SETAF2018

Geotechnical Engineering

User Manual

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1. SOFTWARE FEATURES

SETAF2018 is a software developed for comprehensive analysis, design, drafting, and reporting in the field of geotechnical engineering. It offers a wide range of solutions, from foundation and excavation support systems to slope stability analyses, from laboratory tests to engineering reporting. With its user-friendly interface and robust computational infrastructure, it enables engineers to complete their projects reliably and efficiently.

General Features:

- Geotechnical analysis, design, project drafting, calculations, and quantity takeoff
- Settlement and bearing capacity analysis of shallow and deep foundations
- Modeling, analysis, and design of excavation support systems with bored piles, anchors, soil nails, and reinforced concrete retaining walls
- Slope and embankment stability analyses (using limit equilibrium methods)
- Liquefaction analysis
- Generation of detailed tables, charts, and engineering calculations for geotechnical reports

Foundation Systems:

- **Analysis, design, and drafting of:**
    - Shallow foundations (raft, isolated, and strip footings)
    - Pile and micropile foundations
    - Deep mixing column systems
    - Jet grouting column systems
    - Stone column applications

Excavation Support Systems:

- **Modeling, analysis, design, and drafting of:**

- Bored pile walls
- Anchored excavation support structures
- Soil nail retaining structures
- Reinforced concrete retaining walls

Slope and Embankment Stability Analyses:

- **Analysis, design, and safety assessment:**
  - Stability analysis using limit equilibrium methods
  - Determination of slip surfaces
  - Calculation of safety factor

Laboratory Tests:

- **Experimental analysis to determine geotechnical parameters:**
  - Consolidation test

Project Drawings and Quantity Takeoff:

- **Automated engineering drawings and section generation:**
  - Detailed drawings of foundation systems
  - Sections and reinforcement details of excavation support systems
  - Graphical evaluations for slope stability analyses
  - Generation of drawing and quantity takeoff outputs

Reporting and Visualization:

- **Detailed technical reports and engineering outputs:**
  - Engineering tables and charts, including analysis results
  - Technical report generation and output export
  - Project-based data management

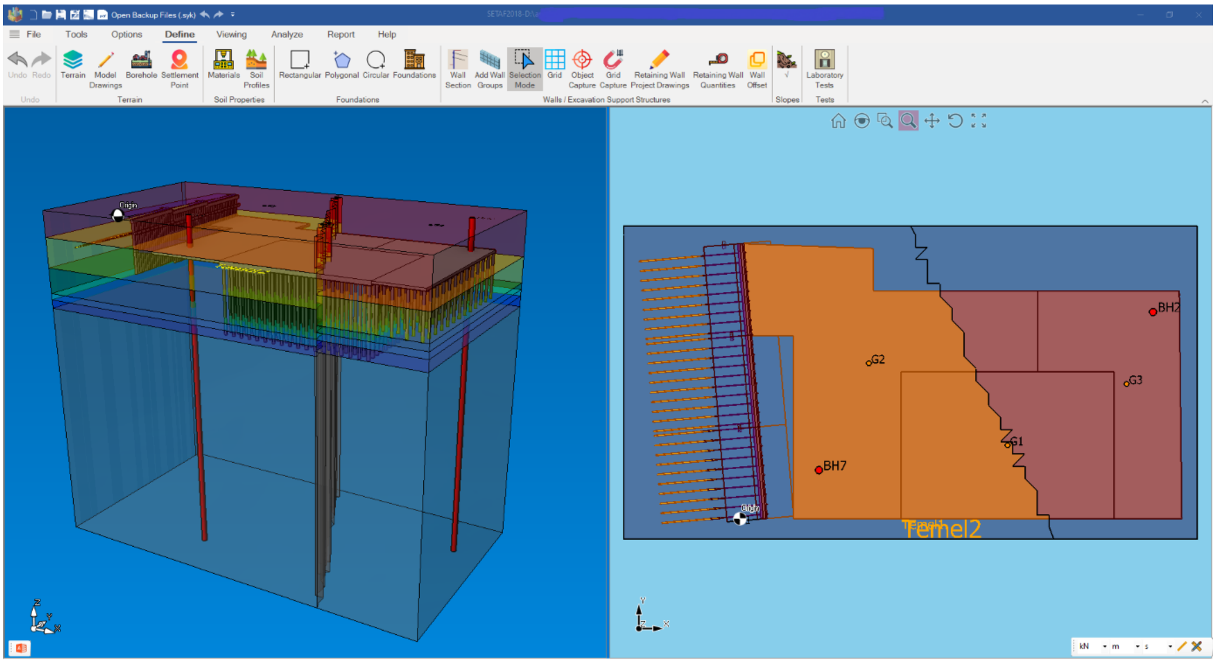


Figure 1. Program Interface

1.1. System Requirements

To ensure stable and efficient operation of SETAF2018, the following minimum system requirements must be met:

