STABILITY ANALYSIS AND FOUNDATION DESIGN USING GEO5 SOFTWARE

Project Report submitted in partial fulfillment of the degree of

Bachelor of Technology

In

Civil Engineering

Under the Supervision of

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By

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To

Department of Civil Engineering

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CERTIFICATE

This is to certify that project report entitled "STABILITY ANALYSIS AND FOUNDATION DESIGN USING GEO5 SOFTWARE", submitted by Shrey Chhabra (111657) and Deepankar Kanwar (111693) on partial fulfillment for the award of degree of Bachelor of Technology in Civil Engineering to Jaypee University of Information Technology, Waknaghat, Solan has been carried out under my supervision.

This work has not been submitted partially or fully to any other University or Institute for the award of this or any other degree or diploma.

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Abstract

With the development of technology and software’s it has become easy to solve difficult problems in every field which before, use to take a lot of time. The use of software’s in the field of civil engineering has grown since the last decade. It has now become easy to analyze or design using different software’s such as Staad Pro, Auto Cad, PLAXIS and many more.

Geo5 is one such software use to solve many geotechnical problems like analyzing and design of slope, design of retaining walls, settlement analysis, foundation design and much more.

In this project we have learned two basic modules of it which are Slope Stability and Foundation Design and then used them for solving problems.

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1.1 Introduction

Slopes may be natural or manmade or earth dam. Every slope has forces acting on it that tend to disturb its stability. The main force is the self-weight of soil mass forming the slope, but seepage, seismic activity and external loads are a=lso disturbing forces. In a stable slope, resisting force due to shear strength are larger than disturbing force. Slope failure is related to the following reasons: soil properties or soil type of slope, geometry of slope, weight, water content (one of the most aggressive factor reducing shearing strength of slope), tension cracks and vibrations due to earthquakes.

Conventional limit-equilibrium techniques i.e. they evaluate the slope as if it were about to fail and determine the resulting shear stresses along the failure surface, are the most commonly used analysis methods. Excellent commercial software's like Geo5 [9], PLAXIS, Z-soil, have made a powerful viable alternative to the assistance of the geotechnical engineer.

Foundations are designed to have an adequate load capacity depending on the type of subsoil supporting the foundation by a geotechnical engineer and the footing itself may be designed structurally by a structural engineer. The primary design concerns are settlement and bearing capacity. When considering settlement, total settlement and differential settlement is normally considered.

The software aids in the design of foundation and give results on the basis of bearing capacity of soil, settlement of foundation and dimensioning of concrete and reinforcement.

1.2 SCOPE AND OBJECTIVES

Stability of slopes, natural and manmade, is particularly important for any hill road. Disturbance to slope can occur due to erosion caused by rain-fall and run-off and consequent slides. During monsoons the road network in hill roads experiences slips, erosions and major and minor landslides at many places. Hence, Slope stability and erosion control are therefore vital for control and prevention of landslides/slips.

Our objective of this project is to learn Geo5 and analyze any given slope using different methods and then asses which one is better. Also we have to stabilize the slope if found unstable using different options available in the software. Also we have to learn the foundation design suite of the software and design different foundations using it.

Chapter 2: Literature

2.1 Stresses in a Soil Body

2.1.1 Geostatic Stress, Uplift Pressure

Stress analysis is based on the existence of soil layers specified by the user during input. 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:

$$\sigma_i = \sum h_i.\gamma_i$$

where:	h_i	-	thickness of the i^{th} layer
	γ_i	-	unit weight of soil

If the layer is found below the **ground water table**, the unit weight of soil below the water table is specified with the help of inputted parameters of the soil as follows:

- "**Standard**" from expression:

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

where:	γ_{sat}	-	saturated unit weight of soil
	γ_w	-	unit weight of water

- "**Compute from porosity**" from expression:

$$\gamma_{su} = (1-n).(\gamma_s - \gamma_w)$$

where:	n	-	porosity
	γ_s	-	specific weight of soil
	γ_w	-	unit weight of water

$$\gamma_s = \frac{G_d}{V - V_p}$$

where:	V	-	volume of soil
	V_p	-	volume of voids
	G_d	-	weight of dry soil

Unit weight of water is assumed in the program equal to *10 kN/m³* or *0,00625 ksi*.

Assuming inclined ground behind the structure ($\beta \neq 0$) and layered subsoil the angle β, when computingthe coefficient of earth pressure$K$, is reduced in the i^{th} layer using the following expression:

$$tg\beta_i = \frac{\gamma}{\gamma_i} tg\beta$$

where:	γ	-	unit weight of the soil in the first layer under ground
	γ_i	-	unit weight of the soil in the i^{th} layer under ground
	β	-	slope inclination behind the structure

2.1.2 Effective/Total Stress in a Soil

Vertical normal stress σ_z is defined as:

$$\sigma_z = \gamma_{ef} .z + \gamma_w .z$$

where:	σ_z	-	vertical normal total stress
	γ_{ef}	-	submerged unit weight of soil
	z	-	depth bellow the ground surface
	γ_w	-	unit weight of water

This expression in its generalized form describes so called concept of effective stress:

$$\sigma_z = \sigma_{ef} + u$$

where:	σ	-	total stress (overall)
	σ_{ef}	-	effective stress (active)
	u	-	neutral stress (pore water pressure)

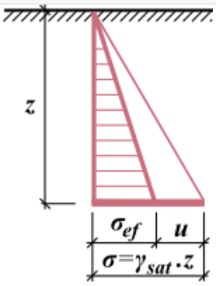


Fig.2.1 Total, effective and neutral stress in the soil

Effective stress concept is valid only for normal stress σ, since the shear stress τ is not transferred by the water so that it is effective. The total stress is determined using the basic tools of theoretical mechanics, the effective stress is then determined as a difference between the total stress and neutral (pore) pressure (i.e. always by calculation, it can never be measured). Pore pressures are determined using laboratory or in-situ testing or by calculation. To decide whether to use the total or effective stresses is no simple. The following table may provide some general recommendations valid for majority of cases. We should realize that the total stress depends on the way the soil is loaded by its self weight and external effects. As for the pore pressure we assume that for flowing pore water the pore equals to hydrodynamic pressure and to hydrostatic pressure otherwise. For partial saturated soils with higher degree of it is necessary to account for the fact that the pore pressure evolves both in water and air bubbles.

Assume conditions	**Drained layer**	**Undrained layer**
short – term	effective stress	total stress
long – term	effective stress	effective stress

In layered subsoil with different unit weight of soils in individual horizontal layers the vertical total stress is determined as a sum of weight of all layers above the investigated point and the pore pressure:

$$\sigma_z = \int_0^z \gamma dz + \gamma_w (z - d)$$

where:	σ_z	-	vertical normal total stress
	γ	-	unit weight of soil
			- unit weight of soil in natural state for soils above the GWT and dry layers
			- unit weight of soil below water in other cases
	d	-	depth of the ground water table below the ground surface
	z	-	depth bellow the ground surface
	γ_w	-	unit weight of water

2.1.3 Increment of Earth Pressure due to Surcharge

Earth pressure increment in a soil or rock body due to surcharge is computed using the theory of elastic subspace (Boussinesq).

Earth pressure increment in the point inside the soil or rock body due to an **infinite strip surcharge** is obtained from the following scheme:

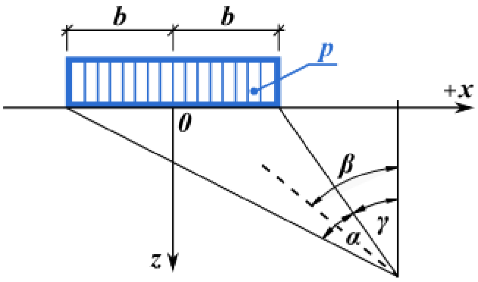


Fig2.2Computation of earth pressure due to infinite strip surcharge

$$\sigma_z = \frac{p}{\pi}(\alpha + \sin\alpha.\cos 2\beta)$$

$$\beta = \gamma + \frac{\alpha}{2}$$

A **trapezoidal surcharge** is automatically subdivided in the program into ten segments. Individual segments are treated as strip surcharges. The resulting earth pressure is a sum of partial surcharges from individual segments.

2.2 Theory of settlement

If the stress change in the soil or in the currently build earth structure, caused by ground surface surcharge, is known, it is possible to determine the soil deformation. The soil deformation is generally inclined and its vertical component is termed the settlement. In general, the settlement is non-stationary dependent on time, which means that it does not occur immediately after introducing the surcharge, but it rather depends on consolidation characteristics of a soil. Permeable, less compressible soils (sand, gravel) deform fast, while saturated, low permeability clayey soils experience gradual deformation called consolidation.

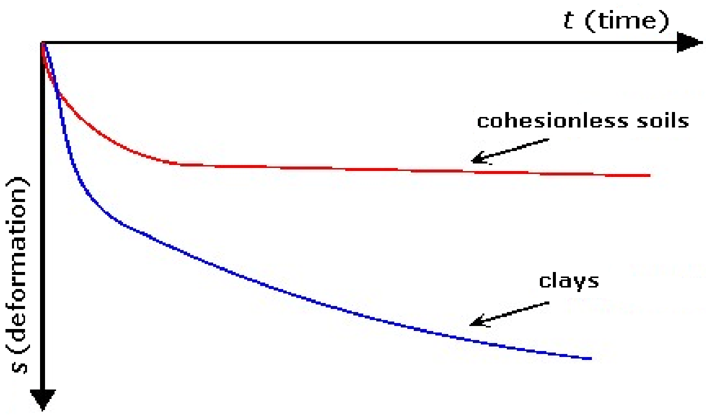


Fig.2.3Time dependent settlement of soils

Applied load yields settlement, which can be subdivided based on time dependent response into three separate components:

- Instantaneous settlement (initial)
- Primary settlement (consolidation)
- Secondary settlement (creep)

Instantaneous settlement

During instantaneous settlement the soil experiences only shear deformation resulting into change in shape without volumetric deformation. The loss of pore pressure in the soil is zero.

2.2.1 Primary settlement

This stage of soil deformation is characterized by skeleton deformation due to motion and compression of grains manifested by volume changes. If the pores are filled with water (particularly in case of low permeability soils), the water will be carried away from squeezed pores into locations with lower pressure (the soil will undergo consolidation). The consolidation primary settlement is therefore time dependent and is terminated by reaching zero pore pressure.

2.2.2 Secondary settlement

When the primary consolidation is over the skeleton deformation will no longer cause the change in pore pressure (theoretically at infinite time). With increasing pressure the grains may become so closely packed that they will start to deform themselves and the volumetric changes will continue - this is referred to as creep deformation of skeleton or secondary consolidation (settlement). Unlike the primary consolidation the secondary consolidation proceeds under constant effective stress. Particularly in case of soft plastic or squash soils the secondary consolidation should not be neglected - in case of overconsolidated soils it may represent app. 10% of the overall settlement, for normally consolidated soils even app. 20%.

2.3 Slope stability

Slope stability is the potential of soil covered slopes to withstand and undergo movement. Slope stability, or the lack thereof, rests upon the ability of a slope to resist stress excess to what is normally acceptable for the material property of the soil or rock inherent to the construction slope. Slope movements, such as translational or rotational slope failures occur when sheer stress exceeds sheer strength of the materials forming the slope (Gray and Leiser 1982). Factors contributing to high sheer stress include: lack of lateral support, excessive surcharges, lateral pressures and removal of underlying support. On the other hand, low sheer strength, due to inherently weak materials, soil weathering (swelling, shirking and cracking) and low inter-granular force due to seepage pressure, also contributes to slope instability.

The field of slope stability encompasses static and dynamic stability of slopes of earth and rock-fill dams, slopes of other types of embankments, excavated slopes, and natural slopes in soil and soft rock.Slope stability investigation, analysis (including modeling), and design mitigation is typically completed by geologists, engineering geologists, or geotechnical engineers. Geologists and engineering geologists can also use their knowledge of earth process and their ability to interpret surface geomorphology to determine relative slope stability based simply on site observations.

Factors of safety are generally used in evaluating slope stability. The factor of safety can be defined as the ratio of the total force available to resist sliding to the total force tending to induce sliding. A stable slope is considered to be in a condition where the resisting forces are greater than the disturbing forces. Conversely, an unstable slope failure situation exists when resisting and disturbing forces are equal and the factor or safety equals F=1. A Limit Equilibrium condition exists when the forces tending to induce sliding are exactly balanced by those resisting sliding

2.4 Slope Stability Analysis

Slope stability analysis is performed to assess the safe design of human-made or natural slopes (e.g. embankments, road cuts, open-pit mining, excavations, landfills etc.) and the equilibrium conditions. Slope stability is the resistance of inclined surface to failure by sliding or collapsing. The main objectives of slope stability analysis are finding endangered areas, investigation of potential failure mechanisms, determination of the slope sensitivity to different triggering mechanisms, designing of optimal slopes with regard to safety, reliability and economics, designing possible remedial measures, e.g. barriers and stabilization.

Successful design of the slope requires geological information and site characteristics, e.g. properties of soil/rock mass, slope geometry, groundwater conditions, alternation of materials by faulting, joint or discontinuity systems, movements and tension in joints, earthquake activity etc. The presence of water has a detrimental effect on slope stability. Water pressure acting in the pore spaces, fractures or other discontinuities in the materials that make up the pit slope will reduce the strength of those materials. Choice of correct analysis technique depends on both site conditions and the potential mode of failure, with careful consideration being given to the varying strengths, weaknesses and limitations inherent in each methodology.

2.4.1 Analysis according to theory of limit states (LSD)

The verification methodology based on the theory of "Limit states" proves the safety by comparing a resisting variable (resisting force, strength, bearing capacity) and a variable causing failure (sliding force, stress).

$$X_{pas} > X_{act}$$

where: X_{pas}- A variable resisting the failure (resisting force, strength, capacity)

X_{act}- A variable causing the failure (sliding force, stress)

X_{act} is in general determined from the design parameters of soils and loading:

- ✓ soil parameters are reduced by corresponding coefficients
- ✓ load (its action) is increased by corresponding coefficients

X_{pas} is determined based on the following assumptions:

- ✓ soil parameters are reduced by corresponding coefficients
- ✓ the calculated structure resistance is reduced by a corresponding coenfficient

In general, it can be stated that the verification based on "Limit states" is more modern and apt approach in comparing to the "Safety factor". However, it is less lucid..

The value of utilization Vu is calculated and then compared with the value of 100%. The value of utilization is given by:

$$V_u = \frac{M_a}{M_p} 100 < 100\%$$

where: M_a- sliding moment

M_p- resisting moment

The resisting moment M_p is determined considering the reduction with the help of overall stability of construction γ_s.

2.4.2 Analysis according to safety factor (ASD)

The verification methodology based on the "Safety factor" is historically the oldest and most widely used approach for structure safety verification. The principal advantage is its simplicity and lucidity.

In general, the safety is proved using the safety factor:

$$FS = \frac{X_{pas}}{X_{act}} > FS_{req}$$

where: FS-Computed safety factor

X_{pas}- A variable resisting the failure (resisting force, strength, capacity)

X_{act}- A variable the causing failure (sliding force, stress)

FS_{req}- Required factor of safety

When performing the analysis using the "Safety factor", neither the load nor the soil parameters are reduced by any of the design coefficients.

Verification according to the factor of safety:

$$\frac{M_p}{M_a} > SF_s$$

where: M_a- sliding moment

M_p- resisting moment

SF_s- factor of safety

2.4.3 Short Term and Long Term Stability

In carrying our slope stability analyses for design purposes it is wise to check both short term and long term conditions. For the short term conditions an effective stress analysis could be used, but this will require an estimate of the pore pressures that will be developed. Alternatively a total stress analysis could be used, but this would only be applicable in cases where the pore pressure changes are entirely dependent upon stress changes. For long term conditions an effective stress analysis is normally carried out, since the pore pressures are usually independent of stress changes. For this analysis estimates of the pore pressures, for example, by means of flownets, are required. For natural slopes and slopes in residual soils, they should be analyzed with the effective stress method, considering the maximum water level under severe rainstorms.

2.4.5 Definition of the factor of safety (FOS)

The factor of safety for slope stability analysis is usually defined as the ratio of the ultimate shear strength divided by the mobilized shear stress at incipient failure. There are several ways in formulating the factor of safety F. The most common formulation for F assumes the factor of safety to be constant along the slip surface, and it is defined with respect to the force or moment equilibrium:

Moment equilibrium: generally used for the analysis of rotational land- slides. Considering a slip surface, the factor of safety F_m defined with respect to moment is given by:

$F_m=M_r/M_d$

Where M_r is the sum of the resisting moments and M_d is the sum of the driving moment. For a circular failure surface, the center of the circle is usually taken as the moment point for convenience. For a non-circular failure surface, an arbitrary point for the moment consideration may be taken in the analysis. It should be noted that for methods which do not satisfy horizontal force equilibrium (e.g. Bishop Method), the factor of safety will depend on the choice of the moment point as ‘true’ moment equilibrium requires force equilibrium. Actually, the use of the moment equilibrium equation without enforcing the force equilibrium cannot guarantee ‘true’ moment equilibrium.

Force equilibrium: generally applied to translational or rotational failures composed of planar or polygonal slip surfaces. The factor of safety F_f defined with respect to force is given by:

$F_f=F_r/F_d$

Where F_r is the sum of the resisting forces and F_d is the sum of the driving forces.

An acceptable factor of safety shall fulfill the basic requirement from the soil mechanics principle as well as the long-term performance of the slope. The geotechnical engineers should consider the current slope conditions as well as the future changes, such as the possibility of cuts at the slope toe, deforestation, surcharges and excessive infiltration. For very important slopes, there may be a need to monitor the pore pressure and suction by tensiometer and piezometer, and the displacement can be monitored by the inclinometers, GPS or microwave reflection. Use of strain gauges or optical fibers in soil nails to monitor the strain and the nail loads may also be considered if necessary. For larger-scale projects, the use of the classical monitoring method is expensive and time-consuming, and the use of the GPS has become popular in recent years.

2.4.6 Critical Slope Surfaces

A critical slope surface exists when a combination of soil and slope factors create a high potential for slope face failure and subsequent erosion. Over-steepened freshly graded or disturbed slopes are considered critical when resistance to surface erosion is low and sheer and strength resistance tolerances are exceeded. The potential for slope face failure of the slope can compound with inadequate slope face compaction under super saturated conditions. In such cases, soil movements are influenced by numerous parameters including, but not limited to, angle of repose, soil structure, slope length and erodibility.

2.4.6.1 Circular slip surface

All methods of limit equilibrium assume that the soil body above the slip surface is subdivided into blocks (dividing planes between blocks are always vertical). Forces acting on individual blocks are displayed in figure.

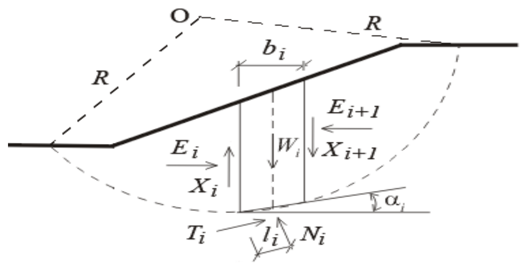


Fig.2.4 Circular slip surface

Here, Xi and Ei are the shear and normal forces acting between individual blocks, Ti and Ni are the shear and normal forces on individual segments of the slip surface, Wi are weights of individual blocks.

Individual methods of slices differ in their assumptions of satisfying the force equations of equilibrium and the moment equation of equilibrium with respect to the center O.

Various methods that can be adopted are:

- Fellenius / Petterson
- Bishop
- Spencer
- Janbu
- Morgenstern-Price
- Shahunyants
- ITF Method

Ground water specified within the slope body (using one of the five options) influences the analysis in two different ways. First when computing the weight of a soil block and second when determining the shear forces. Note that the effective soil parameters are used to relate the normal and shear forces.

Optimization of circular slip surface

The goal of the optimization process is to locate a slip surface with the smallest factor of slope stability SF. The circular slip surface is specified in terms of 3 points: two points on the ground surface and one inside the soil body. Each point on the surface has one degree of freedom while the internal point has two degrees of freedom. The slip surface is defined in terms of four independent parameters. Searching for such a set of parameters that yields the most critical results requires sensitivity analysis resulting in a matrix of changes of parameters that allows for fast and reliable optimization procedure. The slip surface that gives the smallest factor of slope stability is taken as the critical one. Parameters of individual slip surfaces and results from optimization runs can be displayed in output document.

This approach usually succeeds in finding the critical slip surface without encountering the problem of falling into a local minimum during iteration. It therefore appears as a suitable starting point when optimizing general slip surfaces such as the polygonal slip surface.

The optimization process can be restricted by various constraints. This becomes advantageous especially if we wish the searched slip surface to pass through a certain region or to bypass this region. Optimization restrictions are specified as a set of segments in a soil body. The optimized slip surface is then forced to bypass these segments during optimization.

2.4.6.2 Polygonal slip surface

Solution of the slope stability problem adopting the polygonal slip surface is based on the determination of the limit state of forces acting on the soil body above the slip surface. To introduce these forces the slip surface above is subdivided into blocks by dividing planes. Typically, these planes are assumed vertical, but this is not a required condition, e.g. the Sarma method considers generally inclined planes.

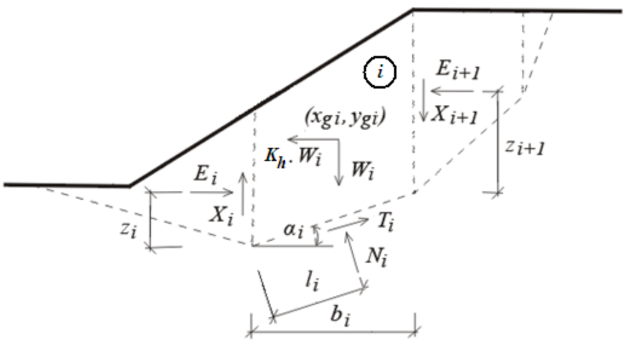


Fig.2.5 Polygonal Slip surface

The figure shows forces acting on individual blocks of soil. If the region above the slip surface is divided in blocks, then for the evaluation of unknowns we have: n normal forces Ni acting on individual segments and corresponding n shear forces Ti; n-1 normal forces between blocks Ei and corresponding n-1 shear forces Xi ; n-1 values of zi representing the points of application of forces Ei, n values of li representing the points of application of forces Ni and one value of the factor of safety SF. Forces Xi can be in some methods replaced by the values of inclination of forces Ei.

To following set of equations is available to solve the problem of equilibrium: n horizontal and n vertical equations of equilibrium written for individual blocks, n moment equations of equilibrium for individual blocks and n relations between Ni and Ti forces developed on blocks according to the Mohr-Coulomb theory. In total there are 4n equations for 6n-2 unknowns. This suggests that 2n-2 unknowns must be chosen a prior. Individual methods differ from each other in the way these values are selected.

Most often points of application of individual forces acting between blocks or their inclinations are selected. Solving the problem of equilibrium it proceeds in an iterative manner, where the selected values must allow for satisfying both the equilibrium and kinematical admissibility of the obtained solution.

Various methods that can be adopted are:

- Sarma
- Spencer
- Janbu
- Morgenstern-Price
- Shahunyants
- ITF Method

Optimization of polygonal slip surface

The slip surface optimization proceeds such that the program changes subsequently locations of individual points of this surface and checks, which change of location of a given point results in the maximal reduction of the factor of slope stability SF. The end points of the optimized slip surface are moved on the ground surface, internal points are moved in the vertical and horizontal directions. The step size is initially selected as one tenth of the smallest distance of neighboring points along the slip surface. With every new run the step size is reduced by one half. Location of points of slip surface is optimized subsequently from the left to the right and it is completed when there was no point moved in the last run.

When optimizing the polygonal slip surface the iteration process may suffer from falling into the local minimum (with respect to gradual evolution of locations of nodal points) so not always the process is terminated by locating the critical slip surface. Especially in case of complex slope profile it is therefore advantageous to introduce several locations of the initial slip surface. Combination with the approach used for circular slip surfaces is also recommended. Therefore, the critical slip surface assuming circular shape is found first and the result is then used to define the initial polygonal slip surface.

The optimization process can be restricted by various constraints. This becomes advantageous especially if we wish the searched slip to pass through a certain region or to bypass this region. The restriction on the optimization process can be performed in two different ways:

Optimization restrictions are specified as a set of segments in a soil body. The optimized slip surface is then forced to bypass these segments during optimization.

Another way of restricting the optimization process is to fix location of selected points along the optimized slip surface or allow for moving these points only in one of two directions, either vertically or horizontally.

2.5 Slope Stability Analysis Methods (Analytical)

2.5.1 Bishop Method

The Modified (or Simplified) Bishop's Method proposed by Alan W. Bishop of Imperial College is a method for calculating the stability of slopes. The simplified Bishop Method uses the method of slices to discretize the soil mass and determine the FS (Factor of Safety).

With this method, the analysis is carried out in terms of stresses instead of forces which were used with the Ordinary Method of Slices. The stresses and forces which act on a typical slice and which are taken into account in the analysis are shown in Fig. 1.

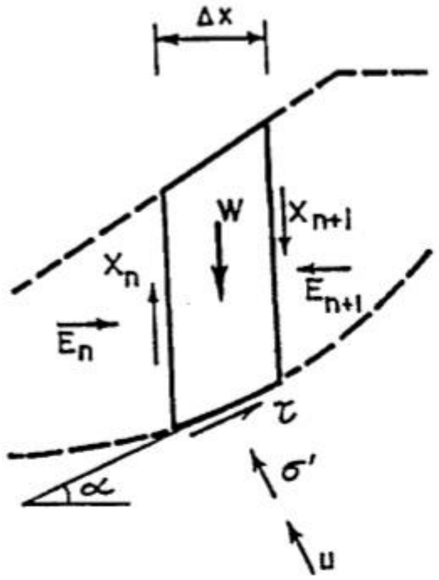


Fig.2.6 Stresses and Forces Acting on a Typical Slice

This method satisfies vertical force equilibrium for each slice and overall moment equilibrium about the center of the circular trial surface. Since horizontal forces are not considered at each slice, the simplified Bishop method also assumes zero interslice shear forces.

