_x t + N p_y}{Q}\right)$$

where:

N - vertical extreme design force in column axis (inputted load),
G - footing self-weight – computed automatically from the footing dimensions and bulk weight of footing material (use the "**Material**" dialog window for its input),
γ_{z1} - design coefficient of footing self-weight (use the „**Geometry**" dialog window for its input - default *1.1*).,
Z - self-weight of soil above the footing (filling) – computed automatically from the footing dimensions and bulk weight of soil above of footing
γ_{z2} - design coefficient of self-weight of soil above footing (use the „**Geometry**" dialog window for its input - default *1.3*),
H_x, H_y - horizontal forces at the footing upper surface,

M_x, M_y - moments along the column axis,
b, l - footing dimensions,
t - footing thickness,
p_x, p_y - column axis offset from the footing center (eccentric footings only).

$$R_d = c_d.N_c.s_c.d_c.i_c + \gamma_1.d.N_d.s_d.d_d..i_d + \gamma_2.\frac{b}{2}.N_b.s_b.d_b.i_b$$

where:

$$N_c = (N_d - 1)\cot an(\varphi_d)$$
$$N_d = \tan^2(45 + \varphi/2).\exp(pi.\tan\varphi_d)$$
$$N_b = 1{,}5.(N_d - 1).\tan\varphi_d$$

$$S_c = 1 + 0{,}2\frac{b}{l}$$
$$S_d = 1 + \frac{b}{l}\sin\varphi_d$$
$$S_b = 1 - 0{,}3\frac{b}{l}$$

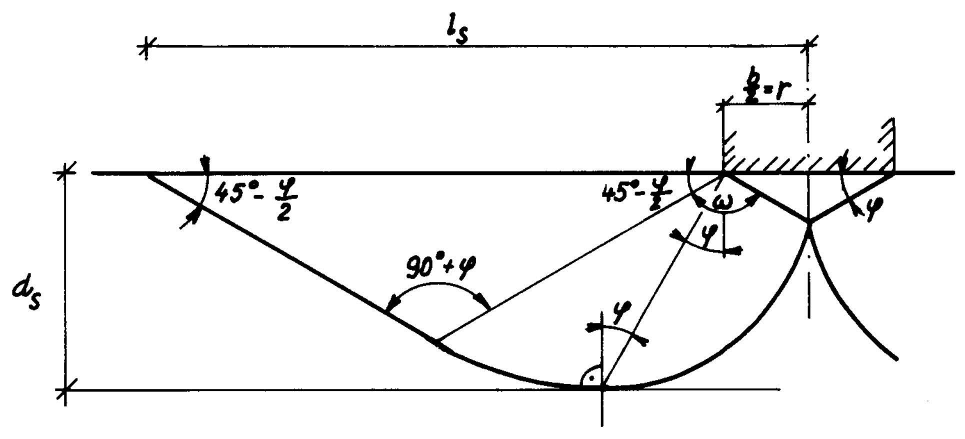
$$d_c = 1 + 0.1\sqrt{\frac{d}{b}}$$
$$d_d = 1 + 0.1\sqrt{\frac{d}{b}\sin 2\varphi_d}$$
$$d_b = 1$$

$$ic = id = ib = (1 - \tan\delta)^2$$

where:

b, l - footing dimensions,
t - footing thickness,
ϕ_d - design angel of friction,
c_d - design value of cohesion
d - depth of footing bottom
δ - angel of deflection of the resultant force

The standard based analysis is applicable only for homogeneous soil. If there is a nonhomogeneous soil under the footing, then the inserted profile is transformed into a homogeneous soil based on a selected slip surface, along which the sliding of footing is assumed.



Obr. 5.5 Slip surface after Terzhaghi

Two types of slip surfaces are available in the program:

- after Terzhaghi (**Fig. 5.5**)
- after Prandtl (**Fig. 5.6**)

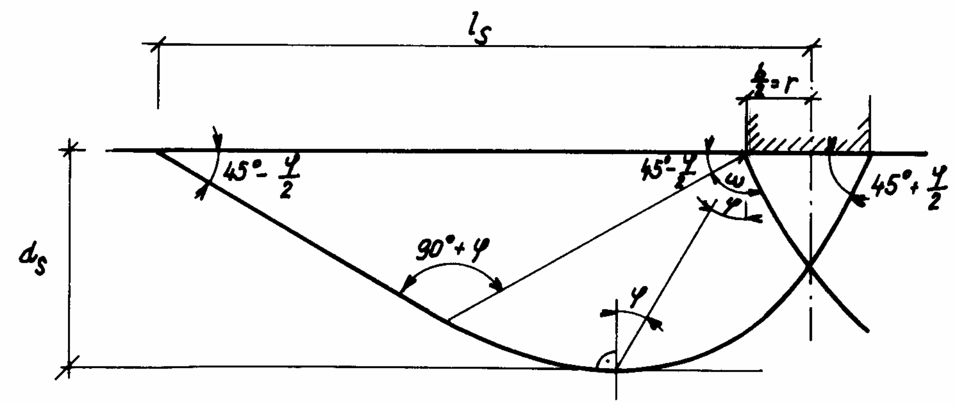


Fig. 5.6 Slip surface after Prandtl

Computation of substitute values of φ (angle of internal friction), c (cohesion), γ (bulk weight under footing) is evident from **Fig. 5.7** and given expressions. The bulk weight of soil above footing is the obtained in the similar manner. To compute the bearing capacity of spread footing with sand-gravel cushion you proceed in the similar way.

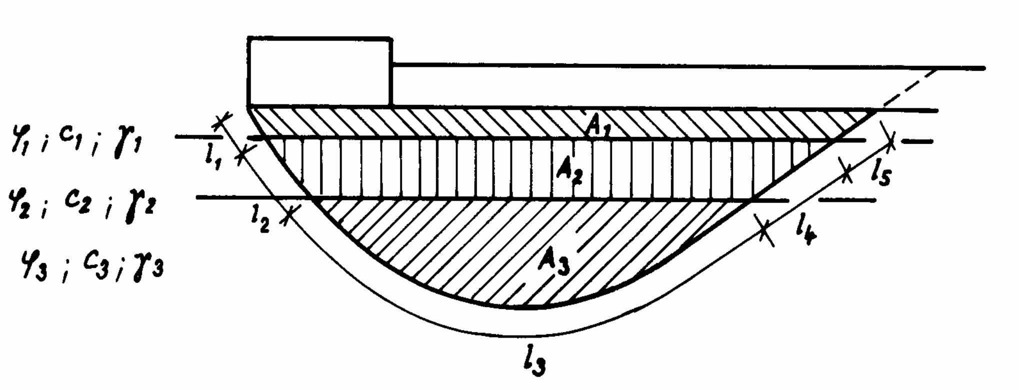


Fig. 5.7 Computation of substitute values

$$\varphi = \frac{\varphi_1(l_1 + l_5) + \varphi_2(l_2 + l_4) + \varphi_3 l_3)}{\sum_{i=1}^{5} l_i}$$

$$c = \frac{c_1(l_1 + l_5) + c_2(l_2 + l_4) + c_3 l_3)}{\sum_{i=1}^{5} l_i}$$

$$\gamma = \frac{\gamma_1 A_1 + \gamma_2 A_2 + \gamma_3 A_3}{\sum_{i=1}^{3} A_i}$$

If the slip surface intersects a rock, then the program cannot determine average characteristics of a body. Such a case requires modeling of rock as a soil and put in the angle of internal friction and cohesion. The next option allows a direct input of average characteristics of a body with respect to possible activation of slip surface. Surcharges in the vicinity of footing are not, in the analysis according to 1st LS, taken into account.