- **Operating System:** Windows 10 (64-bit) or later
- **Memory (RAM):** Minimum 16 GB
- **Screen Resolution:** At least 1920 x 1080
- Internet Connection

These requirements are defined to ensure optimal performance of the program. Higher hardware specifications will improve analysis times and processing speed.

1.2. Software Package

The SETAF2018 installation package includes:

- SETAF2018 installation files
- LicenseManager software
- SupportCenter software

- User manual
- License agreement

UserID and key values are sent via email along with the license card. Please store them securely and do not share them. Make sure to read the license agreement carefully.

1.3. General Features

SETAF2018 supports user-defined data inputs, material properties, boreholes, foundation systems, excavation support structures, and slope/hillside models. Cross-section definitions necessary for slope-hillside analyses can be made. If no changes are made in the relevant input windows, the program will use default values. Users can perform data entries using their preferred units of force, length, and time (Figure 2).

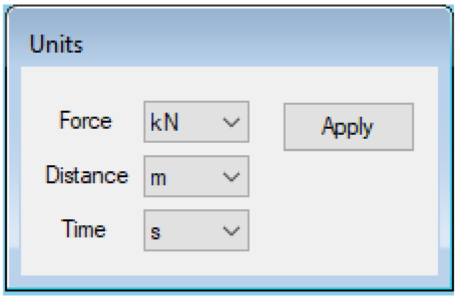


Figure 2. Units

Modeling and Visualization

Defined boreholes, foundations, retaining walls, and settlement points can be examined in the program's visual interface through perspective (3D) and plan (2D) views. Functions such as zoom, zoom extents, magnifier, and pan can be used for detailed inspection.

Within a model, multiple foundations and retaining walls can be defined. Rectangular foundations can be modeled as dilated, isolated, or continuous systems. Systems with piles, shotcrete, well-type, and diaphragm walls, as well as retaining walls with reinforced concrete, steel strand anchors, and soil nails, can be detailed. The program can automatically generate project drawings for these models and provide outputs to the user.

2. MATERIALS

To configure the material assignment options, open the **Program Options** window from the **Settings** menu. Under the **Material Assignment** tab in this window, there are two options available. By default, the **User-Defined Material Assignment** option is active (Figure 3).

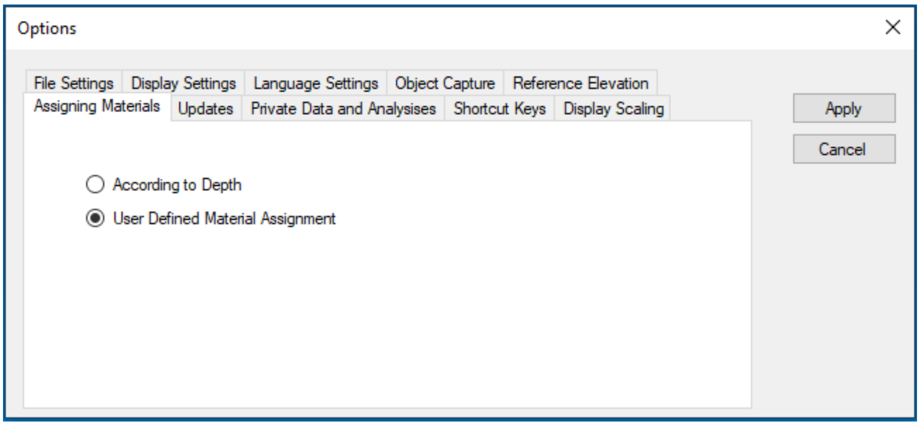


Figure 3. Material Assignment Options

- Depth-Based Material Assignment
  The drillhole number and depth information for assigning the defined material must be entered in the relevant window (Figure 4).
- User-Defined Material Assignment
  In this option, the defined material is not linked to any specific drillhole or depth. When the **User-Defined Material Assignment** option is enabled, the drillhole number and depth input fields (highlighted with a red box in Figure 4) are not displayed in the corresponding window.

The soil layer to which the material will be assigned is defined during the soil profile creation stage in the **Layer Properties** window (Figure 5).