2.5.2 Fellenius / Petterson

Developed by Wolmar Fellenius as a result of slope failures in sensitive clays in Sweden, it reduces the force resolution of the slope to a statically determinate structure. The simplest method of slices assumes only the overall moment equation of equilibrium written with respect to the center of the slip surface i.e. side forces (shear and compressional) are not significant. The shear and normal forces between blocks X_i and E_i are neglected. The factor of safety FS follows directly from the following expression:

$$FS = \frac{1}{\sum_i W_i . \sin \alpha_i} . \sum_i \left[c_i . l_i + \left(N_i - u_i . l_i \right) . \tan \varphi_i \right]$$

where:

u_i- pore pressure within block

c_i,φ_i- effective values of soil parameters

W_i- block weight

N_i- normal force on the segment of the slip surface

α_i- inclination of the segment of the slip surface

l_i- length of the segment of the slip surface

2.5.3 Spencer

The Spencer method is a general method of slices developed on the basis of limit equilibrium. It requires satisfying equilibrium of forces and moments acting on individual blocks. The blocks are created by dividing the soil above the slip surface by dividing planes. Forces acting on individual blocks are displayed in the following figure.

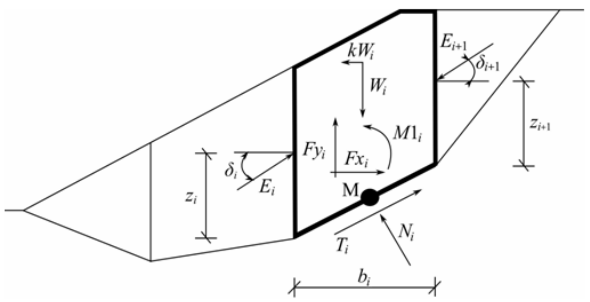


Fig.2.7 Static scheme – Spencer method

Each block is assumed to contribute due to the following forces:

Wi- block weight, including material surcharge having the character of weight including the influence of the coefficient of vertical earthquake K_v

K_h*W_i- horizontal inertia force representing the effect of earthquake, Kh is the factor of horizontal acceleration during earthquake

N_i- normal force on the slip surface

T_i- shear force on the slip surface

E_i ,E_{i+1}- forces exerted by neighboring blocks, they are inclined from horizontal plane by angle δ

F_{xi},F_{yi}- other horizontal and vertical forces acting on block

M_{1i}- moment of forces F_{xi}, F_{yi} rotating about point M, which is the center of the i^{th} segment of slip surface

U_i- pore pressure resultant on the i^{th} segment of slip surface

The following assumptions are introduced in the Spencer method to calculate the limit equilibrium of forces and moment on individual blocks:

- ✓ dividing planes between blocks are always vertical
- ✓ the line of action of weight of block Wi passes through the center of the i^{th} segment of slip surface represented by point M
- ✓ the normal force Ni is acting in the center of the i^{th} segment of slip surface, at point M
- ✓ inclination of forces E_i acting between blocks is constant for all blocks and equals to δ, only at slip surface end points is $\delta = 0$

2.5.4 Janbu

Janbu is a general method of slices developed on the basis of limit equilibrium. It requires satisfying equilibrium of forces and moments acting on individual blocks (only moment equilibrium at last uppermost block is not satisfied). The blocks are created by dividing the soil above the slip surface by dividing planes. Forces acting on individual blocks are displayed in the following figure:

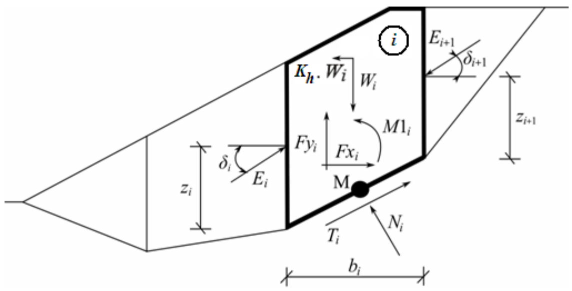


Fig.2.8 Static scheme – Janbu method

Each block is assumed to contribute due to the following forces:

W_i- block weight, including material surcharge having the character of weight including the influence of the coefficient of vertical earthquakeK_v

K_h*W_i- horizontal inertia force representing the effect of earthquake, K_h is the factor of horizontal acceleration during earthquake

N_i-normal force on the slip surface

Ti-shear force on the slip surface

E_i ,E_{i+1}- forces exerted by neighboring blocks, they are inclined from horizontal plane by angle δ_i resp. δ_{i+1} and lie at the height z_i resp. zi_{+1} above slip surface

F_{xi},F_{yi}- other horizontal and vertical forces acting on block

M_{1i}- moment from forces F_{xi}, F_{yi} rotating about point M, which is the center of the i^{th} segment of slip surface

U_i- pore pressure resultant on the i^{th} segment of slip surface

The following assumptions are introduced in the Janbu method to calculate the limit equilibrium of forces and moment on individual blocks:

- ✓ dividing planes between blocks are always vertical
- ✓ the line of action of weight of block Wi passes through the center of the i^{th} segment of slip surface represented by point M
- ✓ the normal force N_i is acting in the center of the i^{th} segment of slip surface, at point M
- ✓ position z_i of forces E_i acting between blocks is assumed, at slip surface end points is z = 0

2.5.5 Morgenstern-Price

Morgenstern-Price is a general method of slices developed on the basis of limit equilibrium. It requires satisfying equilibrium of forces and moments acting on individual blocks. The blocks are created by dividing the soil above the slip surface by dividing planes. Forces acting on individual blocks are displayed in the following figure:

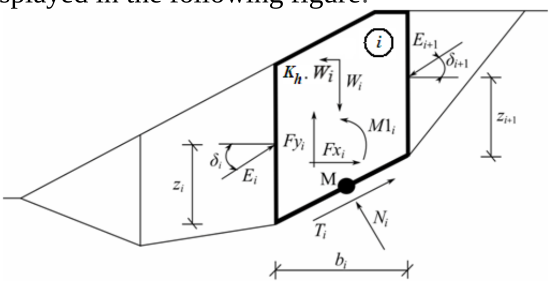


Fig.2.9 Static scheme – Morgenstern-Price method

Each block is assumed to contribute due to the same forces as in Spencer method.
The following assumptions are introduced in the Morgenstern-Price method to calculate the limit equilibrium of forces and moment on individual blocks:

- ✓ dividing planes between blocks are always vertical
- ✓ the line of action of weight of block Wi passes through the center of the ith segment of slip surface represented by point M
- ✓ the normal force Ni is acting in the center of the ith segment of slip surface, at point M
- ✓ inclination of forces Ei acting between blocks is different on each block (δi) at slip surface end points is δ = 0

2.6 Slope Stability Analysis Method- Finite Element Method

The finite element method represents a powerful alternative approach for slope stability analysis which is accurate, versatile and requires fewer a prior assumptions, especially, regarding the failure mechanism. Slope failure in the finite element model occurs `naturally' through the zones in which the shear strength of the soil is insufficient to resist the shear stresses.

With the development of cheaper personal computer, finite element method has been increasingly used in slope stability analysis. The advantage of a finite element approach in the analysis of slope stability problems over traditional limit equilibrium methods is that no assumption needs to be made in advance about the shape or location of the failure surface, slice side forces and their directions. The method can be applied with complex slope configurations and soil deposits in two or three dimensions to model virtually all types of mechanisms. General soil material models that include Mohr-Coulomb and numerous others can be employed. The equilibrium stresses, strains, and the associated shear strengths in the soil mass can be computed very accurately. The critical failure mechanism developed can be extremely general and need not be simple circular or logarithmic spiral arcs. The method can be extended to account for seepage induced failures, brittle soil behaviors, random field soil properties, and engineering interventions such as geo-textiles, soil nailing, drains and retaining walls. This method can give information about the deformations at working stress levels and is able to monitor progressive failure including overall shear failure.

2.6.1 Advantages of the finite element method

The advantages of a FE approach to slope stability analysis over traditional limit equilibrium methods can be summarized as follows:

1. No assumption needs to be made in advance about the shape or location of the failure surface. Failure occurs `naturally' through the zones within the soil mass in which the soil shear strength is unable to sustain the applied shear stresses.
2. Since there is no concept of slices in the FE approach, there is no need for assumptions about slice side forces. The FE method preserves global equilibrium until `failure' is reached.
3. If realistic soil compressibility data are avail- able, the FE solutions will give information about deformations at working stress levels. (d) The FE method is able to monitor progressive failure up to and including overall shear failure.

2.7 Foundations

2.7.1 Spread Footings

A spread footing is an enlargement at the bottom of a column or bearing wall that spreads the applied structural loads over a sufficiently large soil area. Typically, each column and each bearing wall has its own spread footing, so each structure may include dozens of individual footings.

Spread footings are by far the most common type of foundation, primarily because of their low cost and ease of construction. They are most often used in small to medium- size structures on sites with moderate to good soil conditions and can even be used on large structures when they are located at sites underlain by exceptionally good soil or shallow bedrock

The design and layout of spread footings is controlled by several factors, foremost of which is the weight (load) of the structure it will support as well as penetration of soft near-surface layers, and penetration through near-surface layers likely to change volume due to frost heave or shrink-swell.

2.7.2 Pile Foundations

The purpose of a pile foundation is to transmit the loads of a superstructure to the underlying soil while preventing excessive structural deformations. The capacity of the pile foundation is dependent on the material and geometry of each individual pile, the pile spacing (pile group effect), the strength and type of the surrounding soil, the method of pile installation, and the direction of applied loading (axial tension or compression, lateral shear and moment, or combinations).

Pile foundations are relatively long and slender members constructed by driving preformed units to the desired founding level, or by driving or drilling-in tubes to the required depth – the tubes being filled with concrete before or during withdrawal or by drilling unlined or wholly or partly lined boreholes which are then filled with concrete.

Piles are adopted when the loose soil extended to a great depth. The load of the structure is transmitted by the piles to hard stratum below or it is resisted by the friction developed on the sided of the piles. The piles may be placed separately or they may be placed in the form of cluster throughout the length of the wall.

2.7.2.1 Single piles

Prior to the development of reliable computer programs, the design of a single pile was based primarily on the ultimate load capacity of the pile as determined from a load test or from semi-empirical equations. The allowable or working load to which the pile could be subjected was taken as some fraction of the ultimate. Little, if any, emphasis was placed on the load-displacement behavior of the pile.

2.7.2.2 Pile groups

Classical methods (e.g., Culmann's method, the Common Analytical Method, the Elastic Center Method, the Moment-of-Inertia Method, etc.) of analysis for pile groups were based on numerous simplifying assumptions to allow the numerical calculations to be performed by hand. Common to these methods are the assumptions that only the axial resistance of the piles is significant and that the pile cap is rigid. Force and moment equilibrium equations are used to allot the foundation loads to the individual piles.

2.7.3 Micropiles

Micropiles, also known as minipiles, (and less commonly as pin piles, needle piles and root piles) are deep foundation elements constructed using high-strength, small-diameter steel casing and/or threaded bar. Capacities vary depending on the micropile size and subsurface profile. Allowable micropile capacities in excess of 1,000 tons have been achieved.

The micropile casing generally has a diameter in the range of 3 to 10 inches. Typically, the casing is advanced to the design depth using a drilling technique. Reinforcing steel in the form of an all-thread bar is typically inserted into the micropile casing. High-strength cement grout is then pumped into the casing. The casing may extend to the full depth or terminate above the bond zone with the reinforcing bar extending to the full depth.

In general, micropiles are applicable when there are problems with using conventional deep foundation systems like driven piles, drilled shafts, or augercast piling. These problem conditions include: natural or man-made obstructions, sensitive ground with adjacent structures, limited access/confined spaces.

Chapter 3: GEO5 Software

Geotechnical software GEO5 is a very simple and powerful tool for solving geotechnical problems based on traditional analytical methods and Finite Element Methods (FEM).The easy -to -use suite consists of individual programs with a unified and user-friendly interface. Each program is used to analyze a different geotechnical task but all modules communicate with each other to form an integrated suite.

3.1 GEO5 Geotechnical Software Solutions

GEO5 software helps you to address a wide variety of geotechnical problems. Besides common geotechnical engineering tasks, the suite also includes highly sophisticated applications for the analysis of tunnels, building damage due to tunneling, stability of rock slopes etc. GEO5 consists of wide range of powerful programs based on analytical methods and the Finite Element Method.

- Stability Analysis
- Excavation Design
- Retaining Wall Design
- Foundation Design
- Soil Settlement Analysis
- Digital Terrain Model
- Geotechnical Finite Element Analysis

3.2 Key Features of GEO5

1. **Integrated Software Suite**

GEO5 is a software suite for geotechnical analysis. It is consisted of individual applications, and each one solves a specific geotechnical problem. All programs are closely linked together and run the same environment. GEO5 is designed to solve most common geotechnical tasks, as well as highly sophisticated problems, such as tunnel analysis, damage on building due to tunneling, stability of rock slopes etc.

2. **Analytical and Finite Element Solutions**

GEO5 consists of wide range, powerful applications based on analytical methods and Finite Element Method. Analytical methods of computation, including slope stability, sheeting design etc., allow users to design and verify structures quickly and efficiently. The designed structure can be transferred into FEM applications, where general analysis of the structure is performed using FEM. Not only, this saves the users' time, but it also compares two independent solutions, which means it increases the design safety.

3. **User-Friendly Environment**

In most of the GEO5 application, user can design and verify a structure within an hour without any special training. When a user returns to work with the GEO5 software after a while, he instinctively knows how to use the application.

4. **Comprehensive Graphical Output**

One of the most important features of every application is to generate good outputs. GEO5 software allows to easily generate clear graphical and text reports. Output report can be adjusted according to users specific needs (add a company logo, picture etc.), printed, or saved in Word or PDF formats.

5. **Standards**

Basic geotechnical approaches implemented into the GEO5 software can be used all over the world, although most countries adopt their own standards and conventions. GEO5 offers a unique way of applying standards, which significantly simplify the work, and at the same time allow analysis using all required approaches.

3.3 Slope Stability – Slope Stability Analysis

The slope stability problem is solved in a two dimensional environment. The soil in a slope body can be found below the ground water table, water can also exceed the slope ground, which can be either partially or completely flooded. The slope can be loaded by a surcharge of a general shape either on the ground or inside the soil body. The analysis allows for including the effect of anchors expected to support the slope or for introduction of horizontal reinforcing elements – reinforcements. An earthquake can also be accounted for in the analysis.

Two types of approaches to the stability analysis are implemented in the program – classical analysis according to the factor of safety and the analysis following the theory of limit states.

The slip surface can be modeled in two different ways. Either as a circular one, then the user may choose either from the Fellenius/Petterson, Bishop, Spencer, Janbu or Morgenstern-Price, Shahunyants, ITF method, or as a polygonal one, in which case the program exploits the Sarma, Spencer, Janbu or Morgenstern-Price, Shahunyants, ITF method.

Stability problems may also be analyzed in the FEM program which will correctly account for soil/ structure interactions and give realistic deformations.

3.3.1 Influence of water

Ground water can be assigned to the slope plane section using one of the five options:

1) Ground water table:
   The ground water table is specified as a polygon. It can be arbitrarily curved, placed totally within the soil body or introduced partially above the ground surface. Presence of water influences value of pore pressure acting within a soil and reducing its shear bearing capacity.
2) Ground water table including suction:
   Suction table can be introduced above the inputted ground water table. A negative value of pore pressure u is then assumed with the region separated by the two tables. Suction increases as negative hydrostatic pressure from the ground water table towards the suction table.

3) Rapid draw down:

Original table can be introduced above the inputted ground water table. Original water table simulates state before rapid draw down.

First of all, the initial pore pressure u_0 is evaluated:

$$u_0 = \gamma_w . h_0$$

Second step is to calculate change of pore pressure in the area between original and ground water table:

$$\Delta u = \gamma_w . h_d$$

Third step is calculation of final value of pore pressure u. Change of pore pressure Δu is multiplied by coefficient of reduction of initial pore pressure X, which is required for all soils (dialog window "Soils"). X coefficient of the soil in the area of point P is used (NOT soil in the area between original and ground water table).

$$u = u_0 - X . \Delta u$$

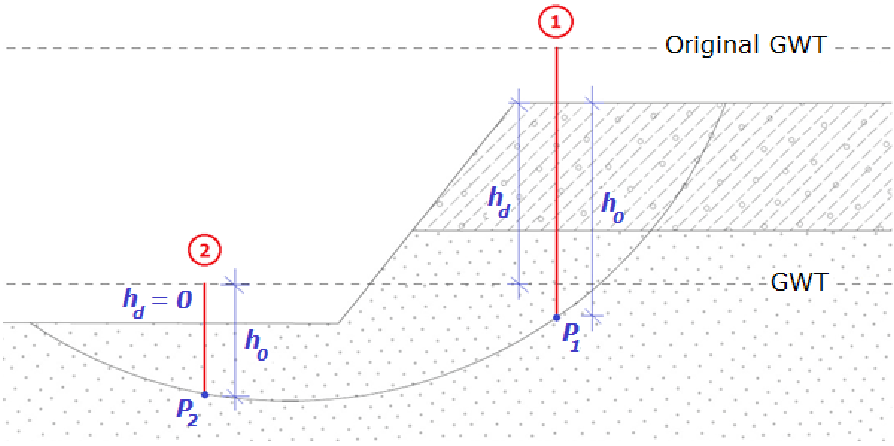


Fig. 3.1 Rapid draw down analysis

4) Coefficient of pore pressure R_u:

The coefficient of pore pressure R_u represents the ratio between the pore pressure and geostatic pressure in a soil body. In the area, where R_u is positive, entered unit weight of saturated soil γ_{sat} is considered; in other case unit weight of soil γ is used.

5) Pore pressure values:

Ground water can be introduced directly through the pore pressure values u within the plane section of a soil body. In the area, where u is positive, entered unit weight of saturated soil γ_{sat} is considered; in other case unit weight of soil γ is used.

3.3.2 Surcharge

The slope stability analysis takes into account even the surcharge caused by neighboring structures. The surcharge can be introduced either as a concentrated force or distributed load acting either on the ground surface or inside the soil body.

Since it is usually assumed that the surcharge is caused by the weight of objects found on the slope body, the vertical component of surcharge having the direction of weight (material component) is added to the weight of blocks. It means that if the earthquake effects are included this component is also multiplied by the factor of horizontal acceleration or vertical earthquake. Material surcharge component also influences the position of block centroid. The components that do not act in the direction of weight are assumed in equations of equilibrium written for a given block as weightless thus neither contribute to inertia effects of the earthquake nor position of block centroid.

The surcharge is always considered in the analysis with respect to one running meter. Providing the surcharge, essentially acting over the area b*l, is introduced as a concentrated force it is transformed before running the analysis into a surface loading spread up to a depth of slip surface along the slope 2:1 as displayed in figure.

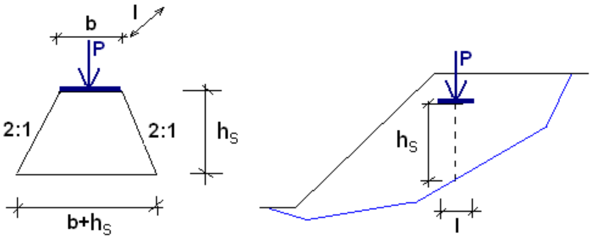


Fig.3.2 Effect of Surcharge

The analysis then proceeds with the resultant of surface load p having the value:

$$p = \frac{P}{(b + h_s).l}$$

3.3.3 Earthquake effect - standard analysis

The program allows for computing the earthquake effects with the help of two variables – factor of horizontal acceleration K_h or the coefficient of vertical earthquake K_v.

Coefficient of vertical earthquake K_v: The coefficient of vertical earthquake either decreases ($K_v > 0$) or increases ($K_v < 0$) the unit weight of a soil, water in a soil and material surcharge by multiplying the respective values by 1 - K_v. It is worth to note that the coefficient K_v may receive both positive and negative value and in case of sufficiently large coefficient of horizontal acceleration the slope relieve ($K_v > 0$) is more unfavorable than the surcharge.

Factor of horizontal acceleration K_h: In a general case the computation is carried out assuming a zero value of the factor K_h. This constant, however, can be exploited to simulate the effect of earthquake by setting a non-zero value. This value represents a ratio between horizontal and gravity accelerations. Increasing the factor K_h results in a corresponding decrease of the safety factor SF.

3.3.4 Influence of tensile cracks

The program makes it possible to account for the influence of tensile cracks that appear on terrain surface and are filled with water h. The only input parameter is the depth of tensile cracks. The effect of cracks is incorporated when calculating normal and shear forces in sections of a slip surface containing cracks – in a section with tensile cracks the shear strength parameters are set to zero (c = 0, φ = 0). Next, a horizontal force F due to presence water in a tensile crack is introduced in the analysis (see figure):

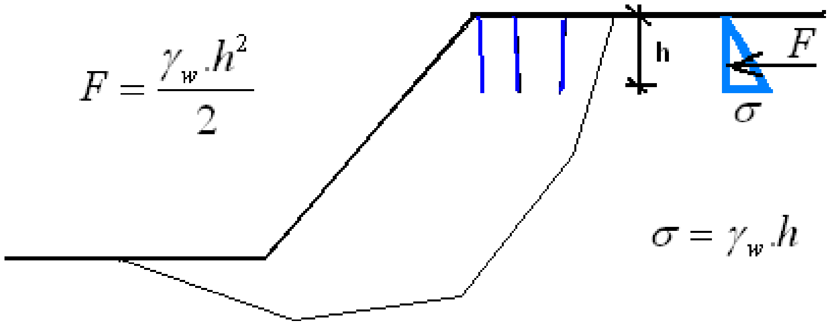


Fig. 3.3 Influence of tensile cracks

3.3.5 Anchors

Anchor is specified by two points and a force. The first point is always located on the ground surface; the force always acts in the direction of a soil body. The anchor force when computing equilibrium on a given block (slice) is added to the weightless surcharge of the slope.

Two options are available to account for anchors:

1. Compute anchor lengths – analysis assumes infinite lengths of anchors (anchors are always included in the analysis) and computes the required lengths of links anchors (distance between the anchor head and intersection of anchor with the slip surface) subsequently. The anchor root is then placed behind the slip surface. This approach is used whenever we wish the anchor to be always active and thus contribute to increase the slope stability and we need to know its minimum distance.
2. Analysis with specified lengths of anchors – the analysis takes into account only those anchors that have their end points (center of roots) behind the slip surface. This approach is used always whenever we wish to evaluate the current state of slope with already existing anchors, since it may happen that some of the anchors may prove to be short to intersect the critical slip surface so that they do not contribute to increase the slope stability.

3.3.6 Reinforcements

Reinforcements are horizontal reinforcing elements, which are placed into the soil to increase the slope stability utilizing their tensile strength. If the reinforcement intersects the slip surface, the force developed in the reinforcement enters the force equation of equilibrium of a given block. In the contrary case, the slope stability is not influenced.

The basic parameter of reinforcement is the tensile strength Rt. A design value of this parameter is used - i.e. the strength of reinforcement reduced by coefficients taking into account the effect of durability, creep and installation damage. The force transmitted by reinforcement can never exceed the assigned tensile strength Rt.

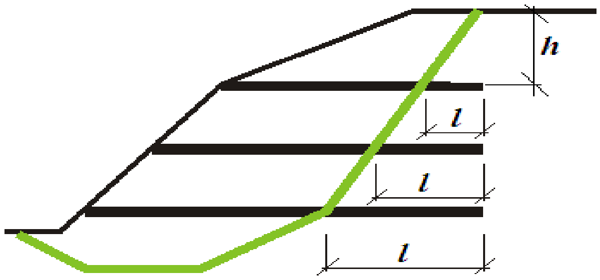


Fig. 3.4 Scheme of accounting for reinforcement

3.4 Input modes

Basic description of individual modes of inputting data into the program:

3.4.1 Interface

The "Interface" frame serves to introduce individual soil interfaces into the soil body. The program makes it possible to import or export interfaces in the *.DXF format. They can also be imported in the gINT format. Inputted interfaces can be copied within all 2D GEO5 programs using "GeoClipboard".

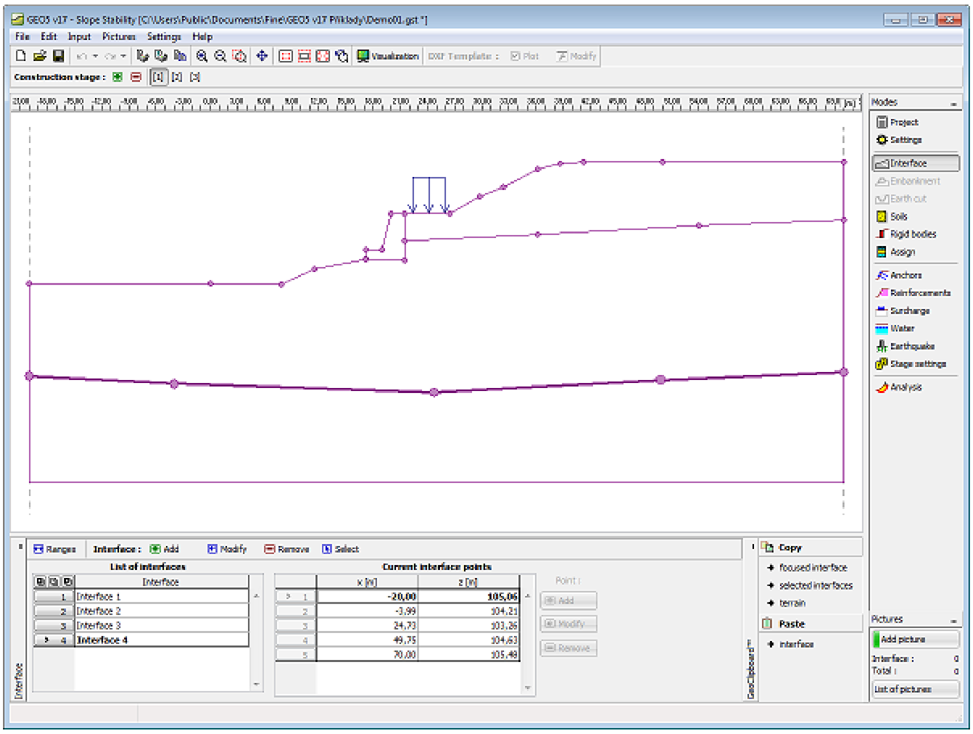


Fig.3.5 Frame “Interface

3.4.2 Soil body

The soil body is formed by a layered profile. An arbitrary number of layers can be used. Each layer is defined by its geometry and material. The material of a layer is usually represented by a soil with specified properties. The geostatic stress in a soil body is determined during the analysis.

A layer can be specified also as a rigid body. Such layer then represents bedrock or a sheeting wall. The slip surface can never pass through the rigid body.