5.3.2 Analysis 1st LS – Horizontal bearing capacity

The horizontal bearing capacity of foundation soil is verified using the following formula

$$R_{dh} A_{ef} < H_{de}$$

where:

$$R_{dh} A_{ef} = Q \tan \varphi_d + c_d A_{ef} + S_{pd}$$

$$H_{de} = \sqrt{H_x^2 + H_y^2}$$

The analysis depends on the design angle of internal friction (ϕ_d) below the footing bottom, the design value of cohesion (c_d) below the footing bottom and the design value of earth resistance (S_{pd}). If the soil-footing frictional angle and the soil-footing cohesion are less than values of the soil under footing, then it is necessary to use those values.

The design values are derived from the standard ones in the following way:

$$\varphi_d = \frac{\varphi}{\gamma_{m\phi}}$$ where for $\phi < 12^o$ $\gamma_{m\varphi} = 1,5$

for $\phi > 12^o$ $\gamma_{m\varphi} = \frac{\varphi}{\varphi - 4°}$

$$c_d = \frac{c}{\gamma_{mc}}$$ where

$\gamma_{mc} = 2,0$

$$S_{pd} = \frac{S_p}{\gamma_r}$$ where for passive pressure $\gamma_r = 1,5$

for pressure at rest $\gamma_r = 1,3$

The earth resistance is assumed as displayed in **Fig. 5.8**.

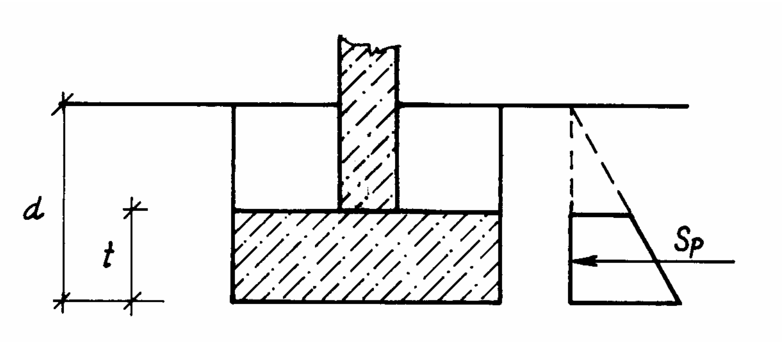


Fig. 5.8 Earth resistance

The lateral pressure attains the form

$$K_r = 1 - \sin\varphi \qquad \sigma_x = \sigma_x K_r$$

$$K_p = \tan^2(45 + \frac{\varphi}{2}) \qquad \sigma_x = \sigma_x K_p + 2c\sqrt{K_p}$$

The passive pressure can be considered providing the deformation needed for its activation do not cause unallowable stresses or deformations in upper structure.

5.3.3 Analysis 2nd LS – settlement and rotation

The service loading is used when performing the 2nd LS analysis. The final settlement of a footing receives the form

$$s = \sum_{i=1}^{n} \frac{(\sigma_{z,i} - m_i \sigma_{or,i}) h_i}{E_{oed,i}}$$

where:

$\sigma_{z,i}$ - vertical stress component below a given point due to surcharge σ_{ol} in the footing bottom in the center of the i^{th} layer,

$\sigma_{or,i}$ - original geostatic stress in the i^{th} layer,

m_i - correction coefficient of surcharge (of structural strength of the i^{th} layer),

h_i - thickness of the i^{th} layer,

$E_{oed,i}$ - oedometric modulus of the i^{th} layer – evaluation from the deformation modulus is carried out according to

$$E_{oed} = \frac{1}{1 - \frac{2\nu^2}{1-\nu}} E_{def}$$

The earth profile below footing is in the program subdivided into fifty layers of a different thickness depending on depth under the footing bottom (*10times 5cm, 10times 10cm, 10times 25cm, 10times 50cm, 5times 200cm* up to depth *250m*).

The stress in the footing bottom is assumed as displayed in **Fig. 5.9**.

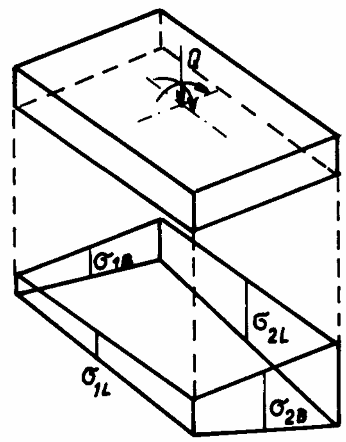


Fig. 5.9 Stress in the footing bottom

where:

$$\sigma_{b_{1,2}} = \frac{Q}{lb} \pm \frac{Qe_b}{W_b} \qquad e_b = \frac{M_x + H_y t + Np_x}{Q}$$

$$W_b = \frac{1}{6} lb^2$$

$$\sigma_{l_{1,2}} = \frac{Q}{lb} \pm \frac{Qe_l}{W_l} \qquad e_l = \frac{-M_y + H_x t + Np_y}{Q}$$

$$W_l = \frac{1}{6} lb^2$$

where:

t - thickness of spread footing [m],

N - normal force in a column for eccentric spread footing [kN],

p_x, p_y - column axis offset from the footing center [m]

If the stresses in some points are negative, then the tension is excluded from the analysis and substitute dimensions *b.l.* are used. Before proceeding with evaluation of stresses due to surcharge, the stresses in the footing bottom are reduced by geostatic stress such that

$$\sigma_{ol} = \max(\sigma_{ol} - \sigma_{or,sp};0)$$

There are three possibilities in the program to treat the geostatic stress. The geostatic stress is computed from the original ground providing the footing bottom has been uncovered in the open cut for less than the time needed for bulking and subsequent loss of stress state. Under the same conditions, the geostatic stress can be derived also from the finished grade or it can be excluded at all.

The actual stress distribution at points under the footing (**Figs. 5.10** and **5.12**) is determined by combining stress diagrams due to surcharges in the footing bottom using diagrams, which correspond to basic surcharges:

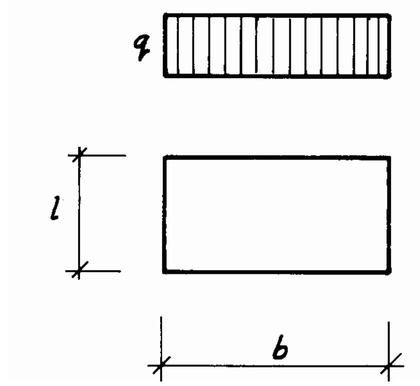


Fig. 5.10 Scheme of loading

$$\sigma_z = \frac{f}{2\pi}\left[arctg \frac{l.b}{z\sqrt{l^2+b^2+z^2}} + \frac{l.b.z}{z\sqrt{l^2+b^2+z^2}} \left(\frac{1}{l^2+z^2} + \frac{1}{b^2+z^2}\right)\right]$$