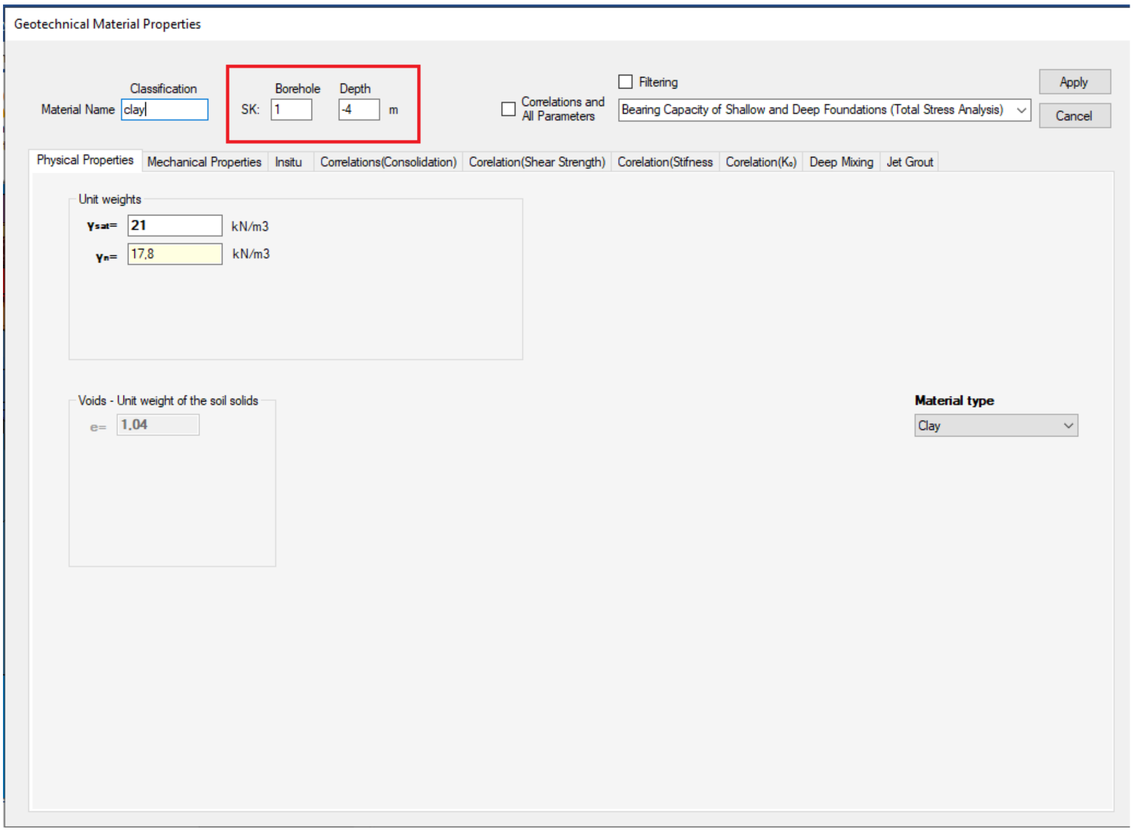


Figure 4. Depth-Based Material Assignment

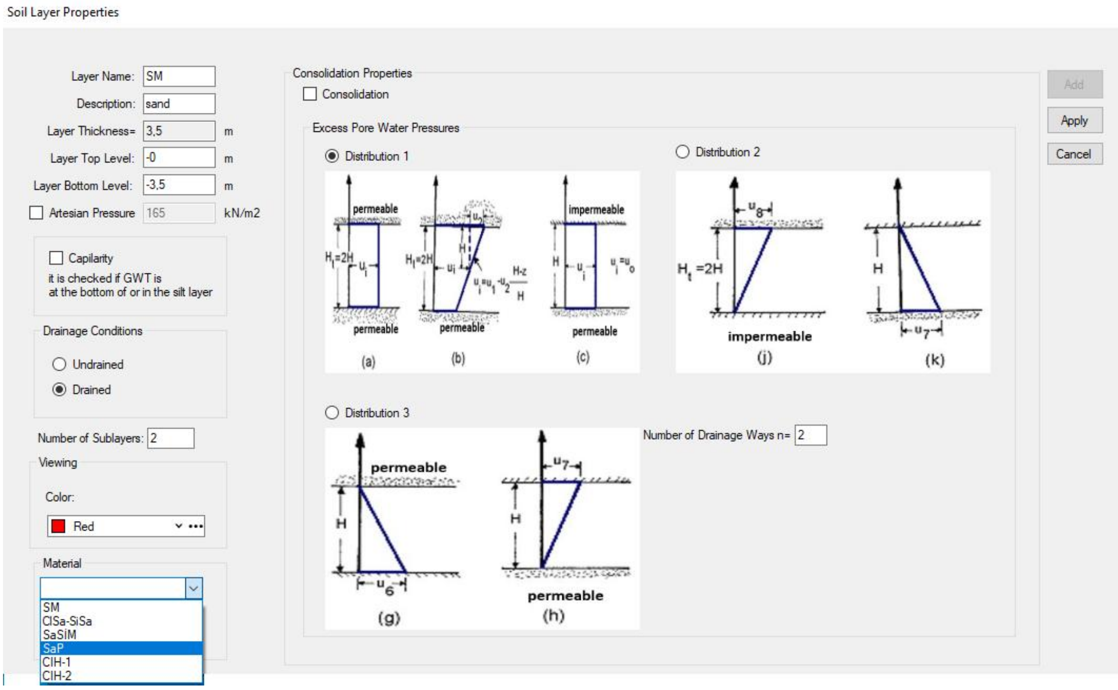


Figure 5. User-Defined Material Assignment

Once the material assignment option is selected, any number of materials can be defined using the **Material Definition** menu (Figure 6).