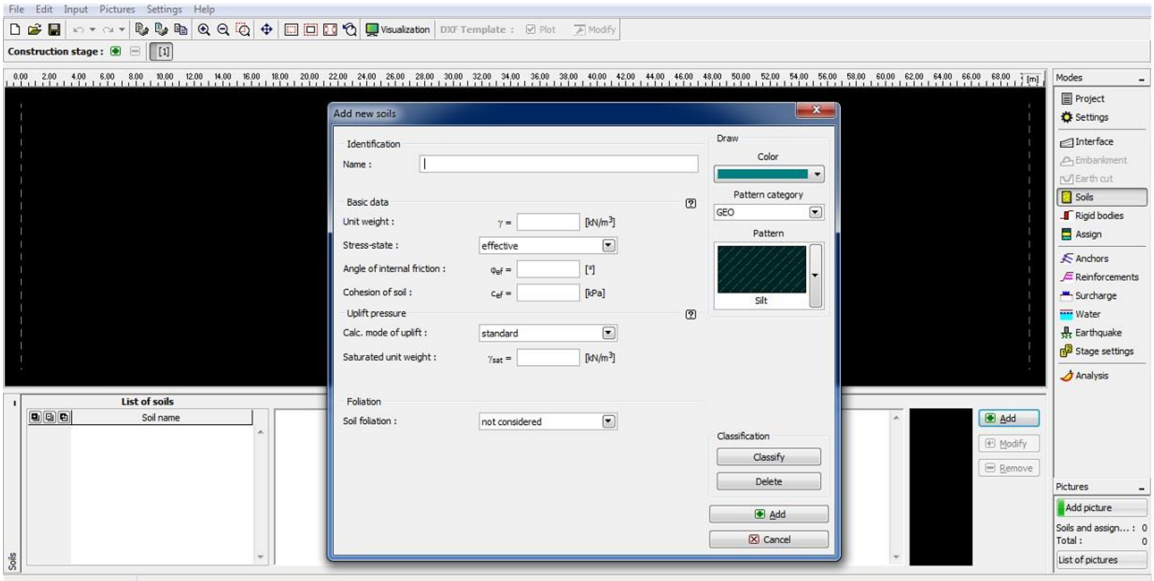


Fig.3.6 Soil Addition

3.4.3 Rigid body

The "Rigid bodies" frame contains a table with a list of inputted rigid bodies. The rigid bodies serve to model regions with a high stiffness – e.g., sheeting structures or rock subgrade.

Rigid bodies are introduced in the program as regions with high strength so they are not intersected by a potential slip surface. Providing we wish the slip surface to cross a rigid body (e.g., pile wall) it is recommended to model the rigid body as a soil with a cohesion corresponding to pile bearing capacity against slip.

3.4.4 Embankment

The "Embankment" frame allows for inputting interfaces to create an embankment above the current terrain. The frame contains a table with a list of interfaces forming the embankment. A table listing the points of currently selected interface of the embankment is displayed in the mid-section of the frame. Inputting an embankment interface follows the same steps as used for standard interfaces.

An embankment cannot be specified in the first stage of construction. An embankment cannot be built if there is an earth cut already specified in a given stage - in such a case either a new stage of construction must be introduced for embankment input or the existing open cut must be first removed.

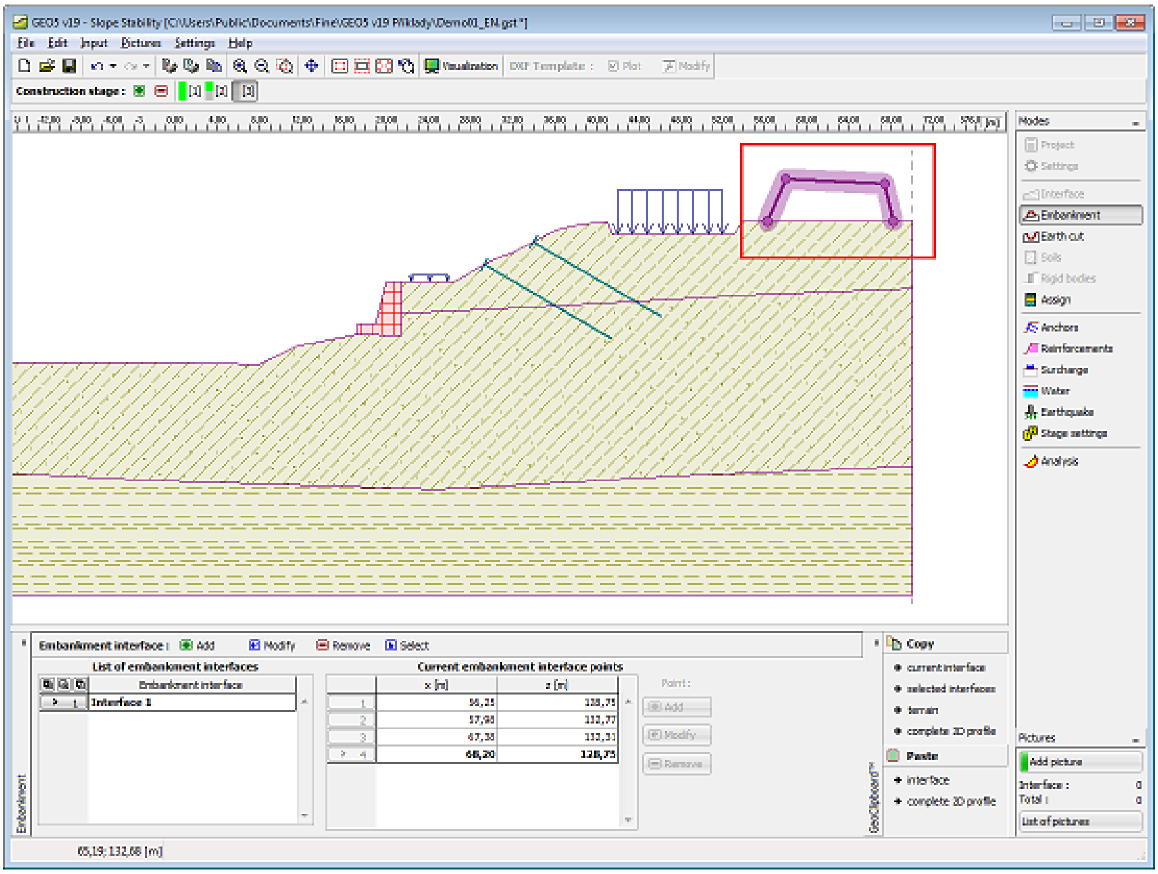


Fig.3.7 Frame "Embankment"

3.4.5 Earth cut

The "Earth cut" frame serves to specify the shape of an open cut. This function allows for modifying the terrain profile within a given stage of construction. Several earth cuts can be introduced at the same time. In such a case some of the lines in the cut appear partially above the terrain.

A table listing individual interface points is displayed in the left part of the frame. Inputting an earth cut interface follows the same steps as used for standard interfaces.

An open cut cannot be specified in the first stage of construction. An earth cut cannot be built if there is an embankment already specified in a given stage - in such a case either a new stage of construction must be introduced for earth cut input or the existing embankment must be first removed.

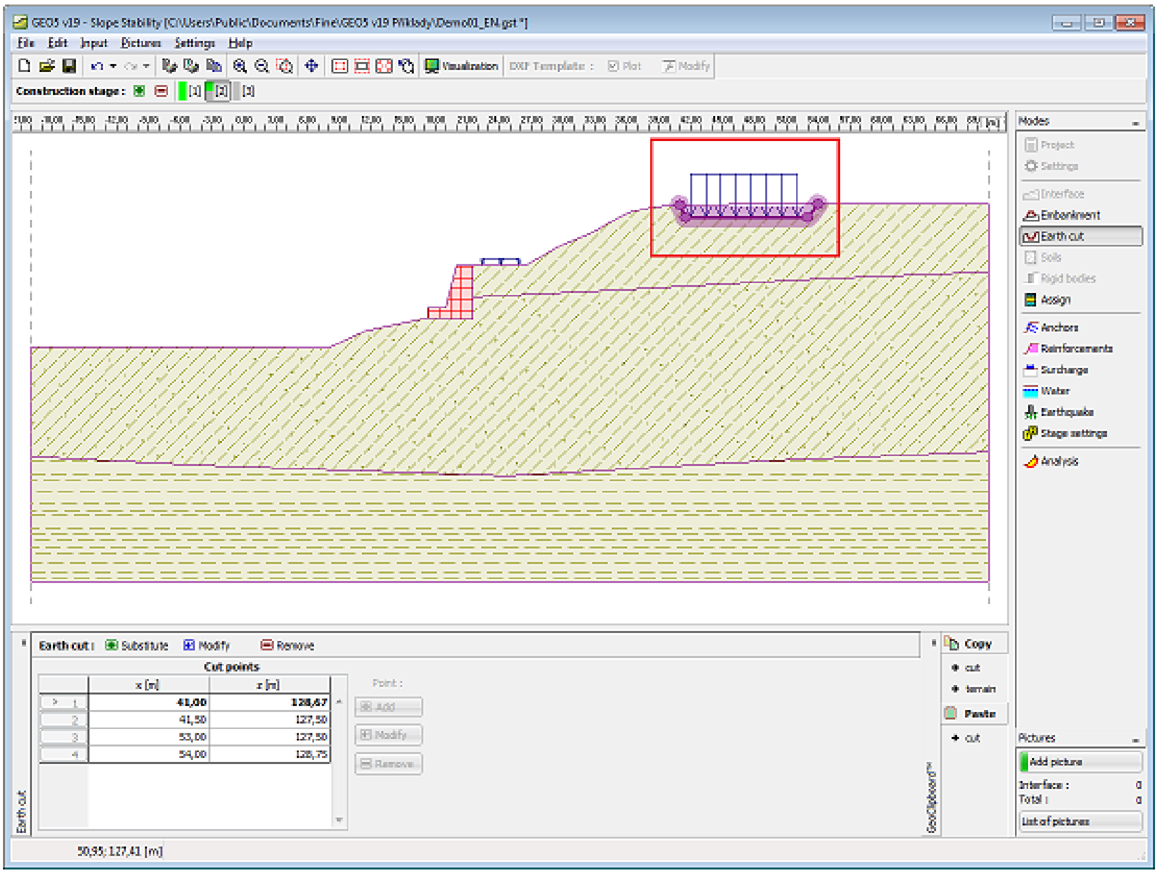


Fig.3.8 Frame “Earth Cut”

3.5 Slope Stability Analysis – FEM

In stability (safety factor) analysis the program reduces the original strength parameters - angle of internal friction and cohesion – until failure occurs. The analysis then results into a factor of safety that corresponds to the classical methods of limit equilibrium.

The safety factor analysis requires using six-node elements. Since plastic slip is the main failure mechanism we also require that the Mohr-Coulomb, the modified Mohr-Coulomb or the Drucker-Prager plasticity model be used for all soils. Default setting can be adjusted in the "Analysis settings" dialog window.

In the stability analysis mode the only variables available for graphical representation are displacements (in the Z and X-directions) and equivalent total and plastic strains. The deformation of a soil body corresponds to the state of failure attained for the reduced soil parameters - therefore, it does not correspond to real state of deformation of the soil body. Instead, it provides a good insight about the entire slope response of earth structure in general at the onset of failure.

A suitable way of presenting the stability analysis results are vectors of displacements plotted together with the equivalent plastic strain. The localized plastic deformation provides visible evidence about the possible location of the critical slip surface.

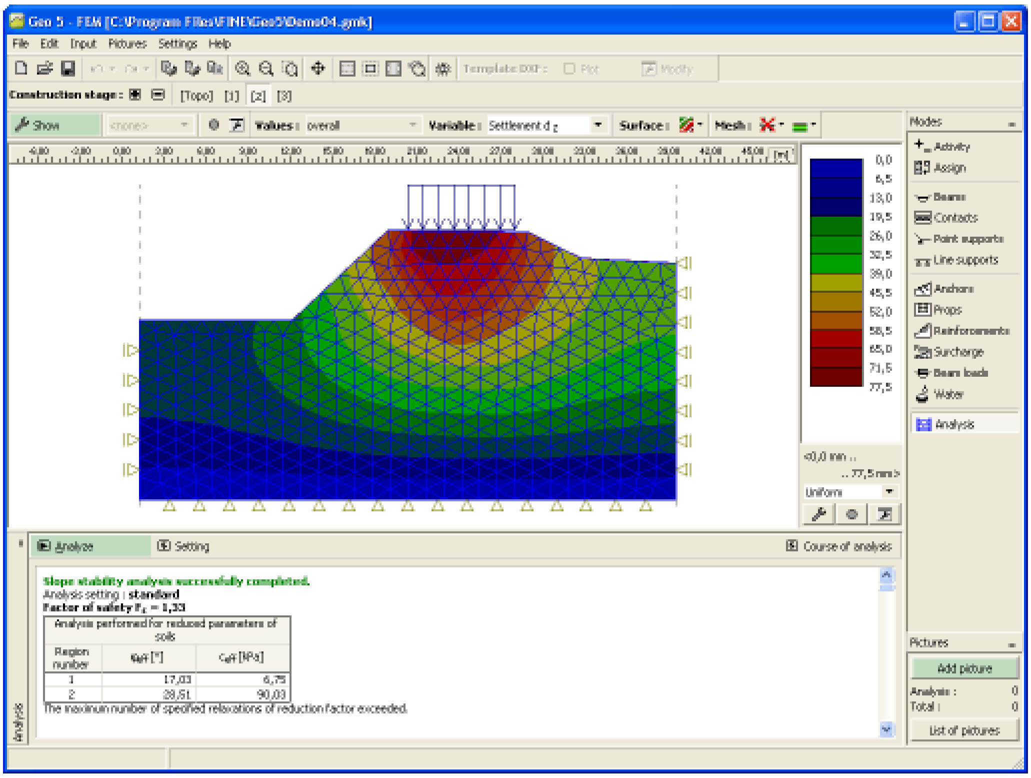


Fig.3.9 Frame "Stability"

3.5.1 Setting basic parameters of slope stability analysis

The safety factor analysis is based on the assumption that the total load applied to the soil/rock body is introduced in a single load step. The actual factor of safety is evaluated using the method of reduction of strength parameters c, φ. Regarding this the factor of safety is defined as a scalar multiplier that reduces the original parameters c, φ to arrive at the state of failure.

Mathematically, the factor of safety is expressed as:

$$F = \frac{\tan \varphi^{orig}}{\tan \varphi^{failure}}$$

where: $\varphi_{original}$ - the original value of the angle of internal friction

$\varphi_{failure}$ - the value of the angle of internal friction at failure

Searching for the critical value of the factor of safety requires a systematic modification (reduction) of strength parameters c, φ leading to failure. In the framework of the NRM the state of failure is determined as the state for which the solution fails to converge. The process of searching for critical c, φ is driven by the following parameters.

1. Reduction – reduction factor (scalar multiplier) to reduce parameters c, φ. During the course of analysis this parameter is progressively updated.
2. Min. reduction factor – the limit value, below which the value of reduction factor should not fall during the searching process. This parameter ensures that the

computation will not continue for needless low values of the reduction factor. It is one of the parameters to terminate the searching process.

3. Reduction of soil parameters – this parameter allows us to define which of the parameters c, φ should be reduced. The default setting assumes that both parameters are reduced at the same time.

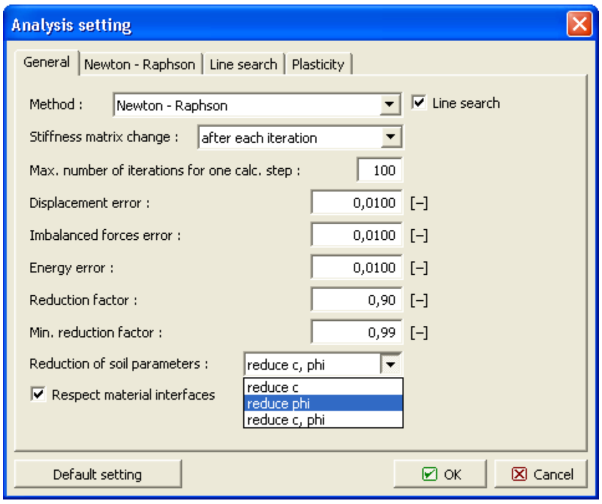


Fig.3.10 Basic parameters of slope stability analysis

3.5.2 Topology

To input data in the FEM program slightly differs from our other programs in that it requires defining the topology of the structure prior to any calculation. This step includes introduction of interfaces between individual layers of soils, line constructions, parameters of soils and interfaces and at last generation of the finite element mesh. To avoid unexpected errors when creating a computational model the user should first become familiar with available coordinate systems.

The topology input regime is selected by clicking the Topo button on the horizontal bar.

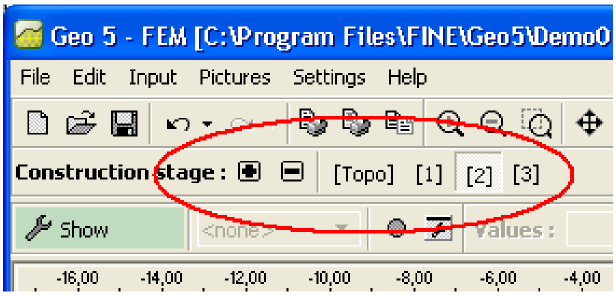


Fig.3.11 Bar "Stages of construction" – switching between "Topology" regime and calculation stages

The actual analysis is performed in individual stages of construction (calculation stages), which allow the user to define activity of structures, to input beams, anchors and surcharge, to model the effect of water, etc.

3.5.3 Mesh generation

The frame "Mesh generator" serves to define the basic setting to generate mesh (left part) and to view information about generated mesh (right part).

A successfully generated mesh completes the topology input stage – the analysis then proceeds with the calculation stages. When generating mesh the program automatically introduces standard boundary conditions. Information about the resulting mesh including warnings for possible weak points in the mesh is displayed in the right bottom window.

Correctly generated finite element mesh is the major step in achieving accurate and reliable results. The program FEM has an automatic mesh generator, which may substantially simplify this task. Nevertheless, certain rules should be followed when creating a finite element mesh:

The basic mesh density can be specified in the "Mesh generator" dialog window. I is generally accepted that the finer the mesh the better the results – computation as well as post-processing, however, may slow down substantially. The goal thus becomes to find an optimum mesh density – this mainly depends on the user experiences.

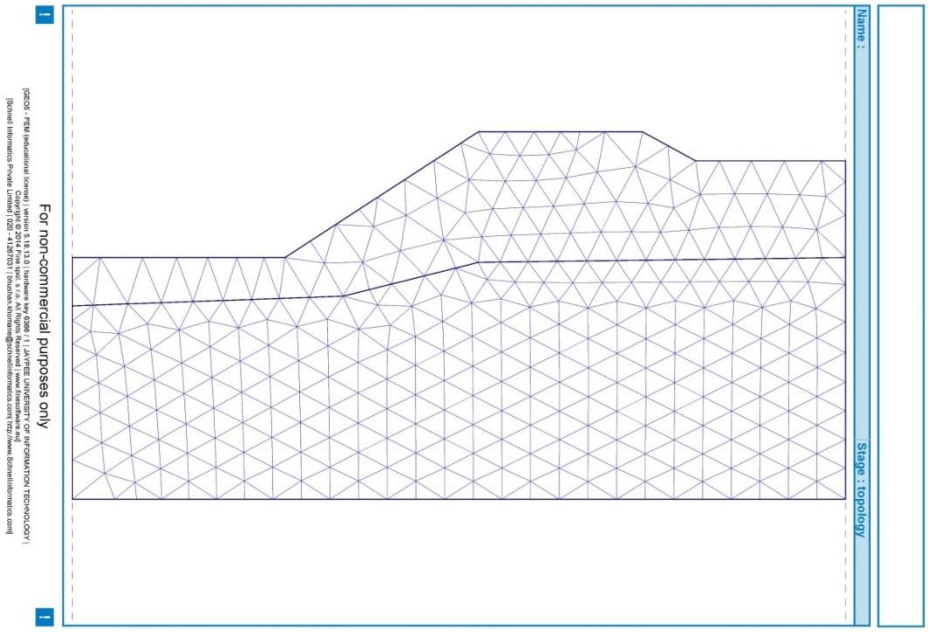


Fig.3.12 Mesh generation

By default program assumes 6-node triangular elements with mesh smoothing. The accuracy of the results more or less corresponds to twice as fine mesh composed of 3-node triangular elements. The 3-node elements are available only in the "Advanced input" mode and serve merely for research and testing purposes. The stability analysis, however, can be performed with 6-node triangular elements only. In case of nonlinear analysis, these elements should be used exclusively.

3.5.4 Construction stages

The actual analysis is performed in individual stages of construction (calculation stages) after the geometrical model and generating the finite element mesh (topology stage). One can move between calculation stages and the "Topology" regime using the buttons on the horizontal tool bar.

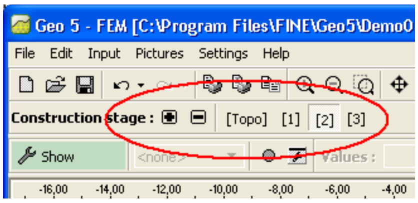


Fig.3.13 Tool bar "Construction stages" – switching between "Topology" regime and other stages of constructions

The calculation stages serve to model gradually build structures. Their correct definition and proper sequence is very important. The analysis of each stage builds (except for the stability analysis) upon the results derived in the previous stage. Information about individual objects and their properties are carried over from one stage to the other – when editing an existing stage or creating a new stage the program applies the principle of heredity.

3.5.5 Analysis

The analysis is performed for individual calculation stages in the frame "Analysis" after pressing the "Analyze" button.

During analysis the program attempts to arrive at such a solution that satisfies for given loading and boundary conditions the global equilibrium. In most cases this step results into an iterative process. The process of iteration and convergence of the solution is displayed on the screen.

The analysis can be stopped any time by pressing the "Interrupt" button. The results are then available for the last converged load increment.

The correct results are obtained when 100% of the applied load is reached. Due to convergence failure the program may stop before reaching the desired load level - only a fraction of the total applied load is reached. In such a case it is possible to adjust standard parameters of the analysis setting.

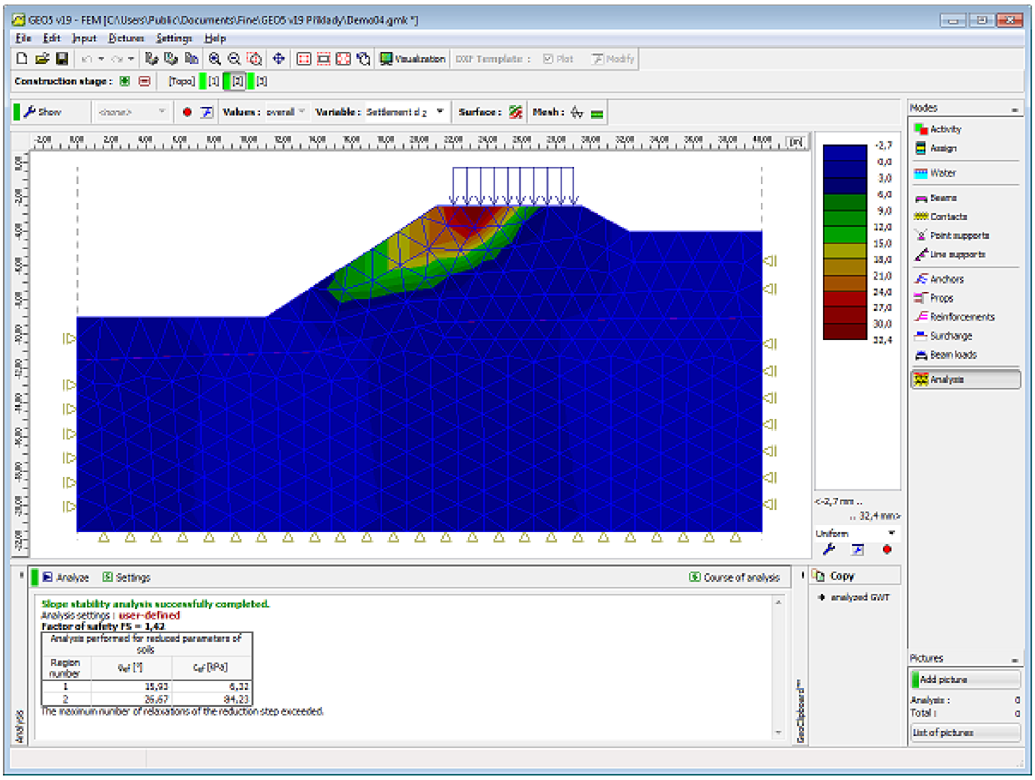


Fig.3.14 Screen after completing analysis

3.6 Spread Footing – Design of Spread Footings

This program is used for the design of spread footings subject to general load. It computes vertical and horizontal bearing capacity, settlement and rotation of a footing, and determines required longitudinal and shear reinforcement (punching).

3.6.1 Analysis of Foundation Bearing Capacity

3.6.1.1 Vertical Bearing Capacity

The vertical bearing capacity of the foundation soil is verified according to the theory of limit states using the following inequality:

$$\sigma \leq \frac{R_d}{\gamma_{RV}}$$

or based on the factor of safety as:

$$\frac{R_d}{\sigma} \geq SF_v$$

where:

σ - extreme design contact stress at the footing bottom

Rd - design bearing capacity of foundation soil

γRV - coefficient of vertical bearing capacity of foundation (for input use the "Spread Footing" tab)

SFv - safety factor for vertical bearing capacity

3.6.1.2 Bearing capacity on drained subsoil

Assuming drained conditions during construction the soil below spread footing deforms including both shear and volumetric deformations. In such a case the strength of soil is assumed in terms of effective values of the angle of internal friction φef and the effective cohesion cef. It is also assumed that there is an effective stress in the soil equal to the total stress (consolidated state). Effective parameters φef, cef represent the peak strength parameters.

Owing to the fact that the choice of drained conditions depends on a number of factors (rate of loading, soil permeability, degree of saturations and degree of overconsolidation) it is the designer's responsibility to decide, depending on the actual problem being solved, if the effective parameters should be used.

All approaches for finding bearing capacity on drained subsoil incorporate coefficients due to Brinch - Hansen (standard analysis) to account for inclined ground surface and inclined footing bottom.