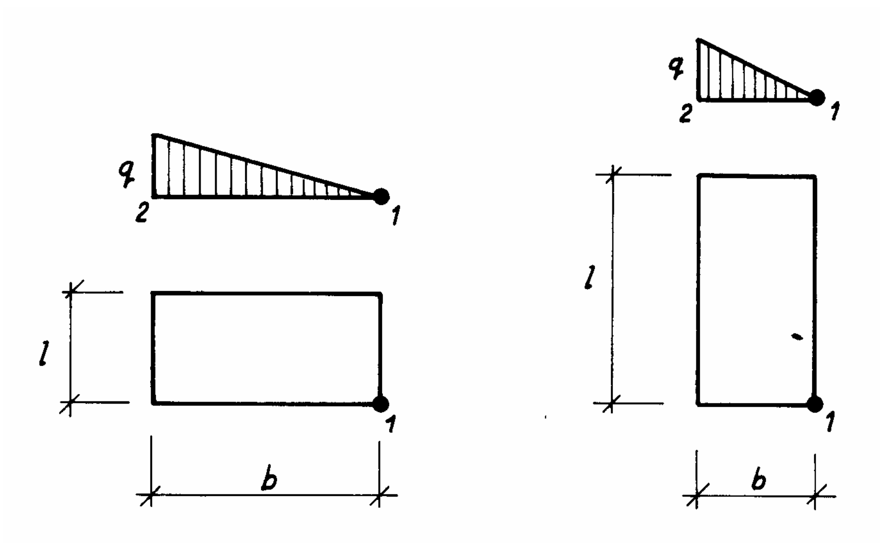


Fig. 5.11 Scheme of loading

$$\sigma_{z_1} = \frac{q}{2\pi}\left[\frac{l.b.z}{R(z^2+b^2)} + \frac{l.z}{b.R}\,\frac{R-\sqrt{l^2+z^2}}{\sqrt{l^2+z^2}}\right] \qquad R = \sqrt{l^2+b^2+z^2}$$

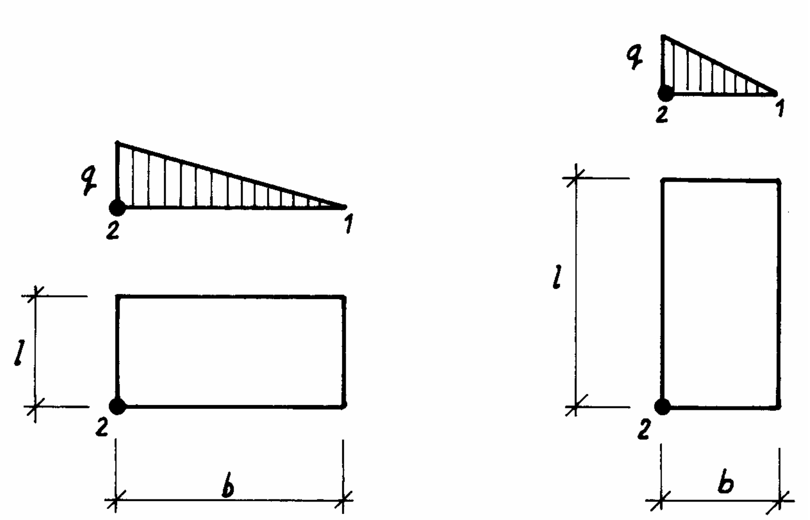


Fig. 5.12 Scheme of loading

The stress σ_z is reduced by the structural strength m.σ_{or}. The geostatic stress is considered from the original ground, since the structural strength is the original property of a soil body.

$$\sigma_{z_2} = \frac{}{2\pi}(arctg\frac{}{z.R} + \frac{}{l^2+z^2}\frac{\sqrt{}}{b})$$

The surcharge in the vicinity of footing increases the stresses as follows (**Fig. 5.13**):

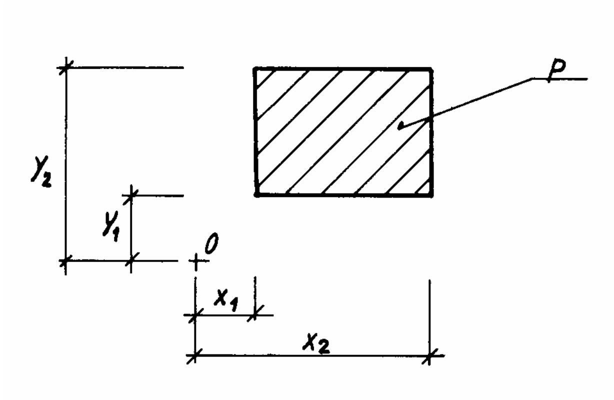


Fig. 5.13 Surcharge taken with respect to point O

$$\Delta\sigma_z = \frac{p}{2\pi}\left(\frac{x_2 z s_2}{y_2 s_{2x}^2} + \frac{x_2 z^3}{y_2 s_{2y}^2 s_2} - \frac{x_2 z s_3}{y_1 s_{2x}^2} + \frac{x_2 z^3}{y_1 s_{1y}^2 s_3} - \frac{x_1 z s_4}{y_2 s_{1x}^2} + \frac{x_2 z^3}{y_2 s_{2y}^2 s_4} + \frac{x_1 z s_2}{y_1 s_{1x}^2} - \frac{x_1 z^3}{y_1 s_{2y}^2 s_1} + \right.$$

$$\left. + arctg\frac{x_2 y_2}{z s_2} - arctg\frac{x_2 y_1}{z s_3} - arctg\frac{x_1 y_2}{z s_4} + arctg\frac{x_1 y_1}{z s_1}\right)$$

Assuming sand-gravel cushion, the values of the deformation modulus Edef in individual layers are computed in the following way: (**Fig. 5.14**)

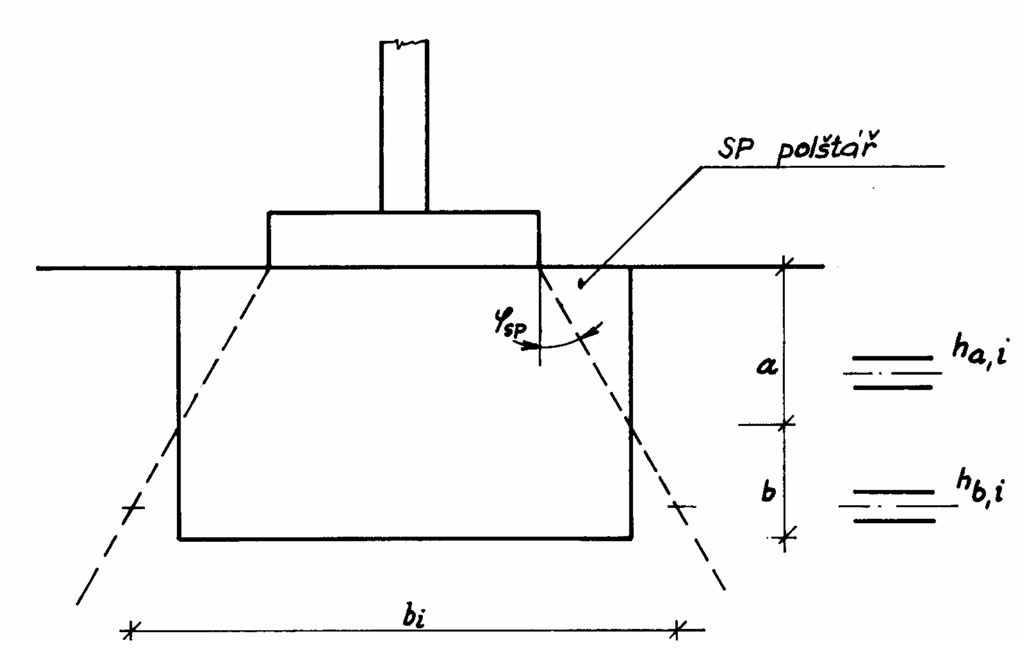


Fig. 5.14 Computation of E_{def} in sand-gravel cushion

Layer $h_{a,i}$ -

$$E_{def,i} = E_{def,sp}$$

Layer $h_{b,i}$ -

$$E_{def,i} = \frac{(A_i - A_{sp})E_{def,sp} + A_i E_{def,sp}}{A_i}$$

$$A_i = b_i l_i$$

where

- A_{sp} - area of sand-gravel cushion
- φ_{sp} - angle of internal friction of sand-gravel cushion,
- $E_{def,sp}$ - modulus of deformation of sand-gravel cushion,
- $E_{def,i}$ - deformation modulus of soil in the layer $h_{b,i.}$

Computation of settlement is carried out assuming constant stress distribution in the footing bottom. This assumtion is fulfilled for so called compliant foundation having stiffness $k<1$. The stiffness is given by

$$k = \frac{E_{ec} t^3}{E_{def} l^3}$$

where:

- E_{def} - a weighted average of the deformation modulus up to depth of influence zone,
- E_{ec} - modulus of elasticity of footing,
- l - footing dimension in the direction of searched stiffness.