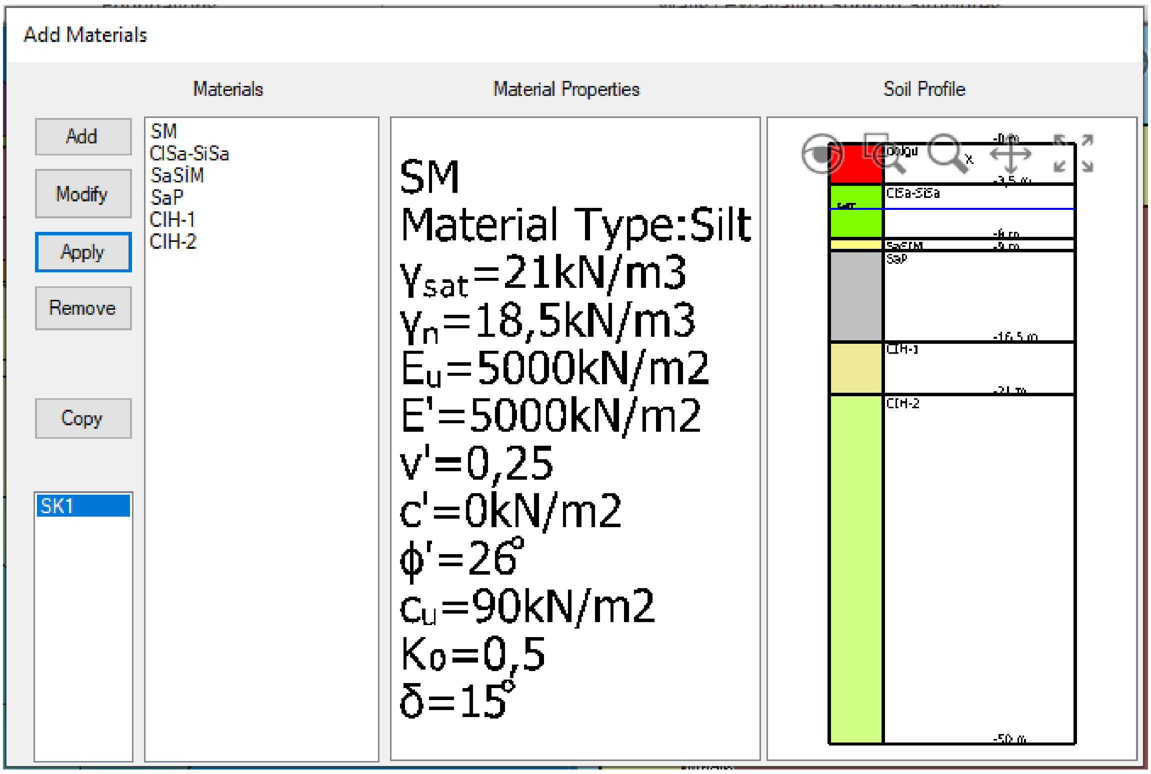


Figure 6. Add Material

2.1. Physical Properties

In the **Physical Properties** tab, physical parameters such as saturated, natural, and dry unit weights, natural water content, liquid limit, plastic limit, and shrinkage limit can be defined (Figure 7).