Standard analysis

By default the solution proposed by J. Brinch - Hansena is used, where the bearing capacity of foundation soil follows from:

$$R_d = c.N_e.s_e.d_e.i_e.b_e.g_e + q_0.N_d.s_d.d_d.i_d.b_d.g_d + \frac{b}{2}.\gamma.N_b.s_b.d_b.i_b.b_b.g_b$$

where:	c	-	cohesion of soil
	q_0	-	equivalent uniform loading accounting for the influence of foundation depth
	d	-	depth of footing bottom
	γ	-	unit weight of soil above the footing bottom
	b	-	width of foundation
	γ	-	unit weight of soil
	N_c, N_d, N_b	-	coefficient of bearing capacity
	s_c, s_d, s_b	-	coefficients of shape of foundation
	d_c, d_d, d_b	-	coefficients of influence of foundation depth
	i_c, i_d, i_b	-	coefficients of influence of slope of loading
	g_c, g_d, g_b	-	coefficients of influence of slope of terrain
	φ	-	angle of internal friction of soil
	l	-	length of foundation
	δ	-	angle of deviation of the resultant force from the vertical direction
	β	-	slope of terrain
	α	-	slope of footing bottom

3.6.1.3 Bearing capacity on undrained subsoil

In case of undrained conditions it is assumed that during construction the spread footing undergoes an instantaneous settlement accompanied by shear deformations of soil in absence of volumetric changes. When the structure is completed the soil experiences both primary and secondary consolidation accompanied by volumetric changes. The influence of neutral stress appears in the reduction of soil strength. The strength of soil is then presented in terms of total values of the angle of internal friction φu and the total cohesion cu (these parameters can be considered as the minimal ones). Depending on the degree of consolidation the value of the total angle of internal friction φu ranges from 0 to φef, the total cohesion cu is greater than cef. Owing to the fact that the choice of undrained conditions depends on a number of factors (rate of loading, soil permeability, degree of saturations and degree of overconsolidation) it is the designer's responsibility to decide, depending on the actual problem being solved, if the effective parameters should be used. Nevertheless, the total parameters are generally used for fine-grained soil.

Standard analysis

The following formula is used by default:

$$R_d = (\pi + 2).c_u.s_c.d_c.i_c.b_c + q$$

with dimensionless coefficients:

$$s_c = 1 + 0{,}2.\frac{b}{l}$$

$$d_c = 1 + 0{,}1.\sqrt{\frac{d}{b}}$$

$$i_c = \sqrt{1 - tg\delta}$$

$$b_c = 1 - \frac{2.\alpha}{\pi + 2}$$

where:	c_u	-	total cohesion of soil
	b	-	width of foundation
	l	-	length of foundation
	d	-	depth of foundation
	δ	-	angle of deviation of the resultant force from the vertical direction
	α	-	slope of footing bottom from horizontal direction
	q	-	overburden pressure at the level of foundation base

3.6.1.4 Horizontal bearing capacity of foundation

The foundation horizontal bearing capacity is verified according to the theory of limit states using the following inequality:

$$H \leq \frac{R_{dh}}{\gamma_{RH}}$$

or based on the factor of safety as:

$$\frac{R_{dh}}{H} \leq SF$$

where:

$$R_{dh} = Q.tan\psi_d + a_d.A_{ef} + S_{pd}$$

$$H = \sqrt{H_x^2 + H_y^2}$$

where:	ψ_d	-	angle of internal friction between foundation and soil
	a_d	-	cohesion between foundation and soil
	A_{ef}	-	effective area of foundation
	S_{pd}	-	earth resistance
	H_x,H_y	-	components of horizontal force
	Q	-	extreme design vertical force
	γ_{RH}	-	coefficient of horizontal bearing capacity of foundation (for input use the "Spread Footing"
	SF	-	safety factor

3.6.1.5 Homogenization of layered subsoil

If the soil below the footing bottom is inhomogeneous (or if there is ground water present) then the inputted profile is transformed into a homogeneous soil based on the Prandtl slip surface (see Fig.), which represents the type and location of failure of the foundation.

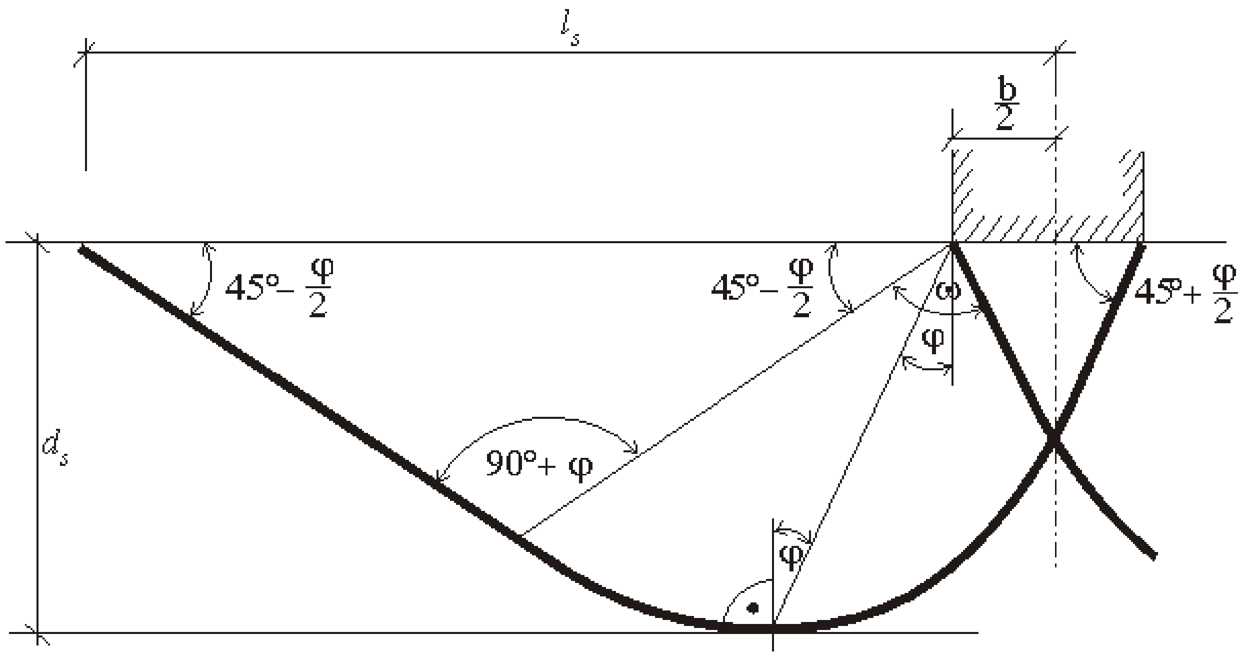


Fig3.15

Determination of equivalent values of φ (angle of internal friction), c (cohesion of soil) γ (unit weight of soil below footing bottom) is evident from the following formulas. The unit weight of soil above foundation is derived in the same way.

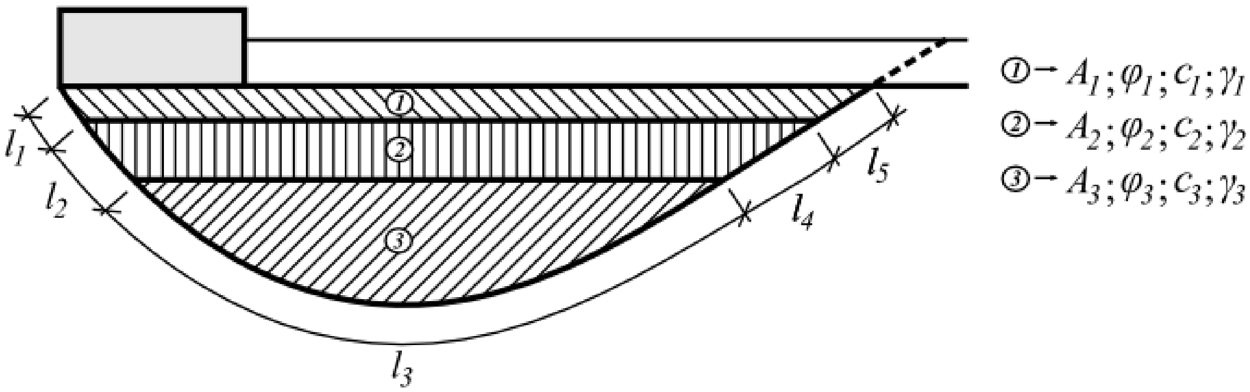


Fig3.16 Procedure for computation of auxiliary values

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

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

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

3.6.1.6 Verification of foundation eccentricity

Verification of foundation eccentricity is carried out for the *1st* LS (foundation bearing capacity) and the *2nd* LS (foundation settlement) analysis.

During analysis the program performs verification for the following cases:

- maximum eccentricity in the direction of base length: $e_x \le e_{alw}$
- maximum eccentricity in the direction of base width: $e_y \le e_{alw}$
- maximum overall eccentricity: $e_t \le e_{alw}$

The value of maximum allowable foundation eccentricity e_{alw} is inputted in the "**Settings**" frame, tab sheet "Spread Footing".

The value of overall eccentricity e_t is provided by:

$$e_t = \sqrt{e_x^2 + e_y^2}$$

where:	e_x	-	maximum eccentricity in the direction of base length
	e_y	-	maximum eccentricity in the direction of base width

For a shallow foundation resting on a rock foundation or for a concrete slab type of foundation it is necessary in some cases to adopt different values of limit eccentricities.

3.6.2 Settlement Analysis

One of the following methods is available to compute settlement:

- Using the oedometric modulus
- Using the compression constant
- Using the compression index
- According to NEN (Buismann, Ladde)
- Using the Soft soil model
- According to Janbu theory
- Using the DMT (constrained modulus)

The theory of elasticity (Boussinesq theory) is employed to determine stress in a soil state in all methods available for the settlement analysis.

General theories of settlement serve as bases in all the above methods.

When computing settlement below the footing bottom the programs first calculates the stress in the footing bottom and then determines the overall settlement and rotation of foundation.

The general approach in all theories draws on subdividing the subsoil into layers of a different thickness based on the depth below the footing bottom or ground surface. Vertical deformation of each layer is then computed - the overall settlement is then defined as a sum of partial settlements of individual layers within the influence zone (deformations below the influence zone are either zero or neglected):

$$s = \sum \Delta s_i$$

where:	s	-	Settlement
	s_i	-	settlement of the i^{th} layer

3.6.3 Verification of spread footing for punching shear

The critical section loaded in shear U_{cr} is distant from the column edge by one half of the footing thickness. It is loaded by the prescribed moments M_x, M_y and by the shear force V_r provided by:

$$V_r = \frac{Q.A_t}{A}$$

where:	A	-	area of footing

Q	-	assigned vertical force developed in column
A_t	-	hatched area in fig**.**

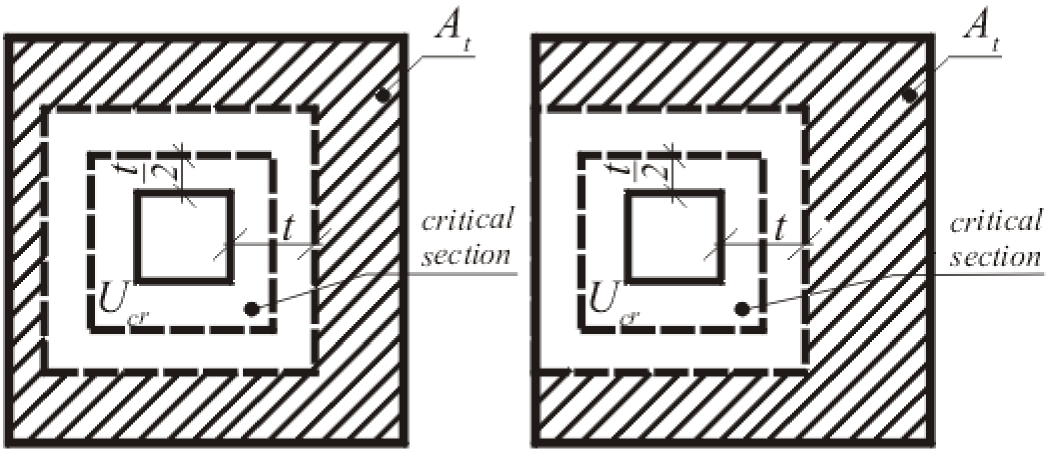


Fig3.17 Dimensioning of shear reinforcement area At

The program computes the maximum shear force V developed in the critical section, the shear force transmitted by concrete with no shear reinforcement V_c, and the maximal allowable force V_{max}:

$$V_c = \tau_{rd} . k_s . h$$

$$V_{max} = 1{,}5 . V_c$$

where:

$$\tau_c = 0{,}25 . \sqrt{f_{ctk}}$$

$$k_s = Min\left(0{,}5 + \frac{c_x}{c_y}\ ; 1\right)$$

where: c_x, c_y - are dimensions of footing column

For $V < V_c$ no shear reinforcement is needed.

For $V > V_c$and $V< V_{max}$ it is necessary to design shear reinforcement. The permissable shear force is given by:

$$V_{rd,3} = \frac{1}{2} . V_c + V_{us}$$

$$V_{us} = \frac{\sum 0{,}87\, A_{sv} . f_{yd} . sin\alpha}{u}$$

where:	u	-	critical cross-section span
	α	-	is angle of crooks

A_{us}	-	overall area of crooks in footing

For $V > V_{max}$ the shear reinforcement cannot be designed. It is therefore necessary to increase the cross-section depth.

3.7 Pile – Analysis of a Single Pile (Both Vertical and Lateral Load)

This program is used for analysis of vertical bearing capacity of a single pile loaded both in tension or compression, pile settlement as well as horizontal bearing capacity of a single pile. Analyses available in the program "**Pile**" can be divided into three main groups:

- Analysis of vertical bearing capacity
- Pile settlement
- Analysis of horizontal bearing capacity

3.7.1 Vertical bearing capacity

Analysis of pile vertical resistance can be carried out using analytical solution. The analytical solution assumes that the pile total compressive resistance Rc is derived as a sum of the pile base resistance R_b and the pile shaft resistance Rs (developed due to friction of the surrounding soil along the shaft). The following generally accepted methods are implemented into the program:

- NAVFAC DM 7.2
- Tomlinson
- Effective stress method
- CSN 73 1002

As Effective stress method is used so that would be discussed.

3.7.1.1 Effective stress method

The effective stress method allows calculating the vertical bearing capacity of an isolated pile in both cohesive and non-cohesive soils. This method is suitable for drained conditions - i.e. conditions that prevail after sufficient time passed the construction.

The pile shaft resistance is given by:

$$R_s = \sum_{j=1}^{n} q_{s,j} \cdot A_{s,j} = \sum_{j=1}^{n} \beta_{p,j} \cdot \sigma_{0,j} \cdot A_{s,j}$$

where:	$q_{s,j}$	-	shaft resistance in the j-th layer
	$\beta_{p,j}$	-	coefficients according to Bjerrum and Burland in the j-th layer
	$\sigma_{0,j}$	-	average effective stress due to overburden acting along the pile in the j-th layer
	A_{sj}	-	pile shaft area in the j-th layer

The pile base resistance is given by:

$$R_b = q_p.A_b = N_p.\sigma_p.A_b$$

where:	q_p	-	unit pile base resistance
	A_b	-	pile base area
	N_p	-	pile base resistance coefficient (according Fellenius)
	σ_p	-	effective stress due to overburden acting at pile base

3.7.1.2 Coefficients of pile bearing capacity

Recommended ranges of values of coefficients of pile base resistance N_p and coefficient β are listed in the following table. The coefficient β is usually found in the given range, it seldom exceeds the value *1,0*.

Range of coefficients N_p **and** β (Fellenius, 1991)

Type of soil	φ_{ef}	N_p	β
Clay	25 - 30	3 – 30	0.23 - 0.40
Silt	28 - 34	20 – 40	0.27 - 0.50
Sand	32 - 40	30 – 150	0.30 - 0.60
Gravel	35 - 45	60 – 300	0.35 - 0.80

Fig3.18

3.7.2 Pile settlement

3.7.2.1 Nonlinear theory (Masopust)

The nonlinear theory constructs load-settlement curve assuming that evolution of settlement as a function of resistance up to full mobilization of skin friction can be represented by parabola. After that the relationship is linear as displayed in figure. This method was derived from equations of regression curves constructed on the basis of statistical analysis of the results of static loading tests of piles and for the determination of vertical bearing capacity it employs regression coefficients.

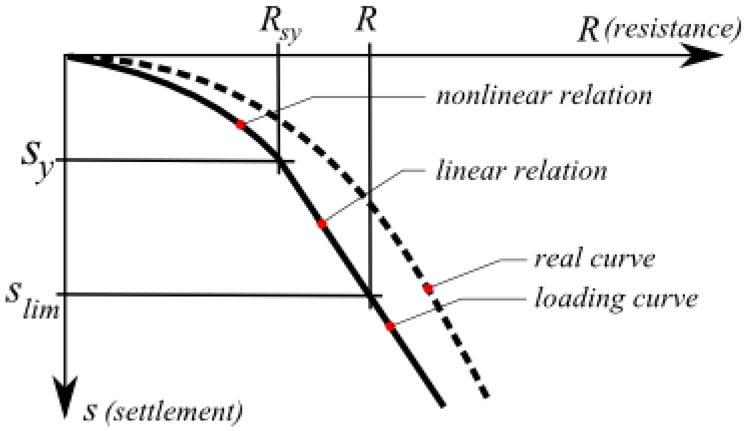


Fig.3.19

3.7.2.2 Linear theory (Poulos)

Analysis of the load-settlement curve of single pile or pile group is based on the solution described in the book Pile Foundations Analysis and Design (H. G. Poulos et. E. H. Davis, 1980) and is based on the theory of elasticity and modifications attributed to in-situ measurements. Foundation soil is therefore characterized by the modulus of elasticity E and by the Poisson´s ratio ν. This method allows the constructinon of the load-settlement curve for pile foundations (single pile, pile group).

The basic input parameters of the analysis are pile base bearing capacity Rbu and pile skin bearing capacity Rsu. Ultimate bearing capacity of pile foundation, respectively ultimate load is given by equation Pu = Rsu + Rbu. These values are obtained by the program from the analysis of vertical bearing capacity of single pile or pile group and it depends on the selected method of analysis. All partial factors of the analysis are assumed equal to 1.0 so that the resulting resistance is greater than the one obtained from actual bearing capacity analysis.

During the analysis of settlement of single pile or pile group according to Poulos method (1980) program don´t consider influence of additional compression of the pile shaft - that´s why displacement of pile material is neglected.

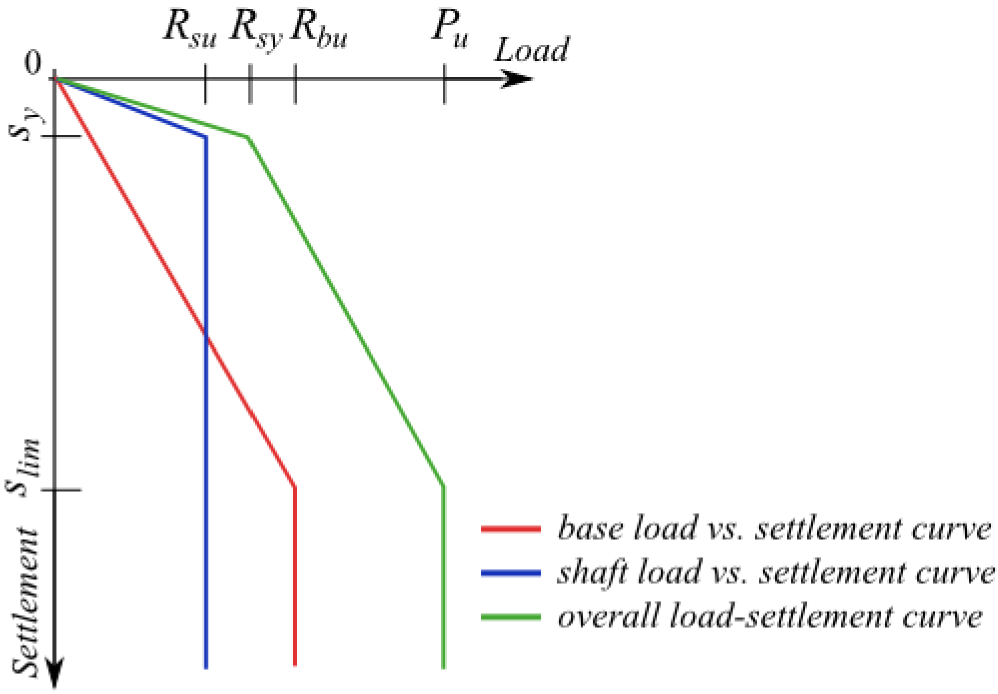


Fig3.20

Basic settlement-influence factor Io

The basic settlement-influence factor Io depends on the pile length l and diameter d and the values of this coefficient are generally provided by the following graph also showing their ranges:

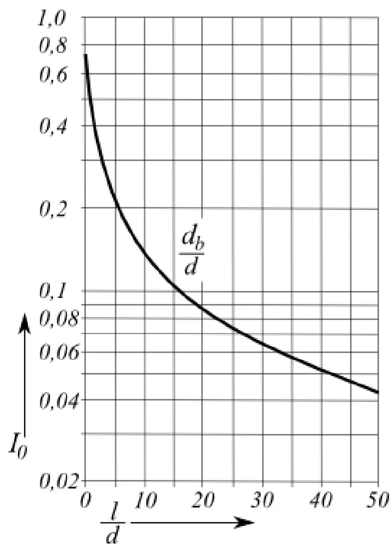


Fig 3.21

3.7.3 Horizontal bearing capacity - elastic subsoil (p-y method)

The horizontally loaded pile is analyzed using the finite element method as a beam on elastic Winkler foundation. The soil parameters along the pile are represented by the modulus of subsoil reaction. By default the pile is subdivided into 30 segments. For each segment the program determines the values of the modulus of subsoil reaction, internal forces and deformation (displacements). The program also allows for dimensioning of the steel-reinforced concrete pile based on the method specified in the frame "Settings" and on the parameters inputted in the "Piles" tab sheet.

The program also enables to analyze a pile loaded by the **prescribed displacements** (translation or rotation of the pile head). In such a case the analysis is carried out only with the prescribed displacement. The inputted mechanical load is excluded.

The following options for inputting the **modulus of subsoil reaction** are available in the program:

- **by distribution** (distribution of the modulus of subsoil reaction along the pile is specified)
- constant distribution
- linear distribution (Bowles)
- according to CSN 73 1004
- according to Matlock and Reese
- according to Vesic

In general, the modulus of subsoil reaction corresponds to the spring stiffness in the Winkler model. This model describes settlement of a rigid slab as a function of the applied load. The corresponding relationship is represented by the following formula:

$$p = ky$$

where:	p	-	load acting along slab-soil interface
	k	-	stiffness of Winkler spring
	y	-	translation (displacement) of slab into subsoil

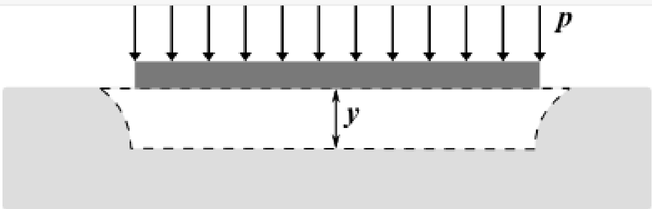


Fig3.22 Definition of the modulus of subsoil reaction

3.7.3.1 Pile horizontal bearing capacity - Brom's method

Analysis of a single pile according to Broms is described in Broms, 1964. This method exclusively assumes a pile in the **homogeneous soil**. Thus the analysis method does not allow for layered subsoil. The type of analysis of the pile horizontal bearing capacity is specified in the "**Settings**" frame, tab "Piles".

When adopting the Broms method for the analysis of horizontal bearing capacity the program disregards up now inputted soil layers. The soil parameters are specified in the "Horizontal bearing capacity" frame based on the **type of soil** (cohesive, cohesionless).

The input parameters for the analysis of pile horizontal bearing capacity are the **pile material characteristics** (modulus of elasticity and strength of a given material), **pile geometry** (pile length *l* and its diameter *d*) and also the **pile loading** due to shear force and bending moment.

The coefficient of pile stiffness β for cohesive soils is given by:

$$\beta = \frac{k_h\,d}{4\,EI}$$

where:	EI	-	bending stiffness of pile section [MNm^2]
	k_h	-	modulus of subsoil reaction [MNm^3]
	d	-	diameter of a single pile [m] - in case of a pile with a **circular variable cross-section** the parameter β assumes a constant value of the pile diameter d_1 inputted in the "Geometry" frame

The coefficient of pile stiffness η for cohesionless soils follows from:

$$\eta = \left(\frac{n_h}{EI}\right)^{\frac{1}{5}}$$

where:	EI	-	bending stiffness of pile section [MNm^2]
	n_h	-	coefficient of soil modulus variation [MNm^3]

3.8 Pile Group – Analysis of a Pile Group (Pile Raft Foundations with a Rigid Pile Cap)

This program is used to analyze a pile group (pile raft foundation with a rigid pile cap) using both spring method (FEM), and analytical solutions. Both floating piles and piles fixed into subsoil can be considered.

Analyses performed in the "Pile Group" program can be divided into two groups:

- Analytical solution - calculation of the vertical bearing capacity of a pile group for cohesive and cohesionless soils and the determination of settlement
- Analysis of a pile group using the spring method together with the determination of reinforcement of piles

3.8.1 Analytical solutions

- Analysis of vertical bearing capacity of a pile group of in cohesive soil as a rigid earth block according to FHWA standards
- Analysis of vertical bearing capacity of a group of piles in cohesionless soil (NAVFAC DM 7.2, Effective stress, CSN 73 1002)
- Reduction of vertical bearing capacity of a pile group (UFC 3-220-01A, La Barré, Seiler-Keeney)
- Settlement analysis of a pile group in a cohesive soil as a settlement of substitute foundation
- Settlement analysis of a pile group in cohesionless soil according to Poulos (load-settlement curve)

3.8.2 Spring method (FEM) – Analysis of a three-dimensional action of a group of piles

- Analysis of rotation, deformation and displacement of a rigid pile cap
- An arbitrary number of load cases and their combinations
- Hinged or fixed connection between piles in a group and a rigid pile cap
- Floating, rested or fixed piles into the foundation subsoil
- Possibility to introduce inclined piles and general shape of a rigid pile cap
- An automatic post-calculation of springs along the pile length piles from soil properties
- Possibility to introduce both horizontal and vertical springs along the pile length
- Distribution of displacements and internal forces along for each pile in a group
- Dimensioning of a pile reinforcement according to EN 1992-1 (EC 2), BS, PN, IS, AS, ACI, GB, CSN, SNiP

3.8.3 Pile group settlement

Cohesionless soil

The analysis of a pile group in a cohesionless soil is developed based on the linear theory of settlement (Poulos). The load-settlement curve for a pile group and the value of the total settlement s_g is increased by so-called **group settlement factor** g_f.