It the footing is stiff *(k>1)*, then the settlement is determined in so called characteristic point (distant from the axis of footing by 0.37times the footing length).

The footing rotation is found from the difference of settlements of centers of individual edges.

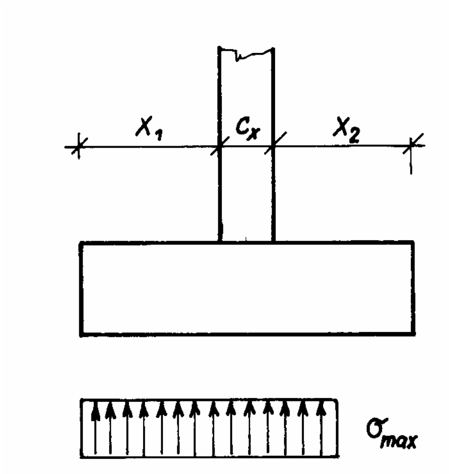
5.3.4 Dimensioning of reinforcement

The program enables dimensioning of longitudinal reinforcement of steel-concrete spread footing in both directions and shear reinforcement. The starting point is the stress distribution in the footing bottom according to the 1st LS (uniform stress). The reinforcement is found for the moment

$$M_d = \frac{\max(x_1; x_2) + 0{,}15 c_x}{2} \sigma . l_{ef}$$

where:

- l_{ef} - effective dimension of footing in transverse direction,
- σ - contact stress in the footing bottom.



Variables c_x, x_1, x_2 follow from Fig. 5.15.

Moment in the other direction is computed in the same way, except the dimensions are interchanged.
Verification of steel-concrete cross-section is carried out according to various standars – see Appendix 1-5.

5.4 Program „Beam on elastic foundation“

5.4.1 Computational model

The beam analysis is performed using the Finite element method. A beam element is enhanced by including the Winkler-Pasternak, model of subsoil.
The subsoil stiffness is characterized by two constants C_1 and C_2. The constant C_1 represents classical stiffness of an elastic spring in the direction of its axis. The constant C_2 can be imagined as shear connection between adjacent springs C_1. The model incorporates an influence of a shear depression along a beam as well as in front of the starting point and behind the end point of a beam. The subsoil stiffness is further affected by the loading width of a beam.
Unfortunately, constants C_1 and C_2 are not known apriory. Therefore, the program enables their evaluation. Their are found by using parameters, which are usually available, i.e., the deformation modulus, the Poisson number ν and a depth of influence zone h_d. The deformation modulus E_{def} can be obtained from standard experiments. The depth of influence zone identifies a depth in subsoil, to which the deformation due to beam loading is significant. Reasonable values of the depth of influence zone can be taken as *1.5* and *5.0* times the beam width.
The theoretical base of the Winkler-Pasternak model is a solution of a very flexible layer of constant thickness resting on an incompressible ground. Such an assumption, with reference to the soil structural strength, is in better agreement with reality than the theory of elastic half-spaces. It is also assumed that the layer does not deform in horizontal plane. The horizontal displacements $u(x,y,z)$, $v(x,y,z)$ are, therefore, neglected and only the vertical displacement $w(x,y,z)$ is taken into account. The solution of this problem is known as the Westergard solution, which provides a slightly stiffer response of a layer than the solution, which allows horizontal displacements.
The vertical displacements are approximated using the Kantorivic method, which results into an infinite system of partial differential equations. In particular, the vertical displacement $w(x,y,z)$ is assumed in the form $w(x,y,z)=\Sigma\, w_i(x,y).\psi_i(x,y,z)$. Functions $\psi_i(x,y,z)$ are called the base functions (recall the Fourier series method). Such a separation of variables reduces the problem dimension and turns the solution into a search for an infinite set of functions $w_i(x,y)$, which are no longer functions of three but rather two variables.

Realizing the diagonal dominance of the matrix coefficients and a quadratic reduction of loading with increasing index, it becomes clear that for reasonable accuracy of the solution it is sufficient to search only for a few first members of the series. Furthermore, a canonic form of the system of equations is attained when having constant material parameters. Each differential equation then becomes formally equivalent to partial differential equation describing the solution of the Winkler-Pasternak subsoil.
Note that in case of influence zones up to depth of three times the beam width, it is sufficient to use only one member of the series. The solution can be improved by a suitable selection of constants C_1 and C_2. To that end, we recommend the user to utilize the option for their determination offered by the program, is particularly suitable for strip foundations as the method ensures the same compliance matrix of a stiff infinite beam for both the Winkler-Pasternak and more accurate Westergard solutions of an elastic layer.

5.4.2 Input of subsoil parameters

Subsoil parameters and solution procedure is selected in the „**Subsoil**“ dialog window, **Fig. 5.16**.