An immediate settlement of the pile group increased by the group settlement factor is provided by:

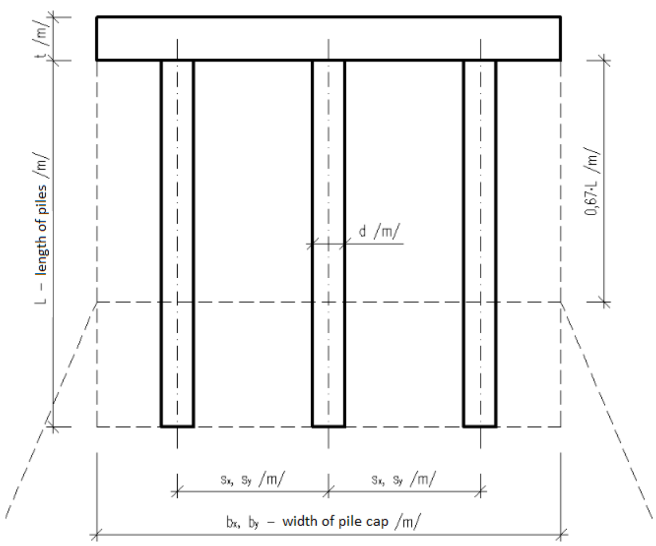
$$s_g = g_f \cdot s_0$$

$$g_f = \sqrt{\frac{b_x}{d}}$$

where:	s_g	-	pile group settlement
	g_f	-	group settlement factor for a cohesionless soil (according to Pile Buck Inc. 1
	s_0	-	settlement of a single pile (determined, e.g. from the load-settlement curve)
	d	-	pile diameter
	b_x	-	minimum width of pile group

Cohesive soil

The pile group settlement in a cohesive soil is determined as the settlement of a substitute foundation at a depth of $0{,}67{*}L$, having a width B and a length $B´$.



3.23 Scheme of substitute foundation - settlement of pile group in cohesive soil

Chapter4: Slope Stability Analysis Example

4.1 Analysis Using Analytical Methods

4.1.1 Settings

First of all settings are chosen according to users requirement.

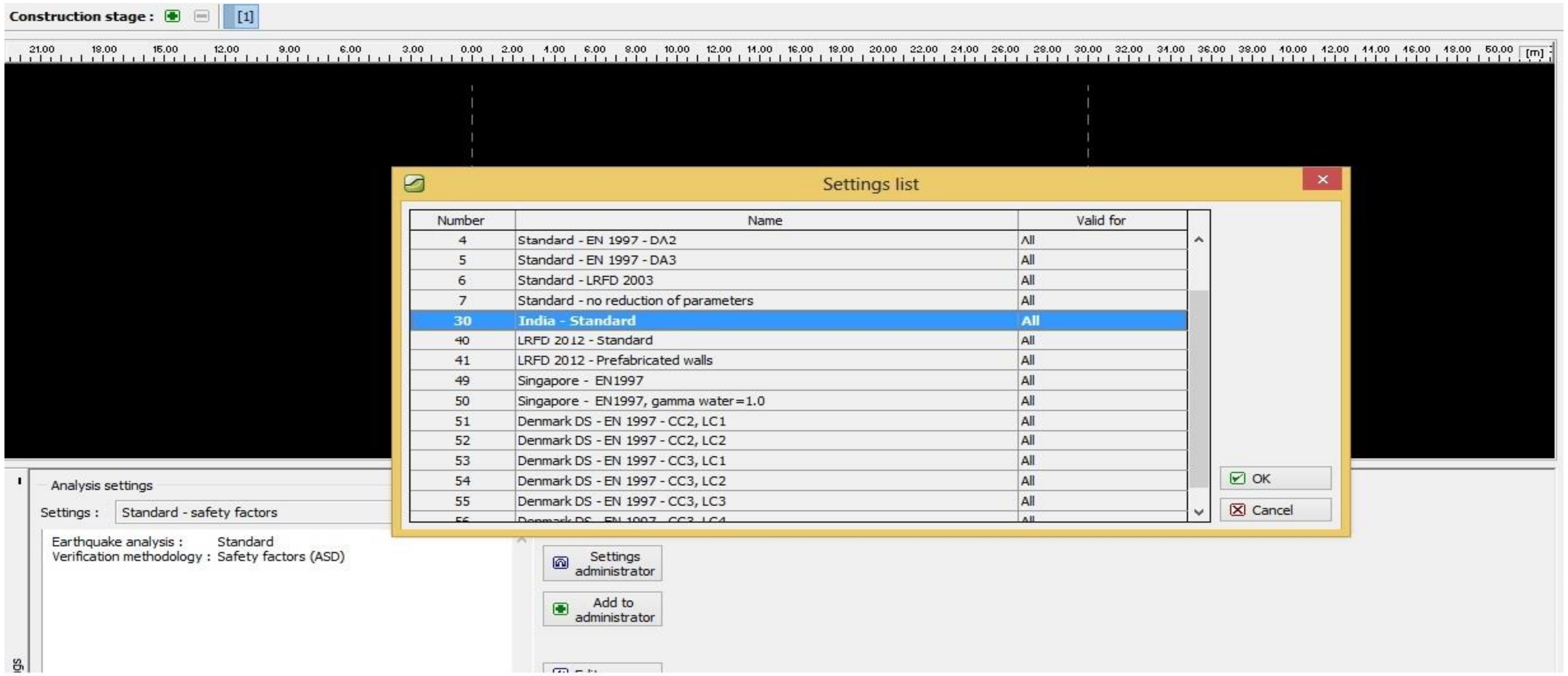
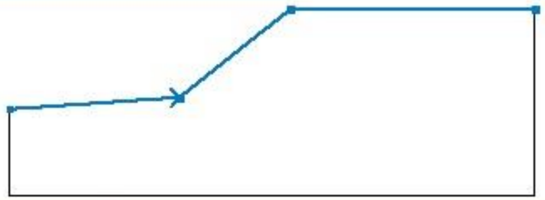


Fig.4.1 Slope Stability Settings frame

4.1.2 Interface and Soil Addition

An Arbitrary interface and soil profile was taken, as shown in the figure.

Interface

No.	Interface location	Coordinates of interface points [m]					
		x	z	x	z	x	z
1		0.00	-5.44	9.73	-4.77	16.05	0.26
		30.00	0.26				

Soil parameters - effective stress state

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m3]
1	Clayey sand (SC)		27.00	8.00	18.50

Soil parameters - uplift

No.	Name	Pattern	γ_{sat} [kN/m3]	γ_s [kN/m3]	n [–]
1	Clayey sand (SC)		19.00		

Fig.4.2 Interface and soil addition

4.1.3 Surcharge and Water Addition

The following surcharge and ground water table was added:

Surcharge

No.	Type	Type of action	Location z [m]	Origin x [m]	Length l [m]	Width b [m]	Slope α [°]	Magnitude q, q_1, f, F	Magnitude q_2	Magnitude unit
1	strip	permanent	on terrain	x = 20.00	l = 5.00		0.00	100.00		kN/m^2

Water

Water type : GWT

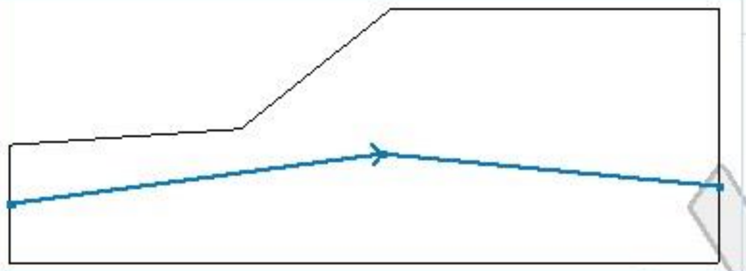
No.	GWT location	Coordinates of GWT points [m] x	z	x	z	x	z
1		0.00	-7.92	15.74	-5.82	30.00	-7.18

Fig.4.3 Surcharge and water table

4.1.4 Earthquake Settings can be done according to the user i.e. to include earthquake analysis or not.

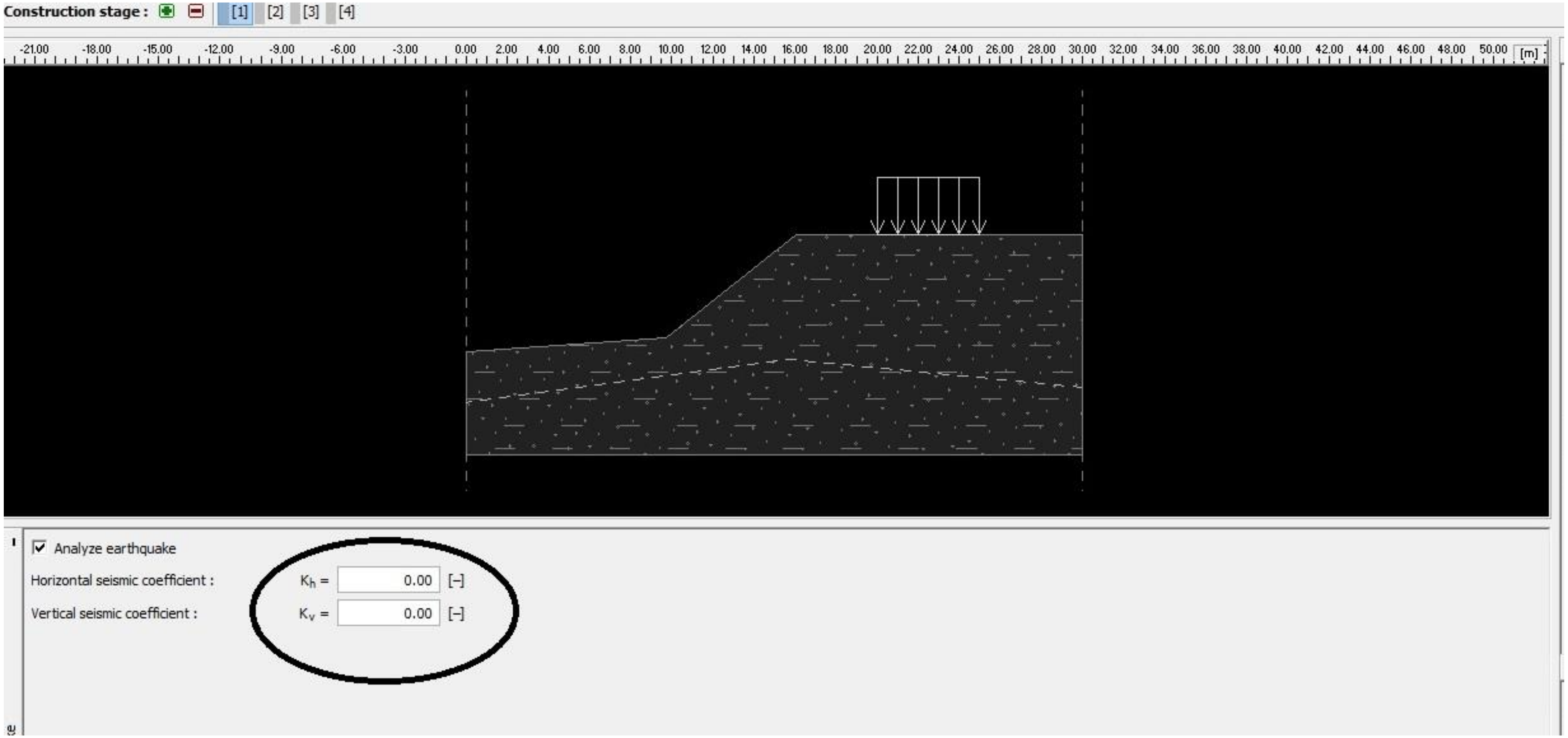


Fig.4.4 Earthquake Settings

If any user wants to include earthquake analysis in the program then input horizontal and vertical seismic coefficient.

4.1.5 Analysis

4.1.5.1 Circular Slip Surface

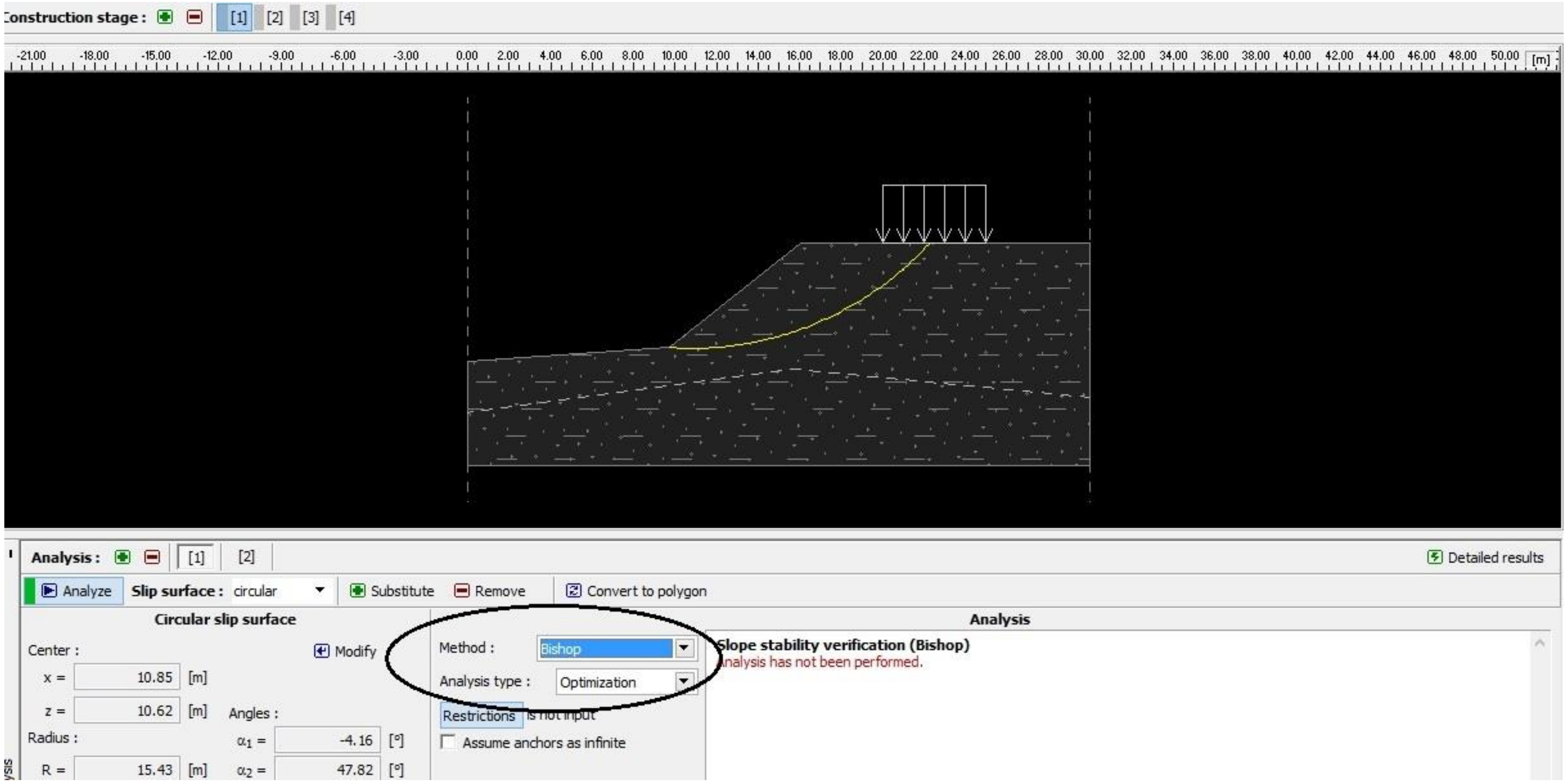


Fig.4.5 Analysis using circular slip surface

Choose the desired method for analyzing and the type i.e. optimization or standard. Optimization would give us the critical slip surface.

4.1.5.2 Polygonal Slip Surface

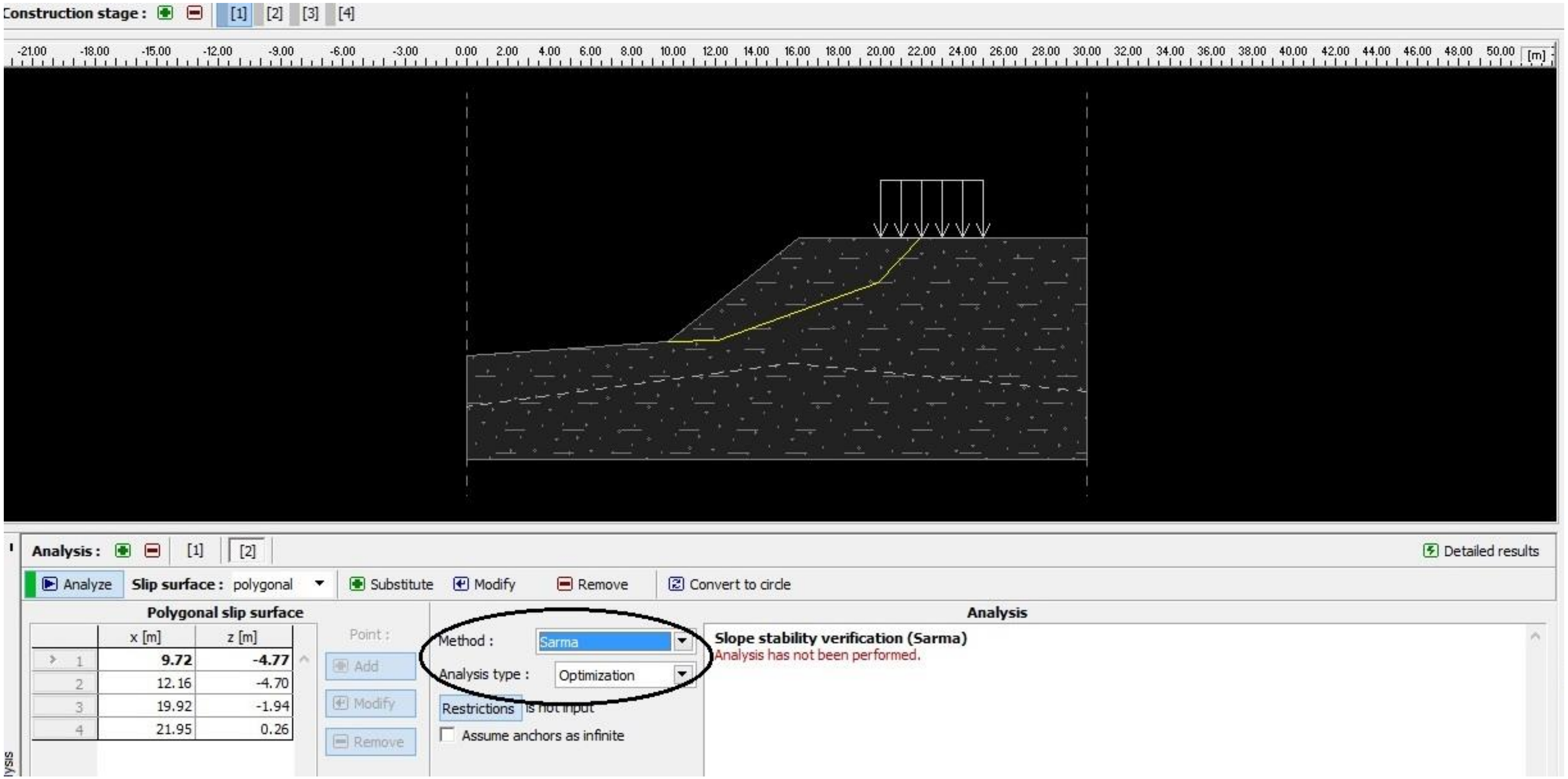


Fig.4.6 Analysis using polygonal slip surface

4.1.6 Anchor or Reinforcement Addition

After analysis if slope fails i.e. if FOS < 1.5 then to stabilize the slope we can either add anchors or reinforcements or both. Other method to stabilize the slope is use of earth cut and embankments to form a pile wall.

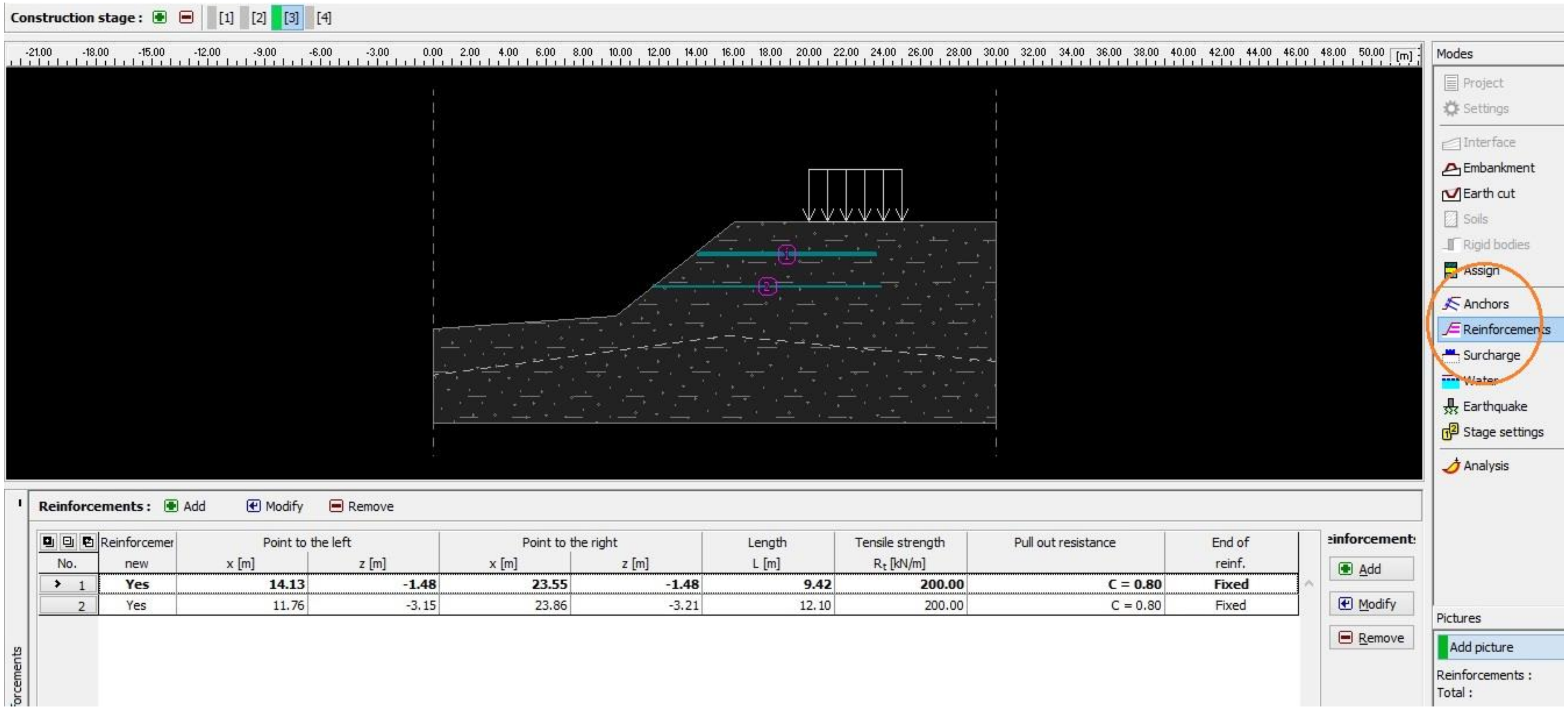


Fig.4.7 Anchors or Reinforcement Addition

4.1.7 Results

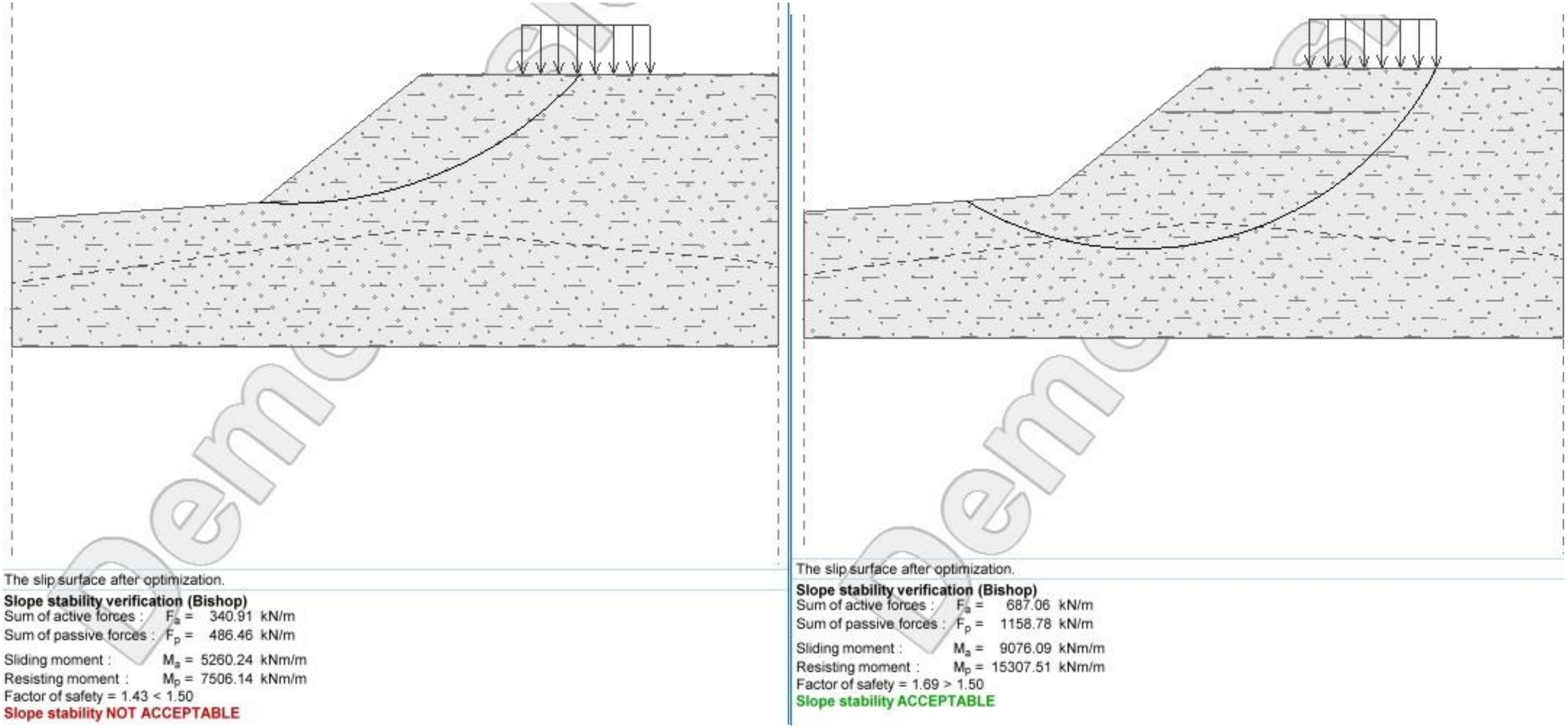


Fig.4.8 Results of Bishop(before and after reinforcement)

Similarly analysis can be performed using other methods.

4.2 Slope Stability Analysis Using FEM

4.2.1 Settings: In settings choose PLAIN STRAIN as Project type and SLOPE STABILITY as Analysis type.

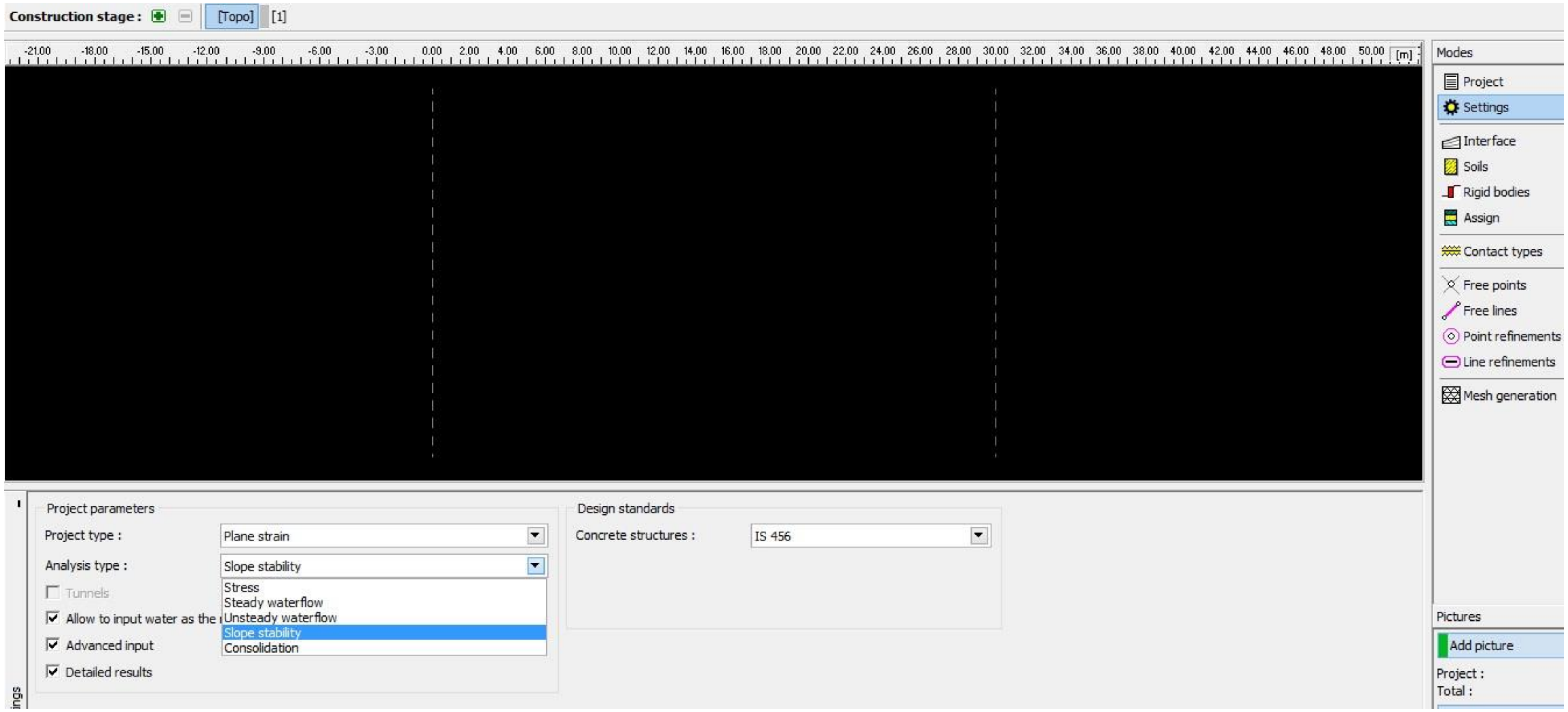


Fig.4.9 FEM module Settings for Slope Stability

4.2.2 Construction Stages

Interface addition and Mesh Generation is done in the “Topo” Construction Stage

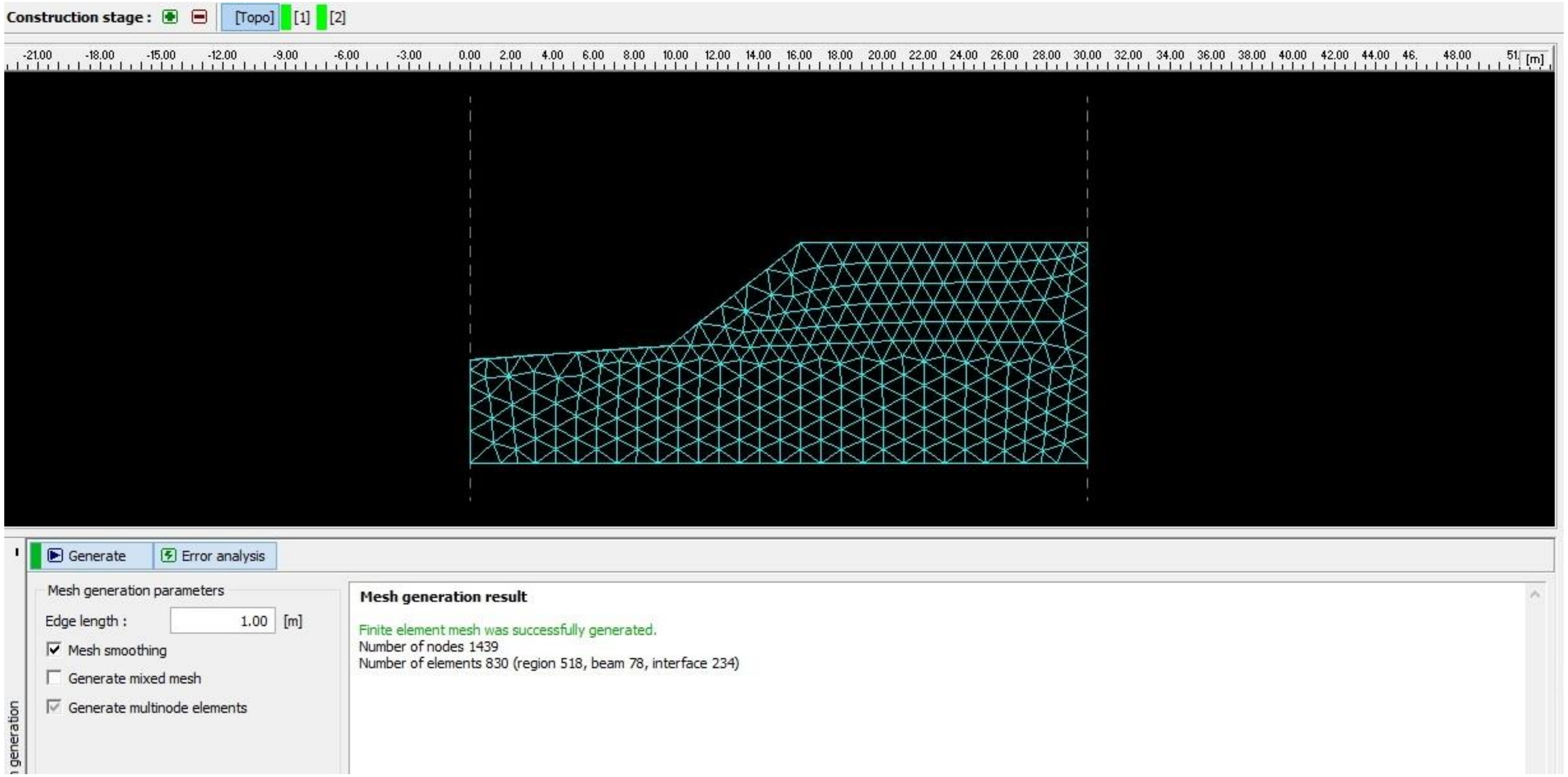


Fig.4.10 Mesh Generation

In the subsequent construction stages other inputs like soil, surcharge, water, etc. are added.

4.2.3 Results: The same inputs were put in the FEM module and analysis was carried out.

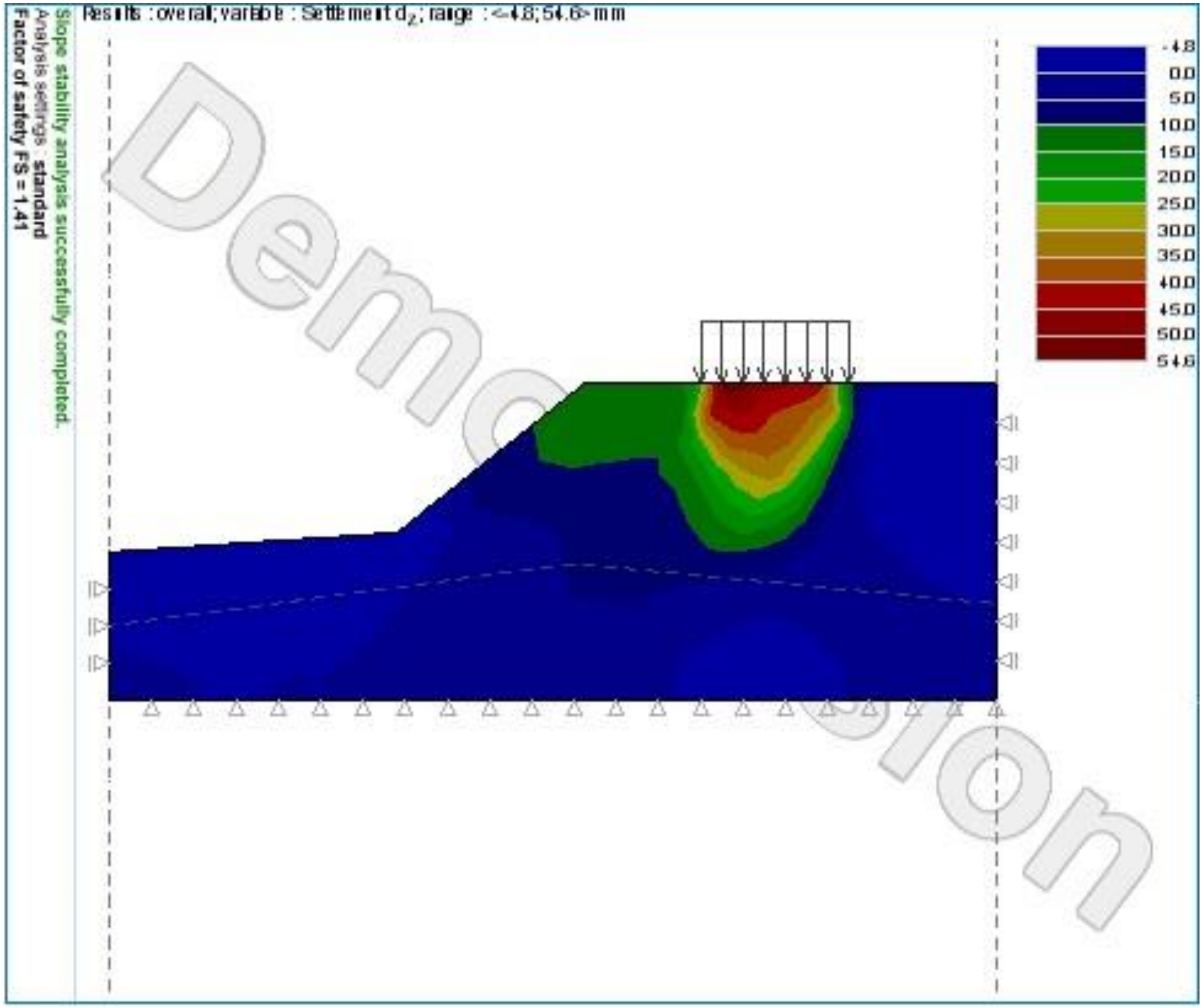


Fig.11

Result of FEM

Showing Settlement in Z direction

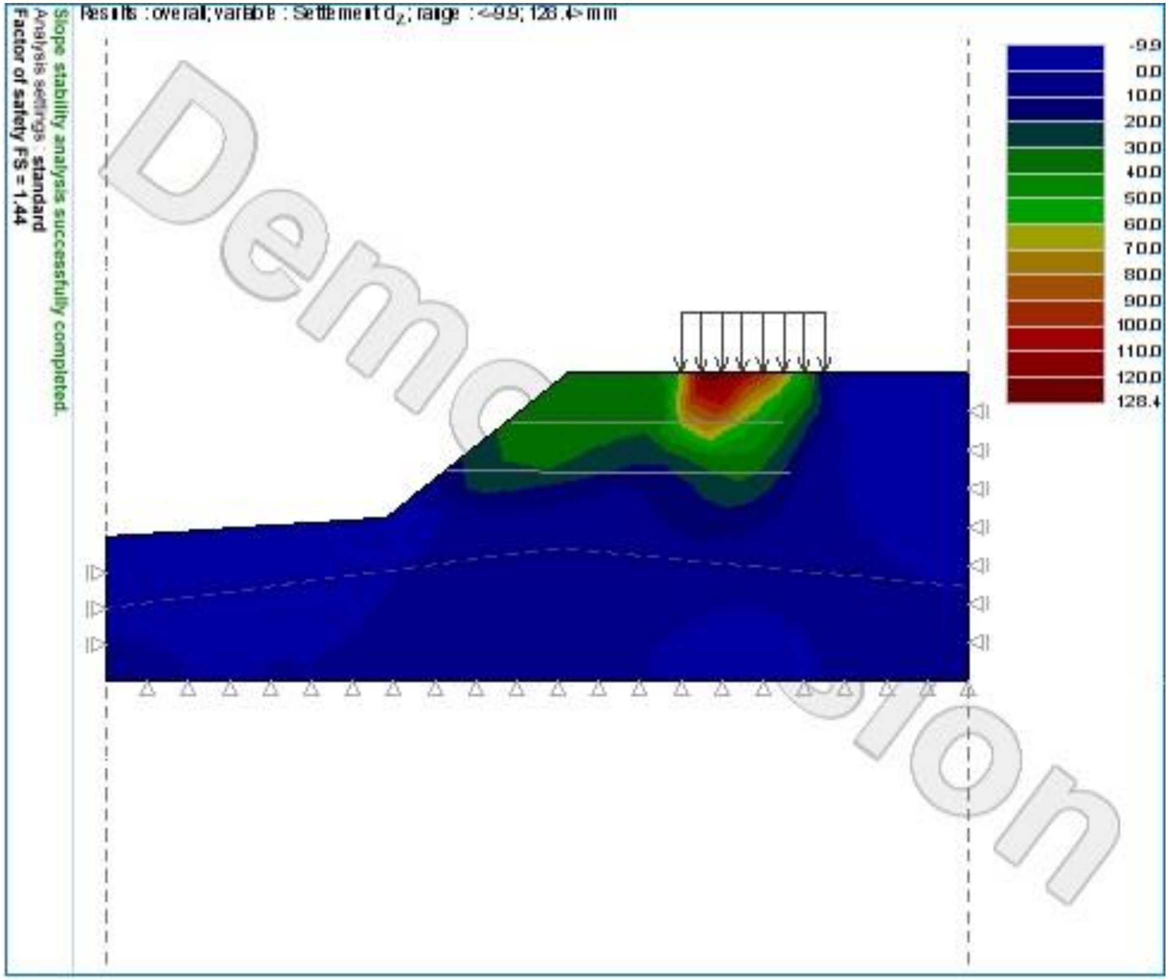


Fig.12

Result of FEM (after reinforcement)

Showing Settlement in Z direction

4.3 Result Comparison

Construction Stage 1 (Before any Reinforcement)

Method	Type Of Slip Surface	Factor Of Safety
Bishop	Circular	1.43
Fellinius/Petterson		1.31
Spencer	Polygonal	1.38
Janbu		1.37
Morgenstern-Price		1.37

Finite Element Method	Mesh Generation	1.41

Construction Stage 2 (After providing Reinforcement)

Method	Type Of Slip Surface	Factor Of Safety
Bishop	Circular	1.69
Fellinius/Petterson		1.44
Spencer	Polygonal	1.68
Janbu		1.66
Morgenstern-Price		1.66

Finite Element Method	Mesh Generation	1.48

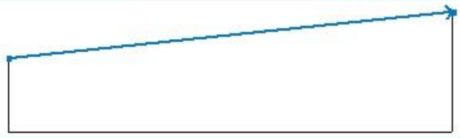
4.4 Analysis of Man-Made Slope

A man made slope was surveyed and analysis was carried out using Slope Stability suite of GEO5 Software.

4.4.1 Inputs

The following inputs were added:

Interface

No.	Interface location	Coordinates of interface points [m]					
		x	z	x	z	x	z
1		0.00	0.00	30.00	3.15		

Soil parameters - effective stress state

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m³]
1	Ramp		28.00	10.00	15.00

Uniform Distributed Load was added as follows:

Surcharge

No.	Surcharge		Type	Type of action	Location	Origin	Length	Width	Slope	Magnitude		
	new	change			z [m]	x [m]	l [m]	b [m]	α [°]	q, q_1, f, F	q_2	unit
1	Yes		strip	permanent	on terrain	x = 0.00	l = 30.00		6.00	1000.00		kN/m²

Earthquake

Horizontal Seismic Coefficient, $K_{h} = 0.1$

Vertical Seismic Coefficient, $K_{v} = 0.05$

4.4.2 Analysis

Circular Slip Surface method (Fellenius/Petterson Method)

Analysis 1 (stage 2)

Circular slip surface

Slip surface parameters							
Center :	x =	-54.38	[m]	Angles :	α_1 =	5.79	[°]
	z =	627.85	[m]		α_2 =	6.20	[°]
Radius :	R =	630.10	[m]				
The slip surface after optimization.							

Slope stability verification (Fellenius / Petterson)

Sum of active forces : F_a = 880.47 kN/m

Sum of passive forces : F_p = 1757.60 kN/m

Sliding moment : M_a = 554786.07 kNm/m

Resisting moment : M_p = 1107465.69 kNm/m

Factor of safety = 2.00 > 1.50

Slope stability ACCEPTABLE

Polygonal Slip Surface method (Sarma Method)

Analysis 2 (stage 2)

Polygonal slip surface

Coordinates of slip surface points [m]									
x	z	x	z	x	z	x	z	x	z
5.80	0.61	6.03	0.62	7.10	0.72	15.49	1.51	27.88	2.93
The slip surface after optimization.									

Slope stability verification (Sarma)
Factor of safety = 1.65 > 1.50
Slope stability ACCEPTABLE

4.4.3 Results

Method	Type of Slip Surface	Factor of Safety
Bishop	Circular Slip Surface	2.01
Fellenius		2
Sarma	Polygonal Slip Surface	1.65
Janbu		1.66
Spencer		1.65
Morgenstren-Price		1.67

4.5 Analysis of Given Soil for Slope

The following data of soil of NCR region was used to solve different problems in GEO5:

DEPTH (m)	I. S. CLASSIFICATION		GRAIN SIZE ANALYSIS			MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %
	DESCRIPTION	HATCHING	GRAVELS %	SAND %	FINES %			
	ML-NP		—	23	77	21.65	—	—
1.5	CLAY OF LOW COMPRESSIBILITY (CL)		—	23	77	22.26	23.5	16.6
			—	22	78	20.95	24.0	11.0
3.0			—	21	79	24.48	23.0	14.3
	POORLY GRADED SILTY SAND (SP-SM)		—	—	—	—	—	—
4.5			—	89	11	15.28	—	—
6.0	POORLY GRADED SAND		—	95	5	20.24	—	—
7.5								
9.0								

4.5.1 Inputs

The following Inputs were added:

Interface

No.	Interface location	Coordinates of interface points [m] x	z	x	z	x	z
1		0.00	0.00	30.00	0.00		
2		0.00	-3.25	30.00	-3.25		
3		0.00	-5.00	30.00	-5.00		
4		0.00	-6.50	30.00	-6.50		

Soil

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m³]
1	Poorly graded sand (SP), dense		35.50	0.00	18.50
2	Sand with trace of fines (S-F), dense		31.50	0.00	17.50
3	Silty sand (SM)		29.00	5.00	18.00
4	Low plasticity clay (CL,CI), consistency soft		19.00	12.00	21.00

Ground Water Table

No.	GWT location	Coordinates of GWT points [m] x	z	x	z	x	z
1		0.00	-9.00	30.00	-9.00		

4.5.2 Analysis and Result

In construction stage 2, earth cut was made and different slopes were analyzed for slope stability and the following result was obtained.

Angle Of Slope	**Method**	**Type Of Slip Surface**	**Factor Of Safety**
45 degrees	Bishop	Circular	0.97
	Fellenius/Petterson		0.94
	Spencer		0.97
	Janbu		No Solution
	Morgernstern-Price		0.98
	Sarma	Polygonal	1.00
	Spencer		1.02
	Janbu		1.04
	Morgernstern-Price		1.00

Angle Of Slope	**Method**	**Type Of Slip Surface**	**Factor Of Safety**
30 degrees	Bishop	Circular	1.46
	Fellenius/Petterson		1.45
	Spencer		1.47
	Janbu		1.47
	Morgernstern-Price		1.47
	Sarma	Polygonal	1.39
	Spencer		1.39
	Janbu		1.37
	Morgernstern-Price		1.37

Angle Of Slope	**Method**	**Type Of Slip Surface**	**Factor Of Safety**
20 degrees	Bishop	Circular	1.66
	Fellenius/Petterson		1.64
	Spencer		1.67
	Janbu		1.67
	Morgernstern-Price		1.67
	Sarma	Polygonal	1.68
	Spencer		1.68
	Janbu		1.69
	Morgernstern-Price		1.67

Chapter 5: Foundation Design Example

5.1 Spread Footing

5.1.1 Profile

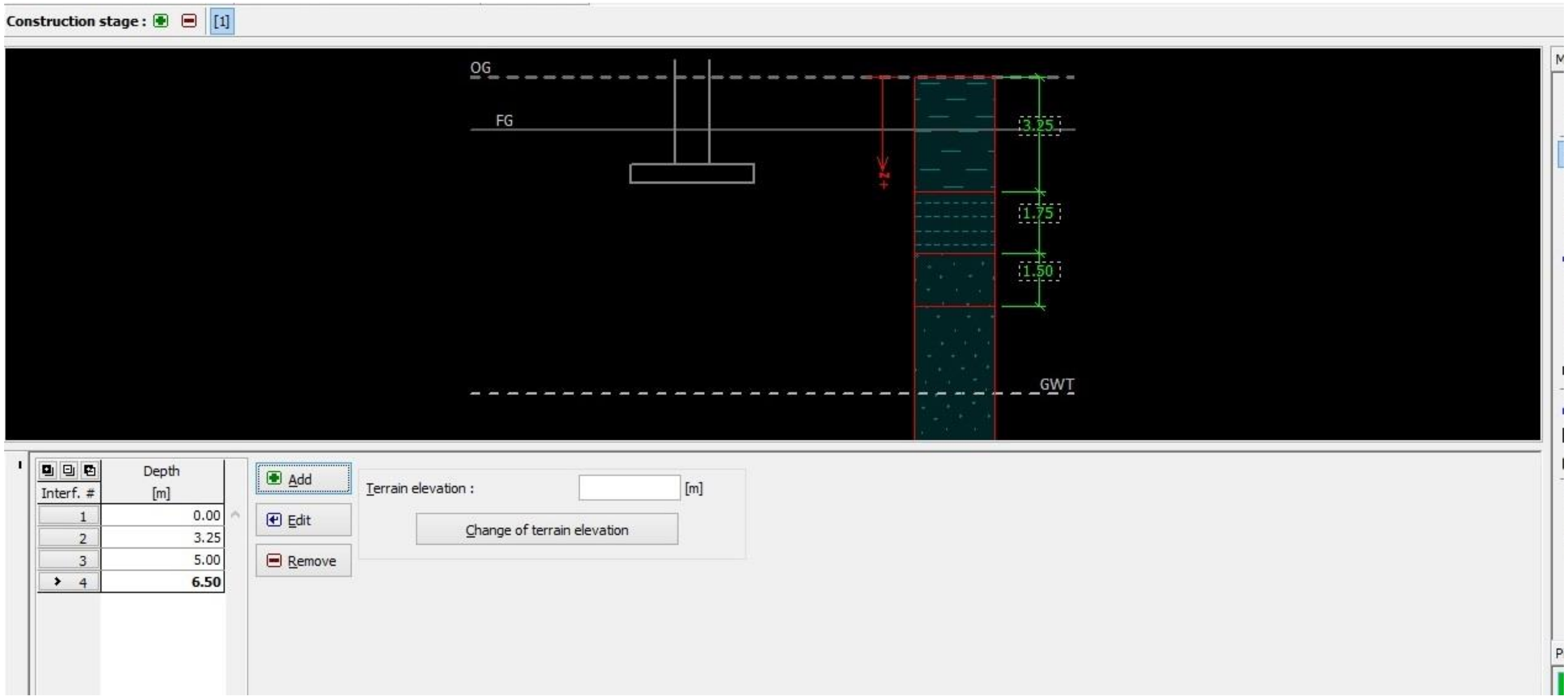


Fig. 5.1 Profile frame of spread footing

5.1.2 Soils

Basic soil parameters

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]	γ_{su} [kN/m^3]	δ [°]
1	Poorly graded sand (SP), medium dense		33.50	0.00	18.50	10.00	
2	Low plasticity clay (CL,CI), consistency soft		19.00	12.00	21.00	12.00	
3	Sand with trace of fines (S-F), dense		31.50	0.00	17.50	9.00	

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	3.25	Low plasticity clay (CL,CI), consistency soft	
2	1.75	Sand with trace of fines (S-F), dense	
3	1.50	Poorly graded sand (SP), medium dense	
4	-	Poorly graded sand (SP), medium dense	

5.1.3 Foundation Type

In the spread footing program the software gives different types of footing to choose from, that are:

- Centric spread footing
- Eccentric spread footing
- Circular stepped spread
- Strip footing
- Centric spread footing with batter
- Stepped centric spread footing
- Eccentric spread footing with batter
- Stepped eccentric spread footing