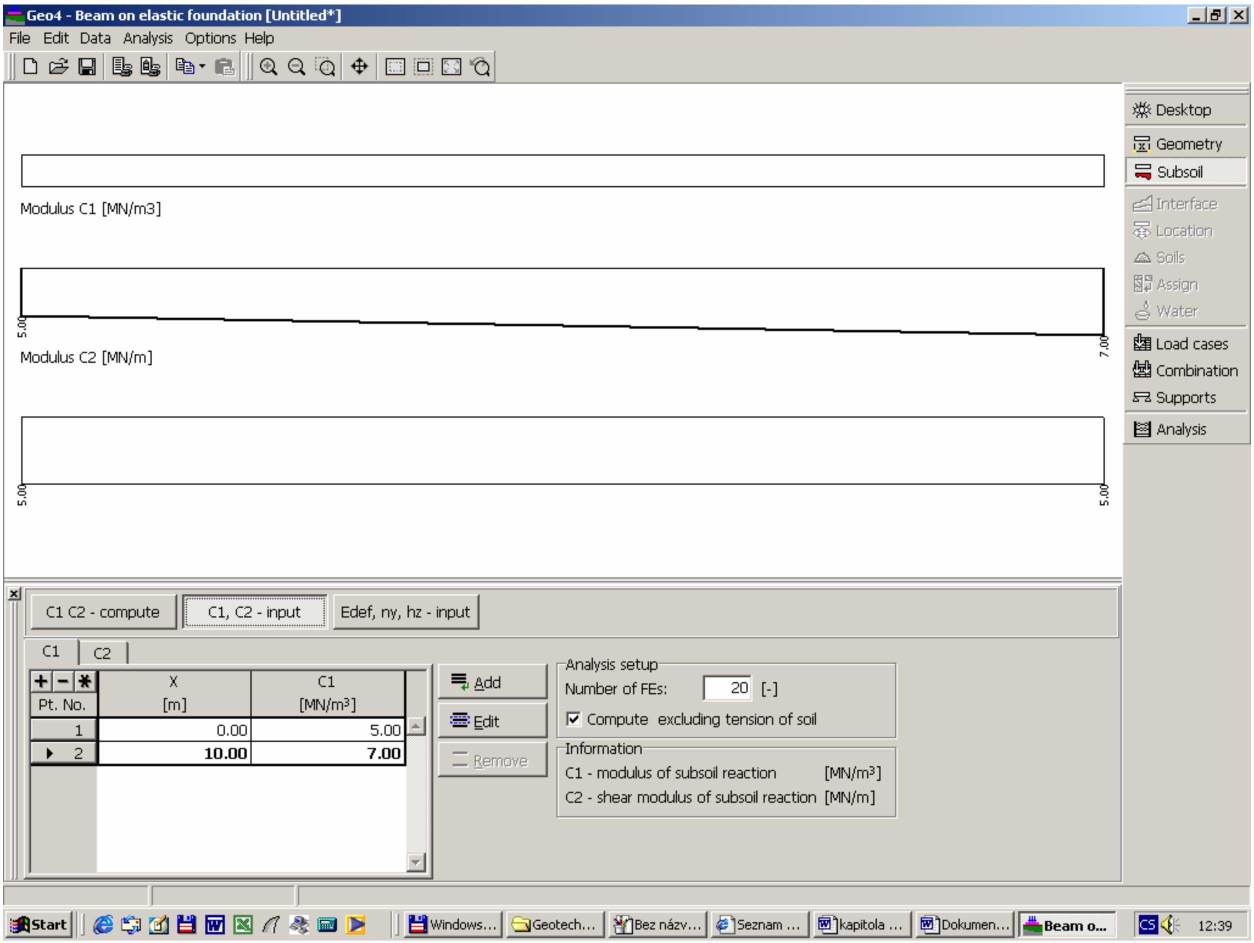


Fig. 5.16 Dialog window „Subsoil“

In this dialog window you may select the method for determination of constants C_1 and C_2 using one of the following variants:

1)Direct input of C_1,C_2

This option allows a direct input of constants C_1 and C_2 along the beam. In this window, you also insert the number of finite elements and select whether the tension should be excluded during the analysis. The latter case leads to a nonlinear an analysis – modules C_1 and C_2 in the locations, where the beam deforms above the subsoil, are set to zero. This essentially corresponds much better to the real behavior of soils. If you wish to analyze a part of the beam with no subsoil you simply set C_1, C_2 equal to zero.

2)Input of distribution of E_{def}, υ and h_z along a beam

In this case, you are required to put in distributions of the deformation modulus E_{def}, the Poisson number υ and a depth of influence zone h_z along a beam. The program then exploits these parameters to compute values C_1 and C_2. In the locations where there is no subsoil you simply set $E_{def} = 0$.

3)Modules of subsoil reactions C_1, C_2 are computed from geological profile

This option allows the user to select a geological profile (buttons Interface, Location, Soils, Assignment, Water) and load characteristics (button Combinations) – the program itself then computes the values of C_1, C_2.

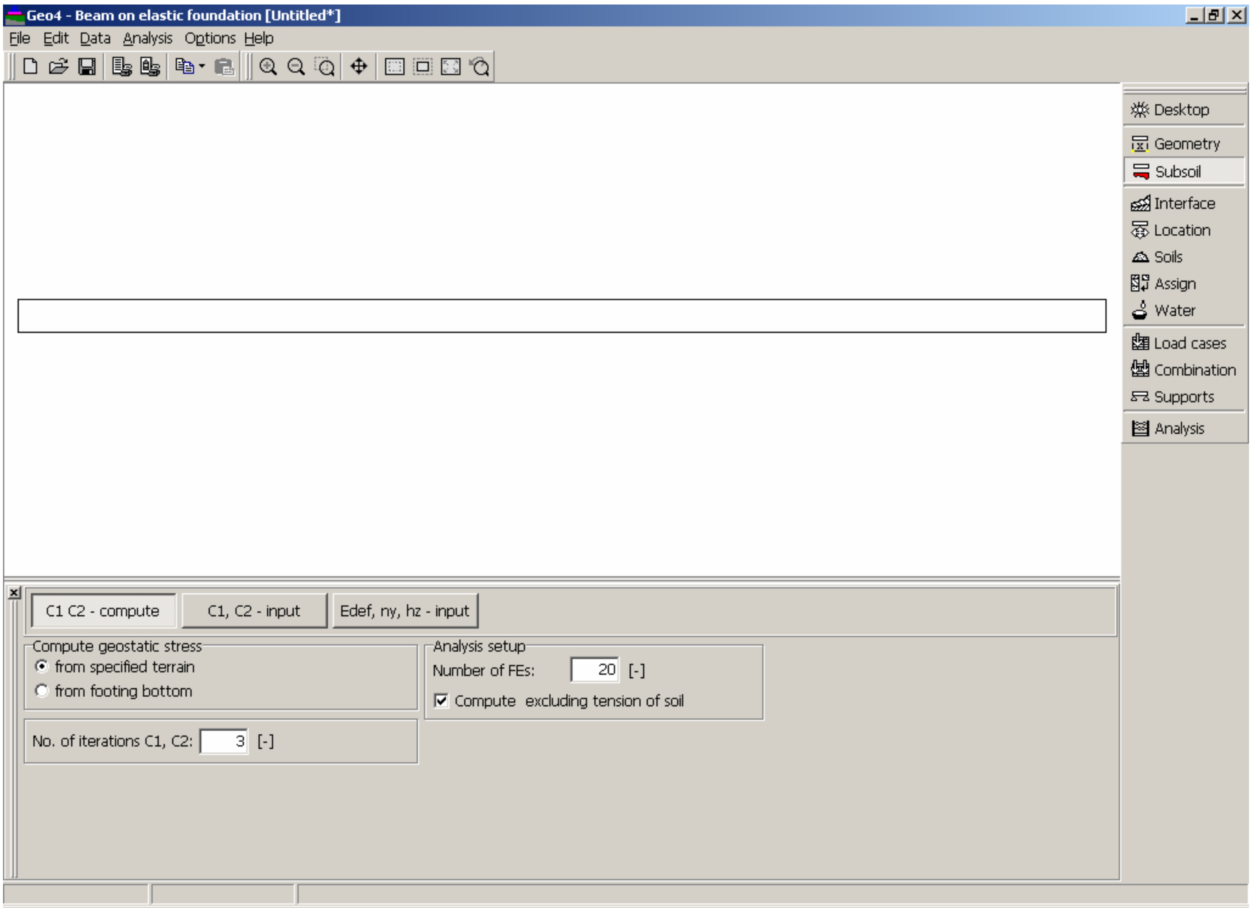


Fig. 5.17 Dialog window „Subsoil“

To determine C_1 and C_2 the program proceeds as follows:

- For a given combination (for input use the "**Combination**" dialog window), the program computes deformations and contact stress using $C_1=10MN/m^3$ and $C_2=5MN/m$,
- Then it determines the geostatic stress state in a soil body (the "**Subsoil**" dialog window, recall the possibility to compute the stresses from the original ground or from the footing bottom),

- The depth of influence zone and the distribution of stresses (contact stresses) due to beam surcharge is computed next together with weighted average characteristics E_{def} and υ,
- The new values of C_1 and C_2 are determined,
- The beam is then analyzed with new values of C_1 and C_2 leading to new values of contact stress,
- The procedure is repeated until the selected number of iterations is exhausted, three iterations are usually sufficient,
- Using the final values of C_1 and C_2, the program then carries out a complete analysis of all load cases and combinations.