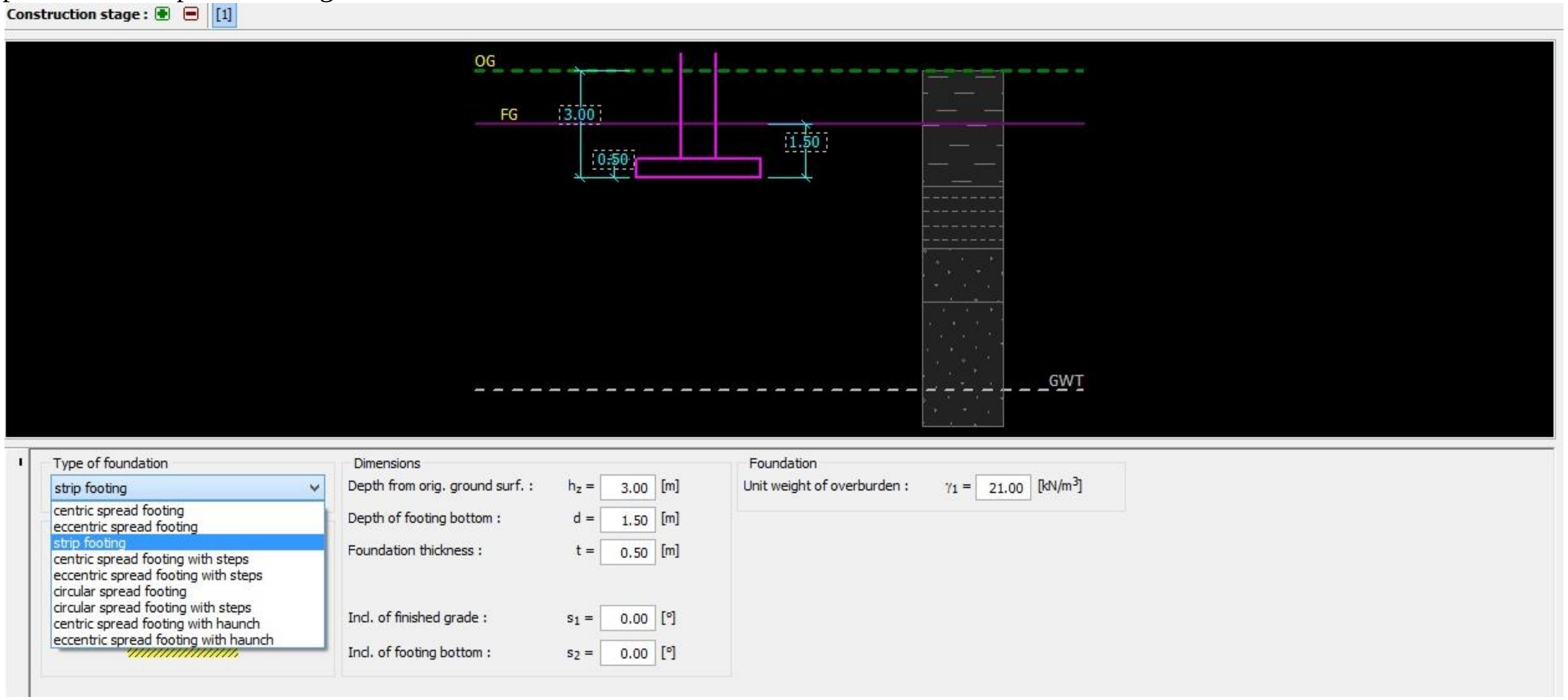


Fig. 5.2 Foundation frame of spread footing

5.1.4 Load

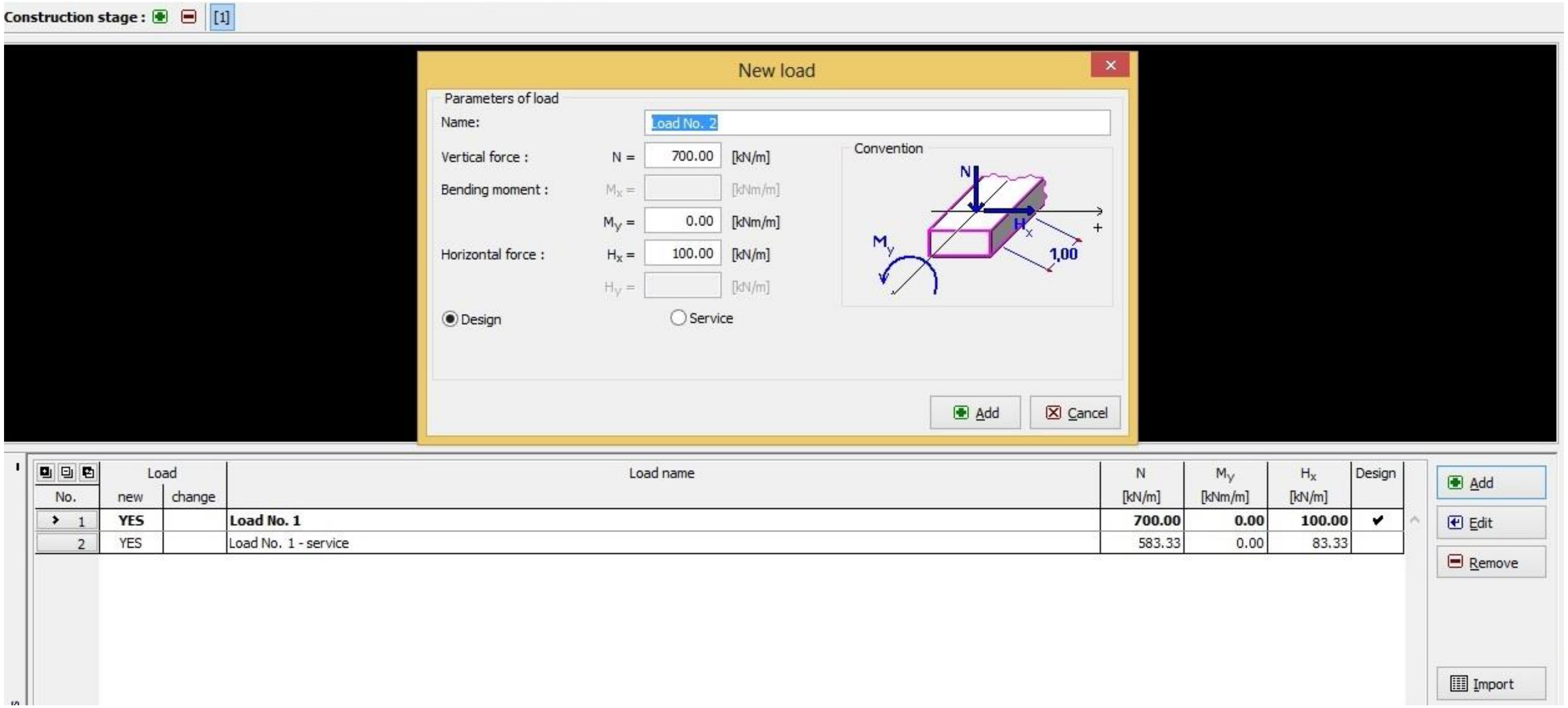


Fig. 5.3 Load frame of spread footing

Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	YES		Load No. 1	Design	700.00	60.00	0.00	100.00	0.00
2	YES		Load No. 1 - service	Service	583.33	50.00	0.00	83.33	0.00

5.1.5 Geometry

The software analyses the problem and gives the dimension of the footing accordingly.

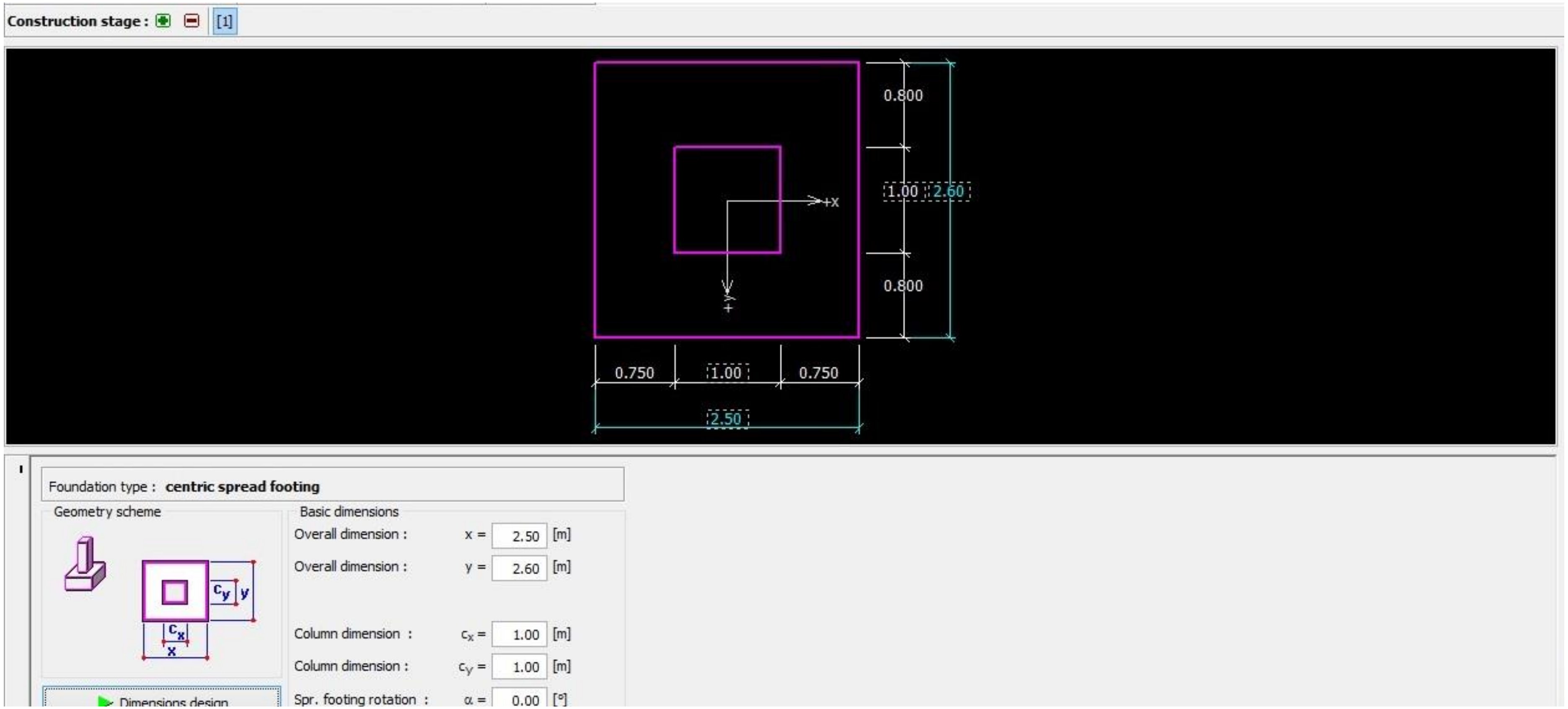
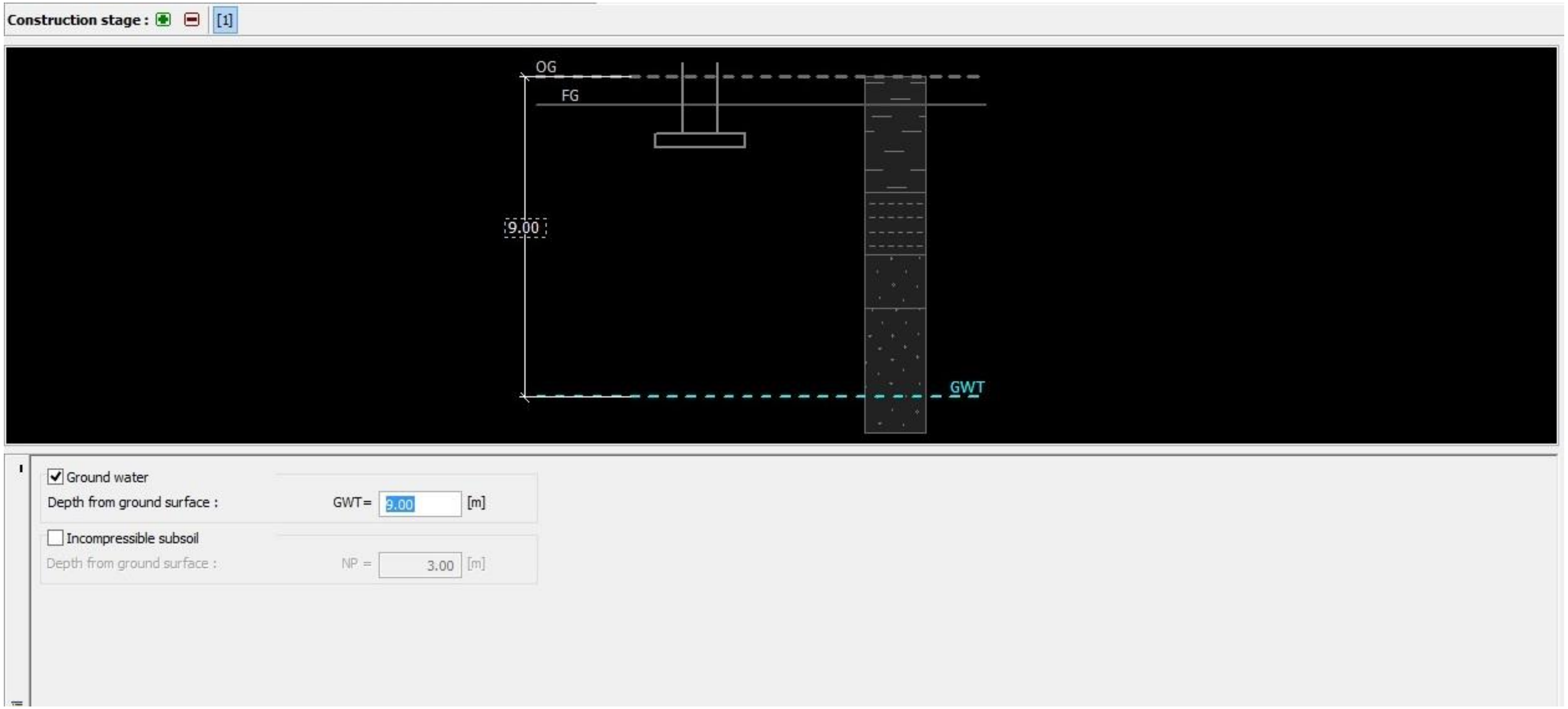


Fig. 5.5 Geometry frame

5.1.6 Ground Water Table



5.1.7 Results

Footing Type	Dimensions	Depth of Footing	Factor Of Safety		Settlement
	(m)	(m)	Vertical	Horizontal	(mm)
Centric Spread Footing	Length=2.5 Width=2.6	1.2	1.54	3.84	16
Eccentric Spread Footing	Length=2.5 Width=2.4	1.2	1.53	3.78	17
Circular Spread Footing	Spread Footing Dia=2.5 Column Dia=0.6	1.5	1.76	3.70	9.3
Strip Footing	Length=4 Width=3.5	1.5	1.53	3.22	16.9

5.2 Piles

5.2.1 Profile

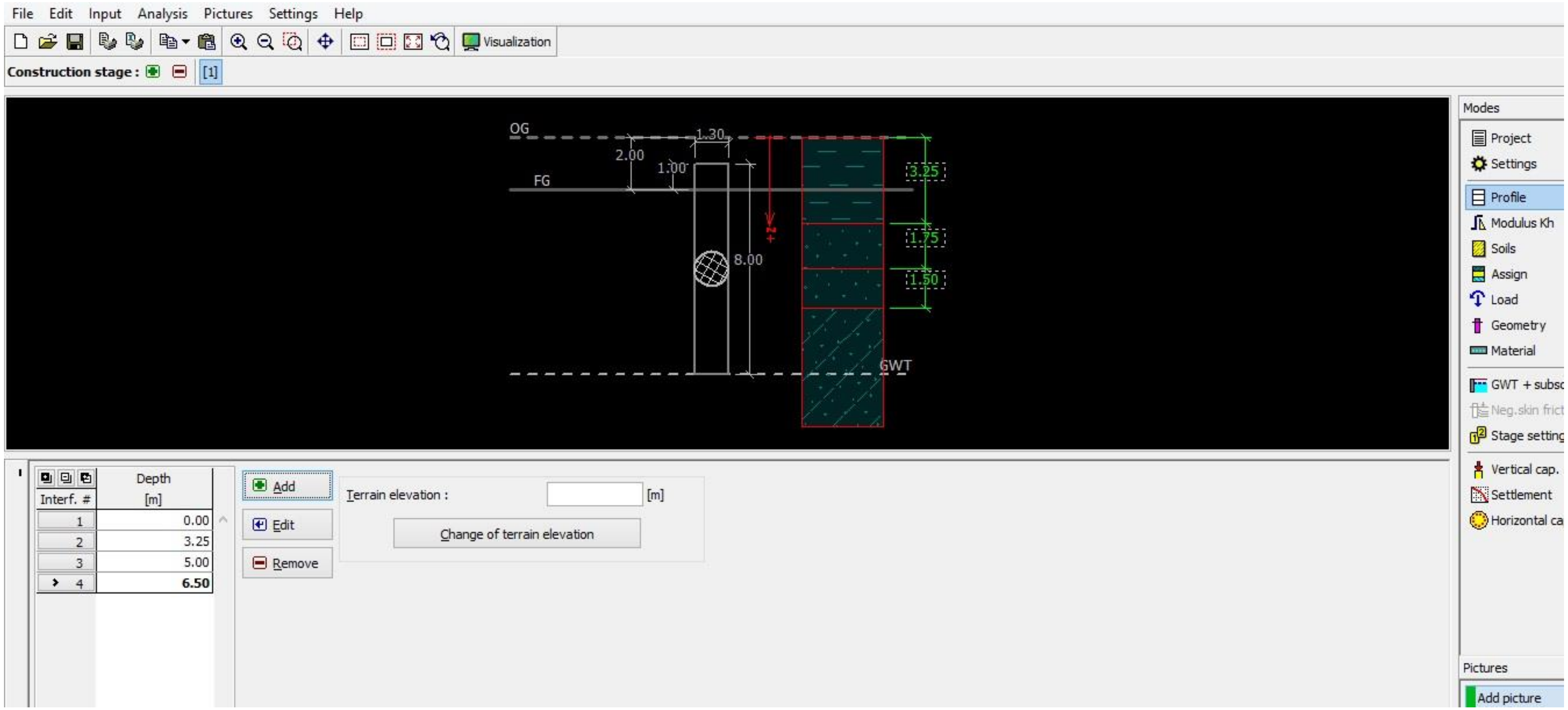


Fig. 5.6 profile frame of piles

5.2.2 Modulus K_h

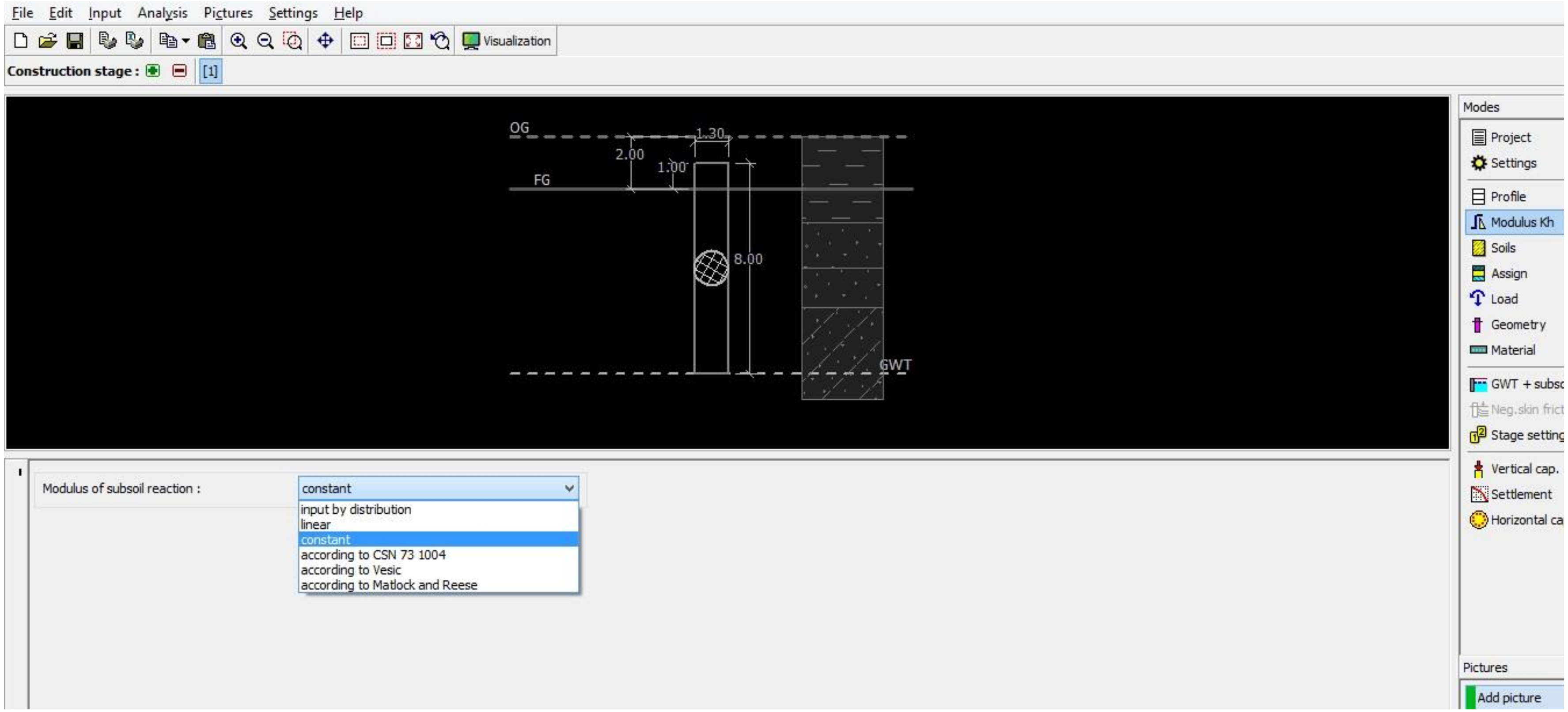


Fig. 5.7 Modulus of subgrade reaction frame

5.2.3 Soil

Basic soil parameters

No.	Name	Pattern	γ [kN/m3]	ν [–]
1	Poorly graded sand (SP), dense		18.50	0.28
2	Sand with trace of fines (S-F), dense		17.50	0.30
3	Silty sand (SM)		18.00	0.30
4	Low plasticity clay (CL,CI), consistency soft		21.00	0.40

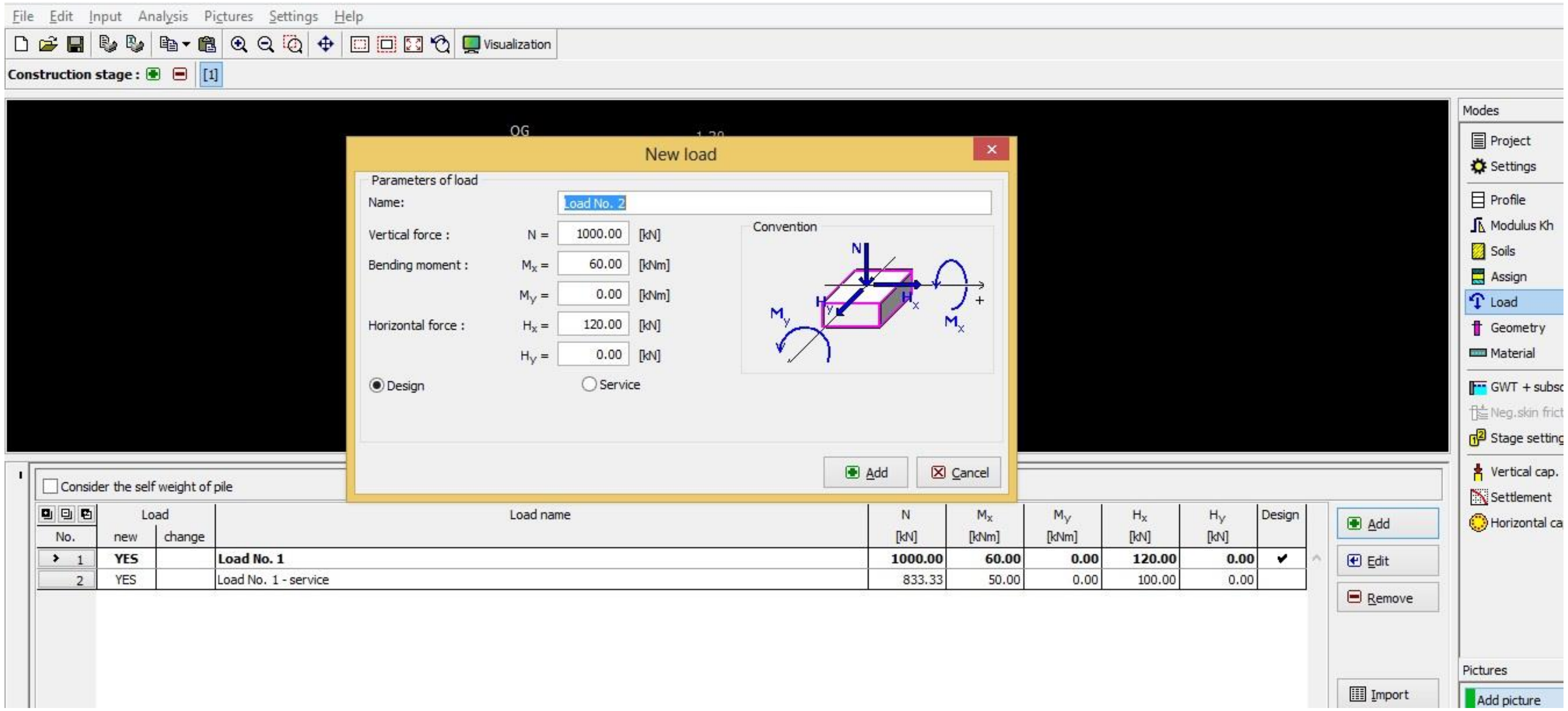
All soils are considered as cohesionless for at rest pressure analysis.

No.	Name	Pattern	E_{oed} [MPa]	E_{def} [MPa]	γ_{sat} [kN/m3]	γ_s [kN/m3]	n [–]
1	Poorly graded sand (SP), dense		51.00	-	20.00	-	-
2	Sand with trace of fines (S-F), dense		28.50	-	19.00	-	-
3	Silty sand (SM)		13.50	-	19.00	-	-
4	Low plasticity clay (CL,CI), consistency soft		4.50	-	22.00	-	-

Parameters of soils to compute modulus of subsoil reaction

No.	Name	Pattern	β
1	Poorly graded sand (SP), dense		12.00
2	Sand with trace of fines (S-F), dense		10.00
3	Silty sand (SM)		10.00
4	Low plasticity clay (CL,CI), consistency soft		8.00

5.2.4 Load



Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	YES		Load No. 1	Design	1000.00	60.00	0.00	120.00	0.00
2	YES		Load No. 1 - service	Service	833.33	50.00	0.00	100.00	0.00

5.2.5 Geometry

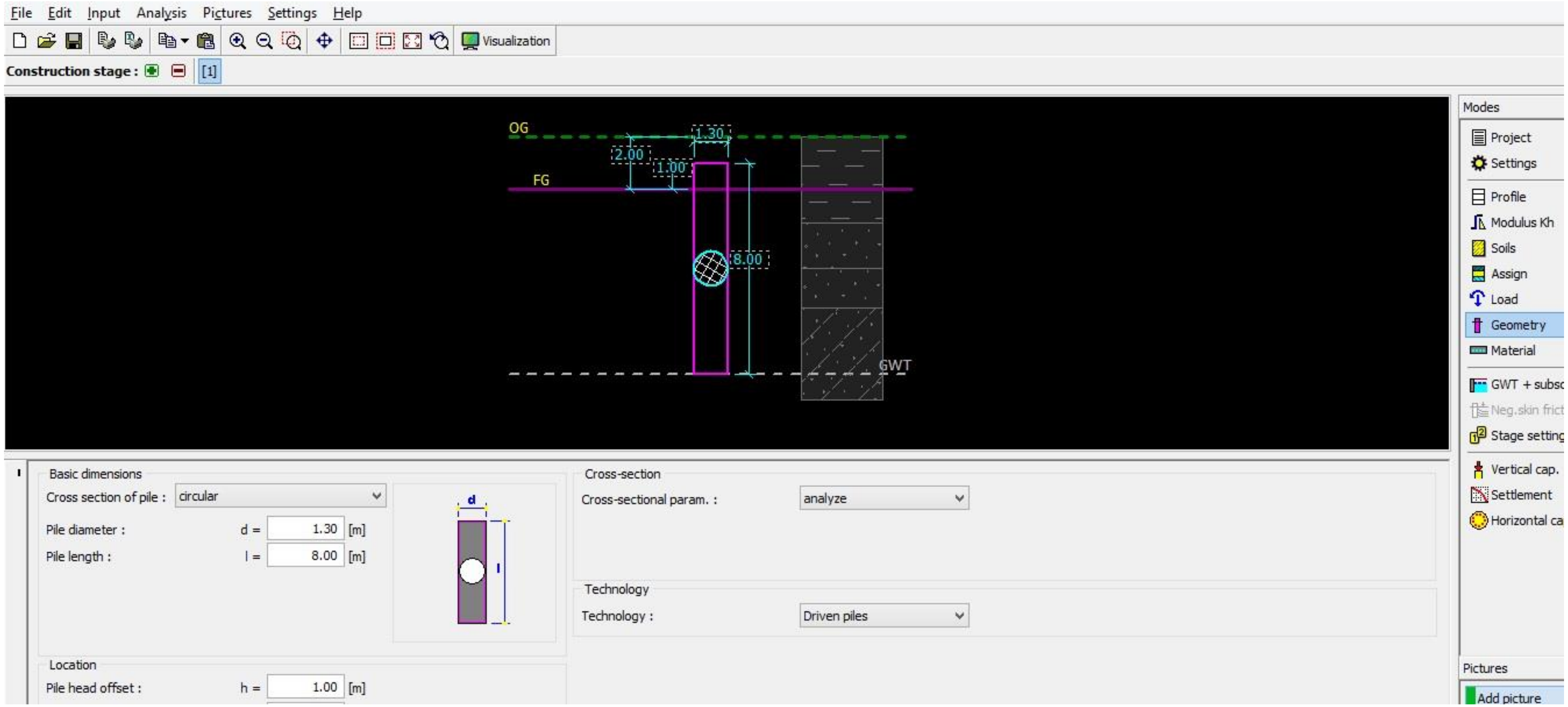
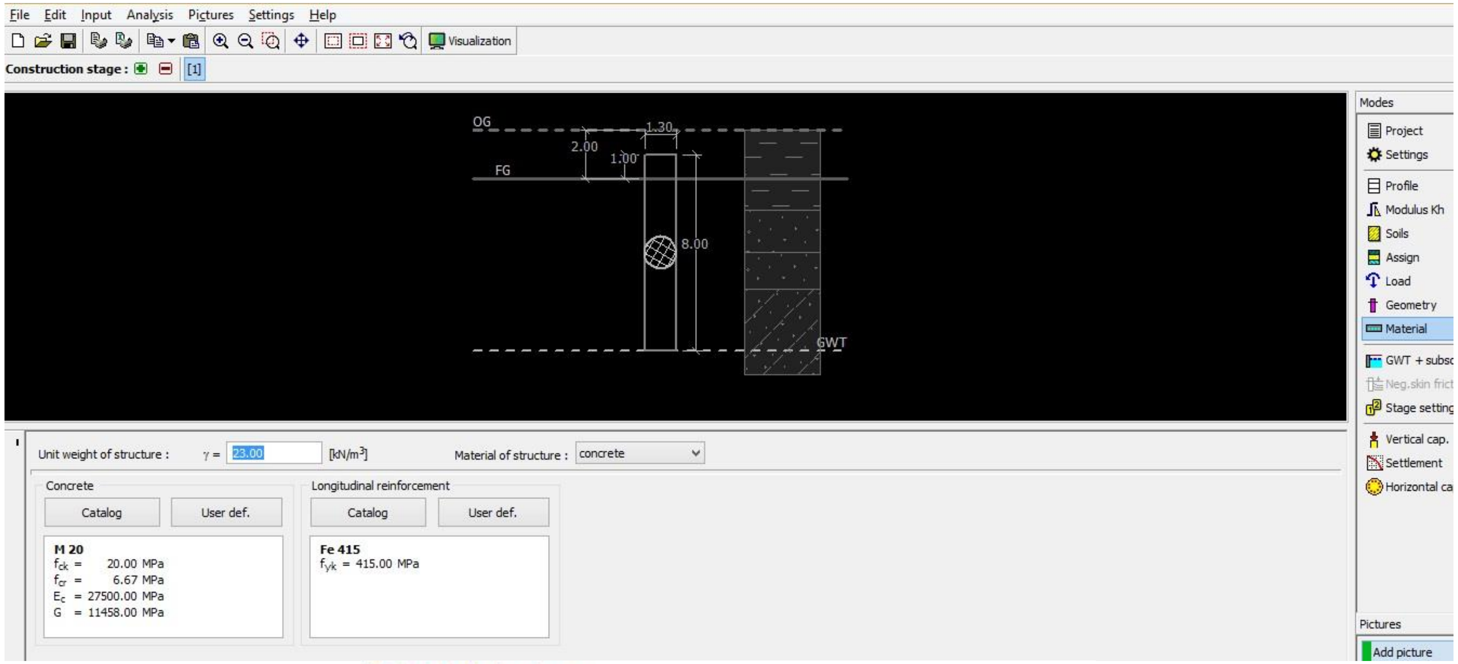


Fig. 5.8 Geometry frame of piles

5.2.6 Material



Material of structure

Unit weight γ = 23.00 kN/m3
Analysis of concrete structures carried out according to the standard IS 456.

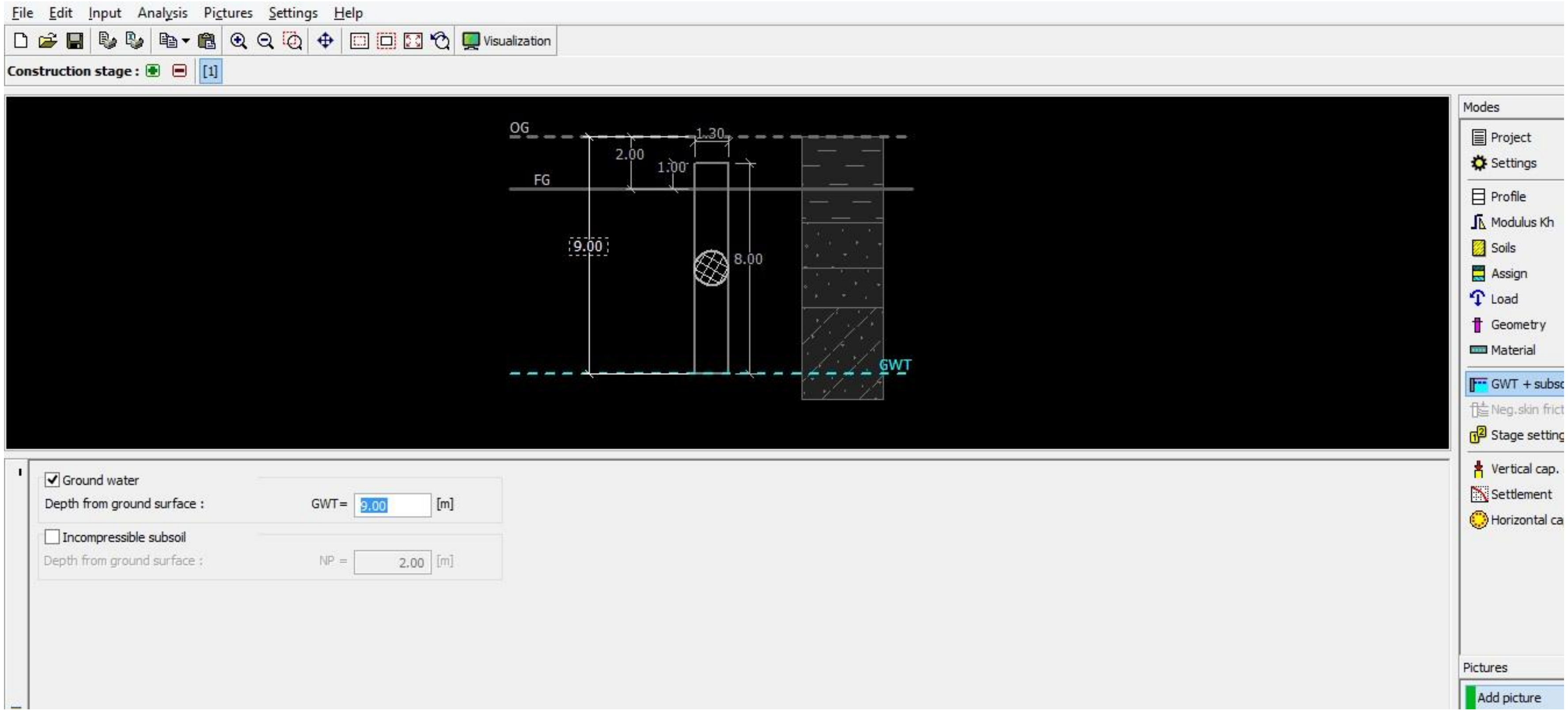
Concrete : M 20

Compressive strength	f_{ck} =	20.00 MPa
Tensile strength	f_{cr} =	6.67 MPa
Elasticity modulus	E_c =	27500.00 MPa
Shear modulus	G =	11458.00 MPa

Longitudinal steel : Fe 415

Tensile strength	f_{yk} =	415.00 MPa

5.2.6 Ground Water Table



5.2.7 Results

Verification of bearing capacity : NAVFAC DM 7.2
Analysis carried out with automatic selection of the most unfavourable load cases.
Factor determining critical depth k_{dc} = 1.00

Pile skin bearing capacity	R_s	=	323.83	kN
Pile base bearing capacity	R_b	=	3096.98	kN
Pile bearing capacity	R_c	=	3420.80	kN
Ultimate vertical force	V_d	=	1000.00	kN

Safety factor = 3.42 > 2.00

Pile bearing capacity is SATISFACTORY

Verification of compressive pile:
Most severe load case No. 1. (Load No. 1)

Load at the onset of mobilization of skin friction	R_{yu}	=	343.90	kN
The settlement for the force R_{yu}	s_y	=	3.2	mm
Total resistance	R_c	=	481.48	kN
Maximum settlement	s_{lim}	=	25.0	mm

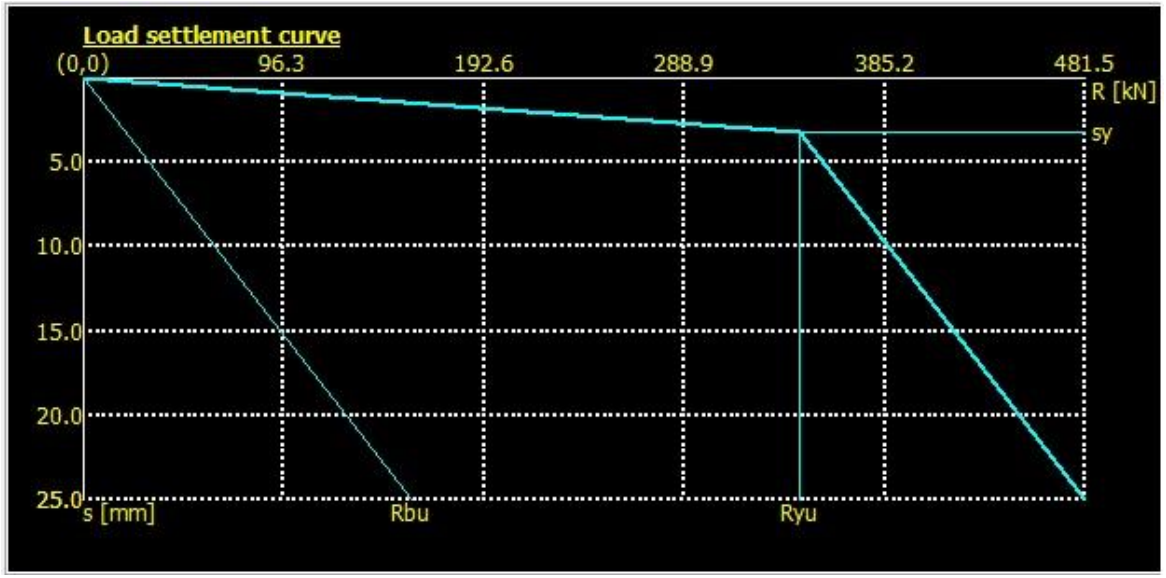
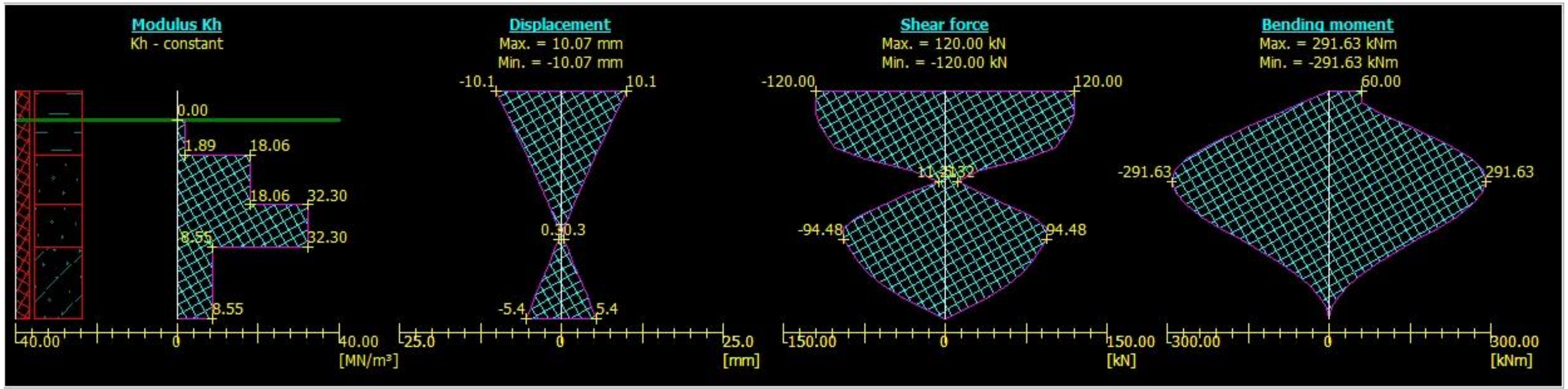


Fig. 5.9 settlement and reinforcement graph of pile



5.3 Pile Group

5.3.1 Structure

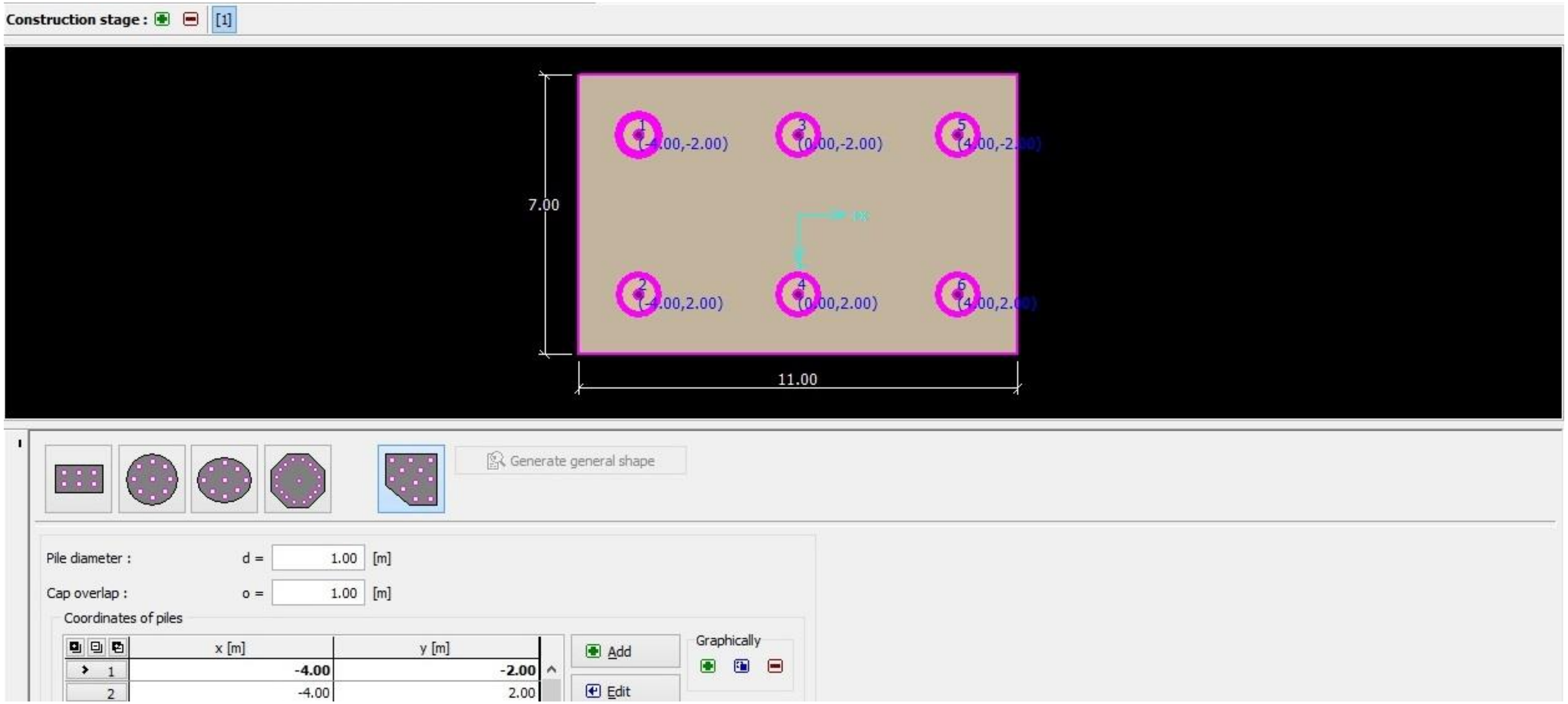


Fig. 5.10 Structure frame of pile group

5.3.2 Construction and Geometry

Construction

Pile diameter d = 1.00 m
Cap overlap o = 1.00 m

Coordinates of piles

No.	x [m]	y [m]
1	-4.00	-2.00
2	-4.00	2.00
3	0.00	-2.00
4	0.00	2.00
5	4.00	-2.00
6	4.00	2.00

Geometry

Depth from ground surface h_z = 2.50 m
Pile head offset h = 0.00 m
Thickness of pile cap t = 0.75 m
Length of piles l = 10.00 m

5.3.3 Material

Material of structure

Unit weight γ = 23.00 kN/m^3
Analysis of concrete structures carried out according to the standard IS 456.

Concrete : M 20	
Compressive strength	f_{ck} = 20.00 MPa
Tensile strength	f_{cr} = 3.13 MPa
Elasticity modulus	E_c = 22361.00 MPa
Shear modulus	G = 9317.00 MPa
Longitudinal steel : Fe 415	
Tensile strength	f_{yk} = 415.00 MPa

5.3.4 Soil and Load

Soil parameters

Poorly graded sand (SP), dense

Unit weight :	γ	=	18.50 kN/m^3
Oedometric modulus :	E_{oed}	=	51.00 MPa
Saturated unit weight :	γ_{sat}	=	20.00 kN/m^3
Coefficient of lateral stress :	K	=	1.00

Sand with trace of fines (S-F), dense

Unit weight :	γ	=	17.50 kN/m^3
Oedometric modulus :	E_{oed}	=	28.50 MPa
Saturated unit weight :	γ_{sat}	=	19.00 kN/m^3
Coefficient of lateral stress :	K	=	1.00

Silty sand (SM)

Unit weight :	γ	=	18.00 kN/m^3
Oedometric modulus :	E_{oed}	=	13.50 MPa
Saturated unit weight :	γ_{sat}	=	19.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	29.00 °

Low plasticity clay (CL,CI), consistency soft

Unit weight :	γ	=	21.00 kN/m^3
Oedometric modulus :	E_{oed}	=	4.50 MPa
Saturated unit weight :	γ_{sat}	=	22.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	19.00 °

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	3.25	Low plasticity clay (CL,CI), consistency soft	
2	1.75	Sand with trace of fines (S-F), dense	
3	1.50	Poorly graded sand (SP), dense	
4	-	Silty sand (SM)	

Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]	M_z [kNm]
1	YES		Load No. 1	Design	1500.00	0.00	0.00	100.00	0.00	0.00
2	YES		Load No. 1 - service	Service	1250.00	0.00	0.00	83.33	0.00	0.00

Ground water table

The ground water table is at a depth of 9.00 m from the original terrain.

5.3.5 Result

Analysis of bearing capacity of pile group in cohesion less soils
Max. Vertical force includes self-weight of pile cap.

Pile skin bearing capacity	R_s	=	289.02	kN
Pile base bearing capacity	R_b	=	1064.71	kN
Vertical bearing capacity of single pile	R_c	=	1353.73	kN
Efficiency of pile group	η_g	=	0.70	
Vertical bearing capacity of pile group	R_g	=	5685.66	kN
Maximum vertical force	V_d	=	2828.25	kN

Safety factor = 2.01 > 2.00

Vertical bearing capacity of pile group is SATISFACTORY

Analysis of settlement of pile group in cohesionless soils
Max. vertical force includes self-weight of pile cap.

Group settlement factor	gf	=	2.65	
Load at the onset of mobilization of skin friction	R_{yu}	=	1734.13	kN
The settlement for the force Ryu	s_y	=	7.0	mm
Total resistance	R_c	=	1734.13	kN
Maximum settlement	s_{lim}	=	13.9	mm

The settlement for maximum service load V = 1250.00 kN is 5.0 mm.

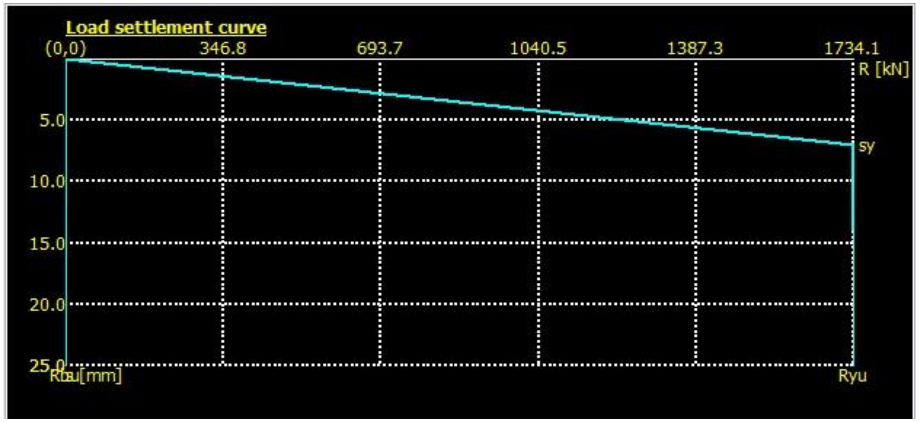


Fig. 5.11 Settlement graph of pile group

Conclusion

GEO5 software is an easy -to -use suite designed to solve various geotechnical problems. Slope Stability Analysis can be done easily and efficiently using Geo5.

In the Stability Analysis suite we can easily draw the interface and add soil accordingly. Embankments, earth cuts, rigid body, surcharge and water table can also be introduced accordingly. Earthquake settings can also be done if the slope analyzed is present in an earthquake zone.

Construction can be done in different stages so that it's easy to edit later. Analysis can be done using either circular slip surface or polygon slip surface and the software can optimize the slip surface.

If the slip surface fails i.e. factor of safety is less than 1.5, the software gives the option of adding either anchors or soil reinforcements. Also we can design pile wall using earth cut and embankments. Then again analysis can be performed to see if the slope is stable now or not.

Slope Stability Analysis can also be performed using FEM suite of GEO5. In FEM module generates mesh and analyses the slope by reducing soil parameters "c" and "φ".

After optimization of circular slip surface, by seeing the results, we can say that Fellenius/Petterson method is safer as it gives least factor of safety. While in case of polygonal slip surface Janbu method is safest.

If our soil profile allows the use of FEM module of GEO5 then it is better than the analytical methods which are used in Slope Stability, FEM is better because it does not make any assumptions like the analytical methods.

The foundation design suite of GEO5 gives option of different spread footings, piles, pile group and micropile. Any of the programs can be used to design the foundation based upon the loading and type of soil on which foundation has to be made.

In all of the foundation design suites interface and soil are added accordingly and then the load. Then the software optimizes the dimensions of foundation according to the bearing capacity of soil.

The results are found out in terms of bearing capacity and it also gives the settlement of footing for the given service load.

Future Scope

Stability of slopes, natural and manmade, is particularly important for any hill road. Disturbance to slope can occur due to erosion caused by rainfall and run-off and consequent slides.

Geo5 Software is vast software and only two of its program is used in this project i.e. Slope Stability and Foundation Design. The understanding of the full software and using it in Design and Analytical problems is wide scope of research.

Using software's in solving geotechnical problems is itself a wide scope of research and GEO5 is a small part of it.

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