The decisive input information influencing the depth of deformation zone and thus the values of C_1, C_2 is the decisive combination used to compute values C_1 and C_2. This combination should represent all permanent and long-time surcharges, without considering the live loading. It is also recommended to consider a service loading as oppose to the extreme one.
If there is no subsoil along a part of a beam, then it is sufficient to consider the original ground at the same level as the footing bottom – the program then automatically assumes no subsoil in these locations.

5.4.3 Load cases and combinations

An arbitrary number of load cases and combinations can be put in. To that end, use the "**Load cases**" dialog window.

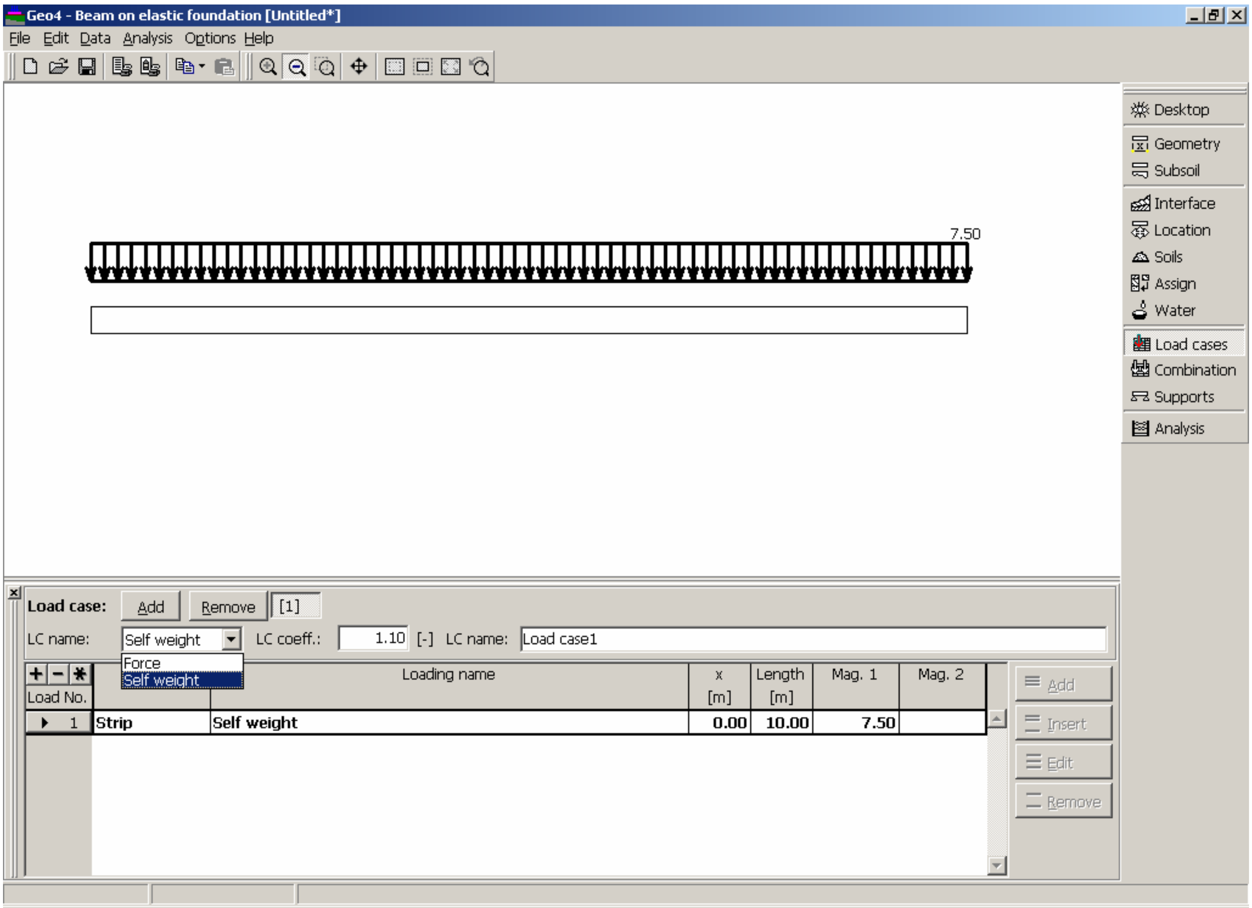


Fig. 5.19 Dialog window „Load cases"

Two type of load cases are defined in the program:

- **Self weight** – this load case is generated automatically based on a given geometry. Each change of geometry is automatically reflected in the program by modifying the load case.
- **Force** - serves to assign an arbitrary number of loads caused by a concentrated force, moment and by continuous uniform or trapezoidal loading.

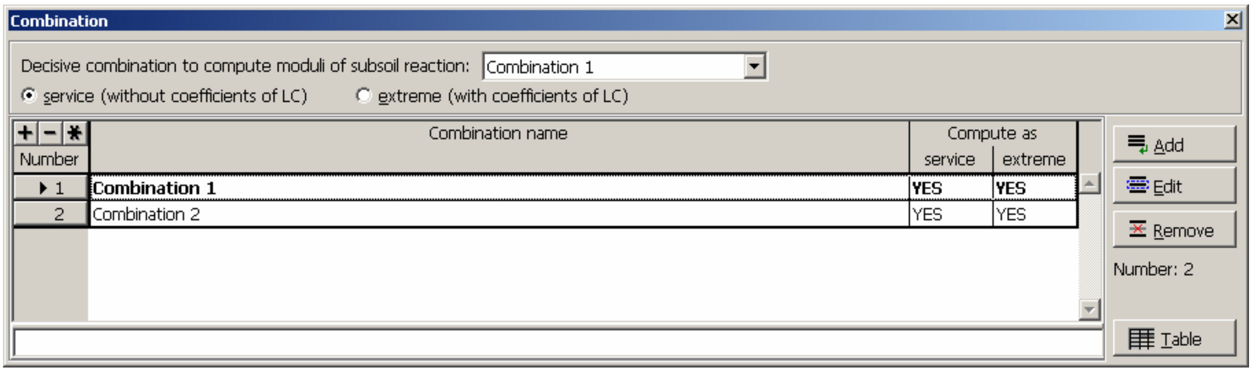


Fig. 5.20 Combinations of load cases

An "**LC.Coeff**" can be assigned to each load case – a design coefficient to multiply loading in extreme combinations. For service combinations, the coefficient is set to one.

An arbitrary number of combinations can be introduced in the "**Combinations**" dialog window. This window also serves to put in the decisive combination for computation of C_1 and C_2.
Each combination is defined by the selected load cases and by the coefficient of combinations. The type of combinations (service, extreme), which determines whether to multiply the load cases by the coefficient of combination, must also be specified.

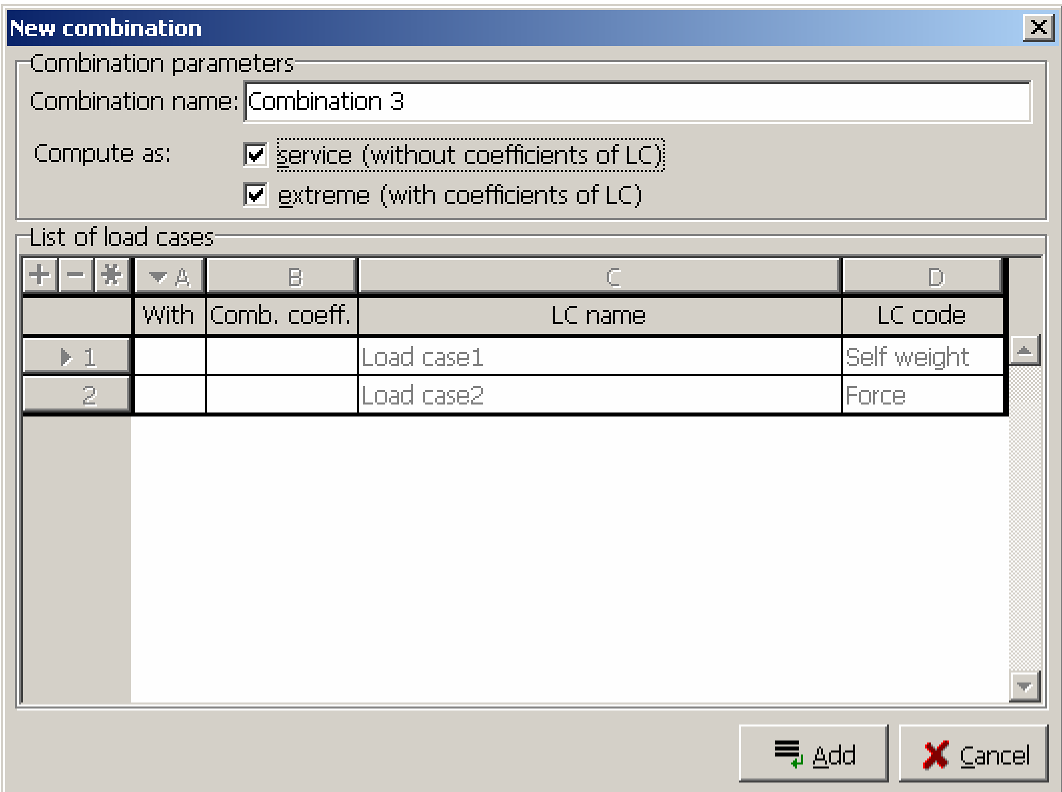


Fig. 5.20 Combinations of load cases input

Extreme combinations are used when verifying the structure according to the 1st limit state (strength), service combinations then apply to the 2nd limit state (deformation).
Since the beam analysis is in most cases nonlinear, it is possible to plot results of combinations only or envelopes of extreme and service combinations, respectively.

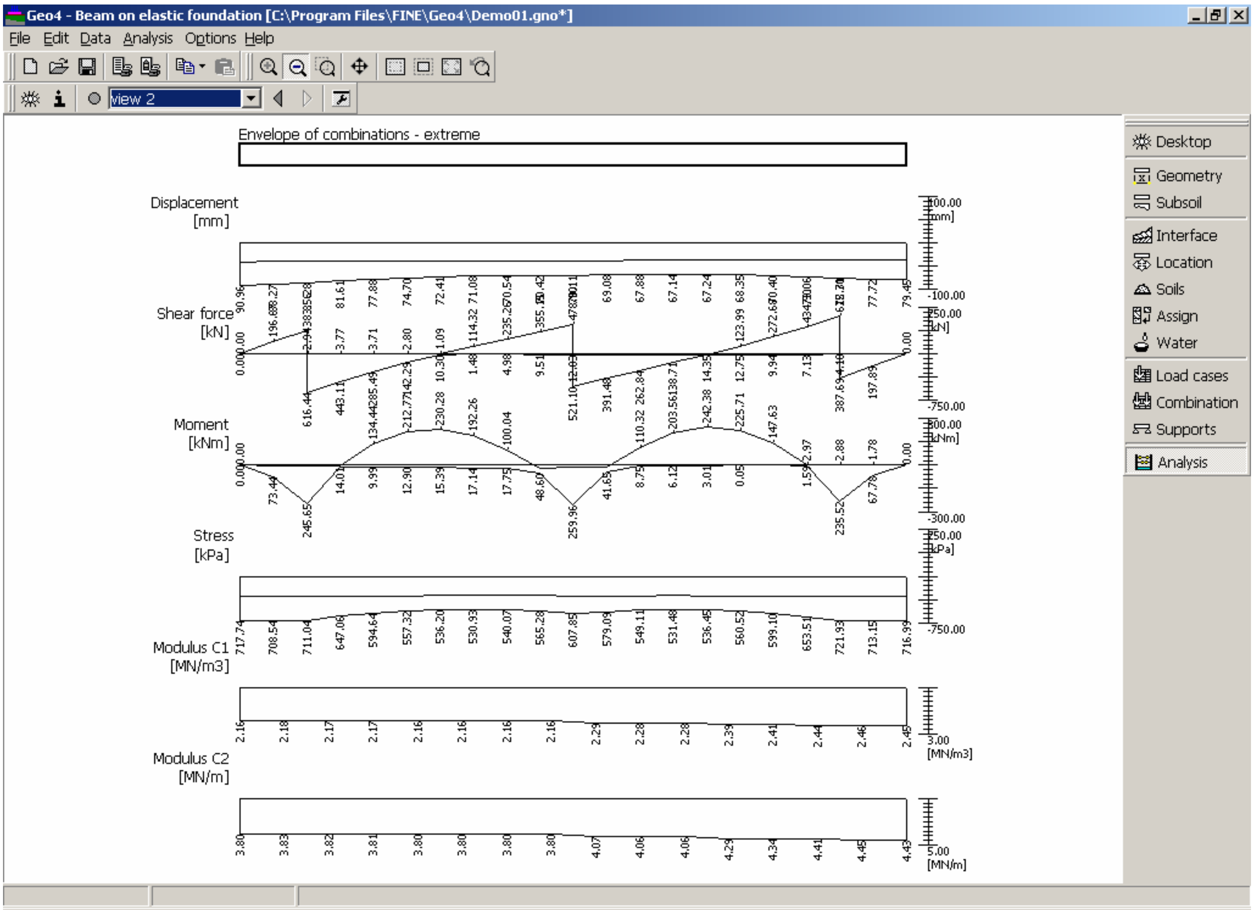


Fig. 5.21 Analysis Results

5.5 Pile

5.5.1 Vertical bearing capacity.

The program serves to determine the vertical bearing capacity of a pile – derivation of the limit-loading curve. The main advantage of the program is accessibility of soil and rock parameters, respectively. The program requires the knowledge of the angle of internal friction, cohesion, bulk weight and the modulus of deformation. As a result, the program provides the limit-loading curve, where the user may determine a limiting deformation.

5.5.1.1 Theoretical grounds

The program is based on a semi-analytical solution. The pile is modeled using standard beam elements, while the behavior of surrounding soil is described by known fundamental solution of a layered soil structure. In case of a semi-infinite body the solution is known as Mindlin's solution. The solution is improved by incorporating the structural strength of soil in a similar way as used for modeling settlement of spread footings. The influence of underground water is incorporated using the Archimedes law. The shear behavior of pile-soil interface is described using the elastic-plastic material model with the Mohr-Coulomb yield condition. The normal stress is found from the geostatic stress and the stress of soil (concrete mixture) at rest. The unknown kinematically admissible displacement follows from the equilibrium condition in the vertical direction. The material nonlinearity is reflected by using the variable secant modules.

5.5.1.2 Solution procedure

1) The pile structure is modeled as a member composed of several elements. The number of elements is then determined from the length to pile diameter ratio, for which we derived the solution for evaluation of shear stiffness of soil surrounding the pile. The element length should be about 2.5 times larger then the pile diameter. Nevertheless, at least ten elements are used to avoid rugged results. The shear stiffness evaluation, however, is still based on ratio l/d = 2,5.

2) Each element is supported at its bottom end by a spring. The spring stiffness is derived by employing parameters of elastic subsoil C_1, C_2 and modified Bessel's functions. Values of C_1 and C_2 are determined from parameters E_{def} and ν of a particular soil. The depth of influence zone, which affects values of C_1 a C_2, is variable and changes with pile deformation (settlement). For zero deformation it is set equal to 1 times the pile diameter, whereas at the onset of strength failure at the interface equals 2.5 times the pile diameter.
Recall that reliability of values C_1 and C_2 depends on determination of the deformation modulus. Here, one should be cautious when accepting values provided by standards.
Note that longer piles should be considered as deep foundations and the soil at the pile heel is certainly stiffer than recommended by the standard for spread footings. In any case, the most reliable results are provided by laboratory measurements of the deformation modulus on samples taken at individual depths.

3) For each pile element we determine the limit value of shear force transmitted by the pile skin. To proceed, we first determine the geostatic stress at a given location using

$$\sigma_z = \sum \gamma h$$

and the limit shear stress

$$\tau = \sigma_z k.tg\varphi + c$$

where (k) is the coefficient of increase of the limit skin shear (k) due to technology. Its default value is one. There are no recommendations for its magnitude provided by the standard. Its selection depends solely on the user practical experience. Experimental measurements in situ on real piles suggest the value of k around 1.5, but theoretically, this value can be even less then one.
The limit shear force is then found from

$$T_{\lim} = 2\pi r l \tau$$

4) The spring stiffness under the pile heel is provided by

$$K_p = \pi r^2 C_1$$

where C_1 is the parameter of subsoil at the heel point discussed in paragraph 2.

5) The pile is loaded incrementally with the force applied at the top. Forces developed in individual springs of all elements are computed at each increment. These forces are then compared with the allowable shear force T_{lim} for a given element. If the spring force exceeds T_{lim}, then the stiffness of this spring is reduced such that for a given deformation the spring force equals $T_{lim.}$

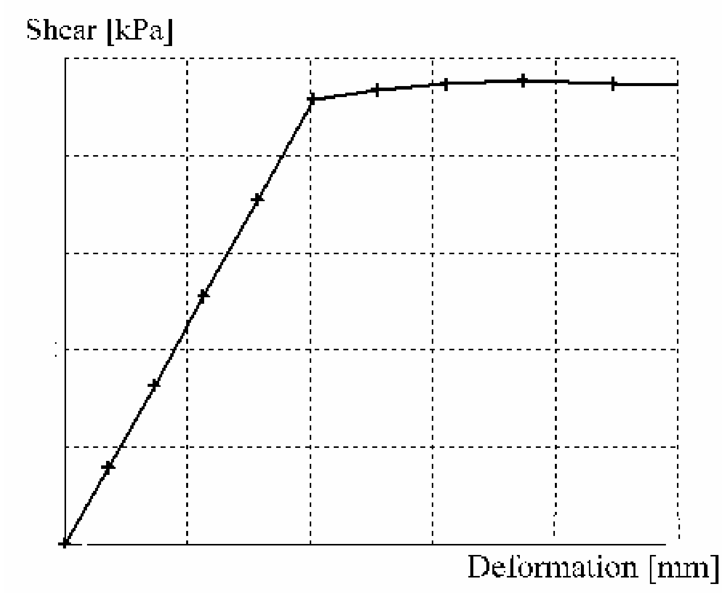


Fig. 5.22 Deformation versus jacket friction

This solution increment is then repeated and the reduced force is redistributed into the remaining springs. Each load increment is iterated until the force developed in every spring is less than T_{lim}. Gradual softening of individual springs then leads to nonlinear loading curve. Evidently, at certain load level all springs will lose capability of increasing its force and the pile will start to penetrate into a soil in a linear manner supported only by the heel spring. Note that there is no restriction on the magnitude of force developed in the heel spring.

6) The program provides a limit-loading curve of a vertically loaded pile. It is derived for the maximum allowable settlement equal to 25 mm. This default value, however, can be changed arbitrarily.

Apart from the limit-loading curve, the program also delivers the distributions of normal shear forces along a pile. The normal force decreases from the top to the bottom as the shear for picks up and thus the shear skin force must decrease from the bottom to the top. Both variables are plotted in values relative to the applied vertical load.

The program also enables to track the shear skin force in arbitrary depth as a function of deformation (settlement) of a given pile point. The variation is linear until the spring force exceeds T_{lim}. Then the spring stiffness is gradually reduced and the curve tends to horizontal threshold.

5.5.1.3 Negative skin friction

Negative skin friction means reduction of the shear force transmitted by the pile skin, due to depression of the soil surrounding the pile. The soil settlement draws the pile down and reduces its bearing capacity.
The soil settlement w and depth of influence zone h are input values for negative skin friction analysis. The value w should be measured in the distance 3d from the pile, where d means diameter of the pile. The value h is depth of influence of the soil settlement. The soil is not depressed below the level h.

The negative skin friction analysis is added to the T_{lim} forces determination. The soil deformation changed linearly, from the value w at the terrain surface to the zero value in the depth h. For every pile element and its spring we are able to determine a value of soil settlement. This deformation induces a force in each spring, dependently on the spring stiffness. We subtract these forces from the T_{lim} values, and so we reduce the skin shear bearing capacity.

It is evident, that for high values of the w and/or h parameters, any of the T_{lim} values could be reduced to zero. Extremely, when all forces T_{lim} are reduced to zero, the pile would be supported only by the heel spring.

5.5.2 Horizontal bearing capacity, dimensioning

Horizontally loaded pile is analyzed as a beam on elastic (Winkler) foundation using the finite element method. Therefore, the required soil parameters are the modulus of subsoil reaction k_h (units - N/m^3). Default setup assumes subdivision of a pile into 30 elements. Modules of subsoil reaction, magnitudes of internal forces and deformations are then evaluated at these elements. In particular, the determination of modules of subsoil reaction creates major difficulty in the analysis of bearing capacity of horizontally loaded pile. To that end, the program offers three variants of their determination:

1) User defined distribution of the modulus of subsoil reaction

- the user assigns the modulus of subsoil reaction k_h [MN/m^3]

2) Constant distribution of the modulus of subsoil reaction

- the modulus of the i^{th} layer is provided by the following formula

$$k_h = \frac{3E_{def}}{2r}$$

where:

E_{def} - deformation modulus of a pile

r - a reduced pile width given by

$$r = d + d \tan \frac{\beta}{\gamma}$$

where:

d - pile diameter

β - angle of influence assigned by the user – its magnitude depends on the angle of internal friction φ and should be taken from the interval $\varphi/4$-φ

γ - design coefficient, the value of *1,4* is considered

3) Linear distribution of the modulus of subsoil reaction

- the modulus of subsoil reaction at a depth z is determined by

$$k_h = k(0{,}308 + 1{,}584\frac{d}{l})\frac{z}{r.l}$$

where:

r - a reduced pile width

d - pile diameter

l - pile length

k - user defined soil parameter, its approximate values are listed in a relevant help of a given program

Dimensioning of a steel-concrete pile is performed for loading due to normal force and bending moment using the method of limit deformation.

Verification of the cross-sections is carried out according to various standards (EC, IS, PN, CSN.). The dimensioning using the various standards are describes in the guide appendix.