Parameters Defined in the Physical Properties Tab:

γ_{sat}: Saturated unit weight

γ_n: Natural unit weight

γ_d: Dry unit weight

γ': Submerged unit weight

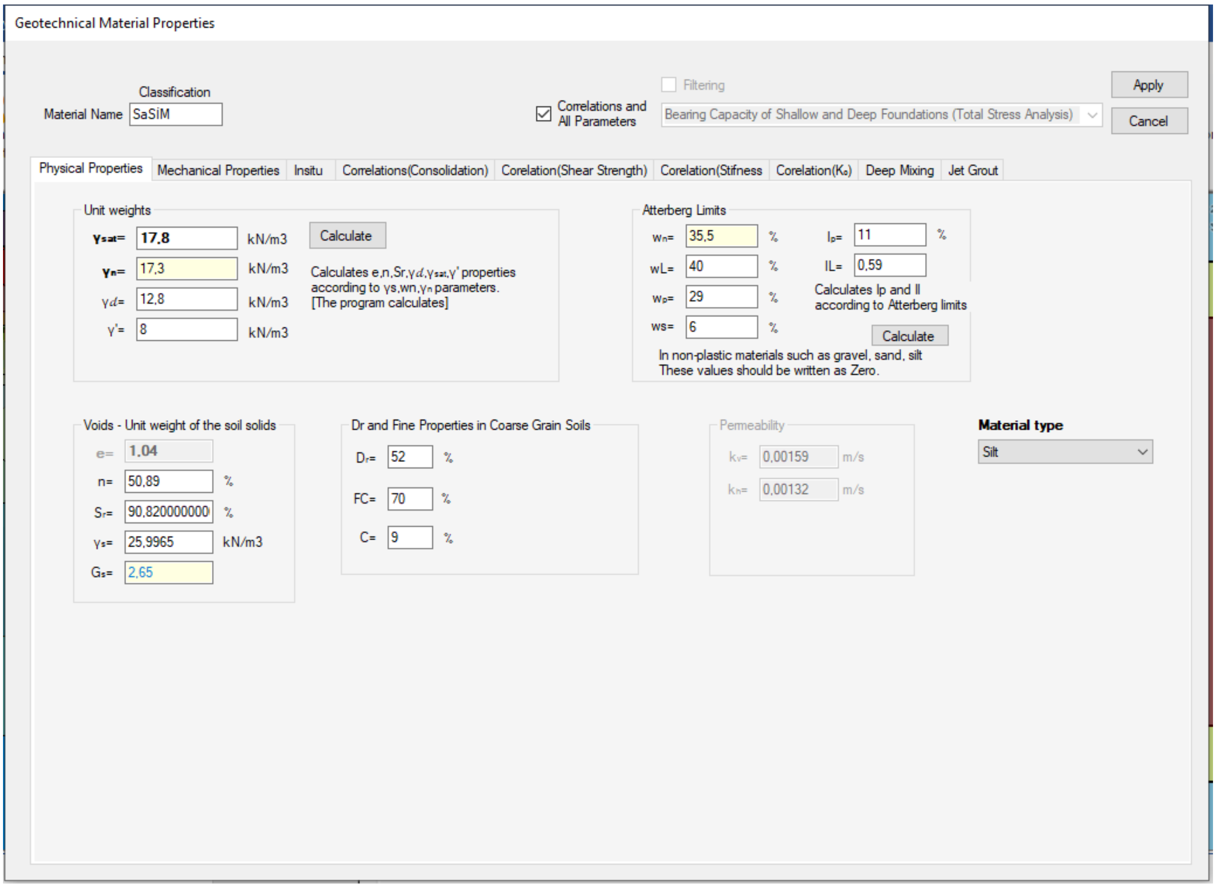


Figure 7. Defining Material Properties

w_n: Natural water content

w_L: Liquid limit

w_P: Plastic limit

w_s: Shrinkage limit

I_p: Plasticity index

I_L: Liquidity index

e: Void ratio

n: Porosity

S_r: Degree of saturation

γ_s: Unit weight of solids

D_r: Relative density

FC: Fines content

k_v: Vertical permeability

k_h: Horizontal permeability

The program can automatically calculate parameters such as void ratio, porosity, degree of saturation, saturated unit weight, dry unit weight, submerged unit weight, plasticity index, and liquidity index using the **Calculate** button. Alternatively, the user can manually enter these values if desired.

2.2. Mechanical Properties

In the **Mechanical Properties** tab, values related to total stress, effective stress, consolidation, and compaction are entered.

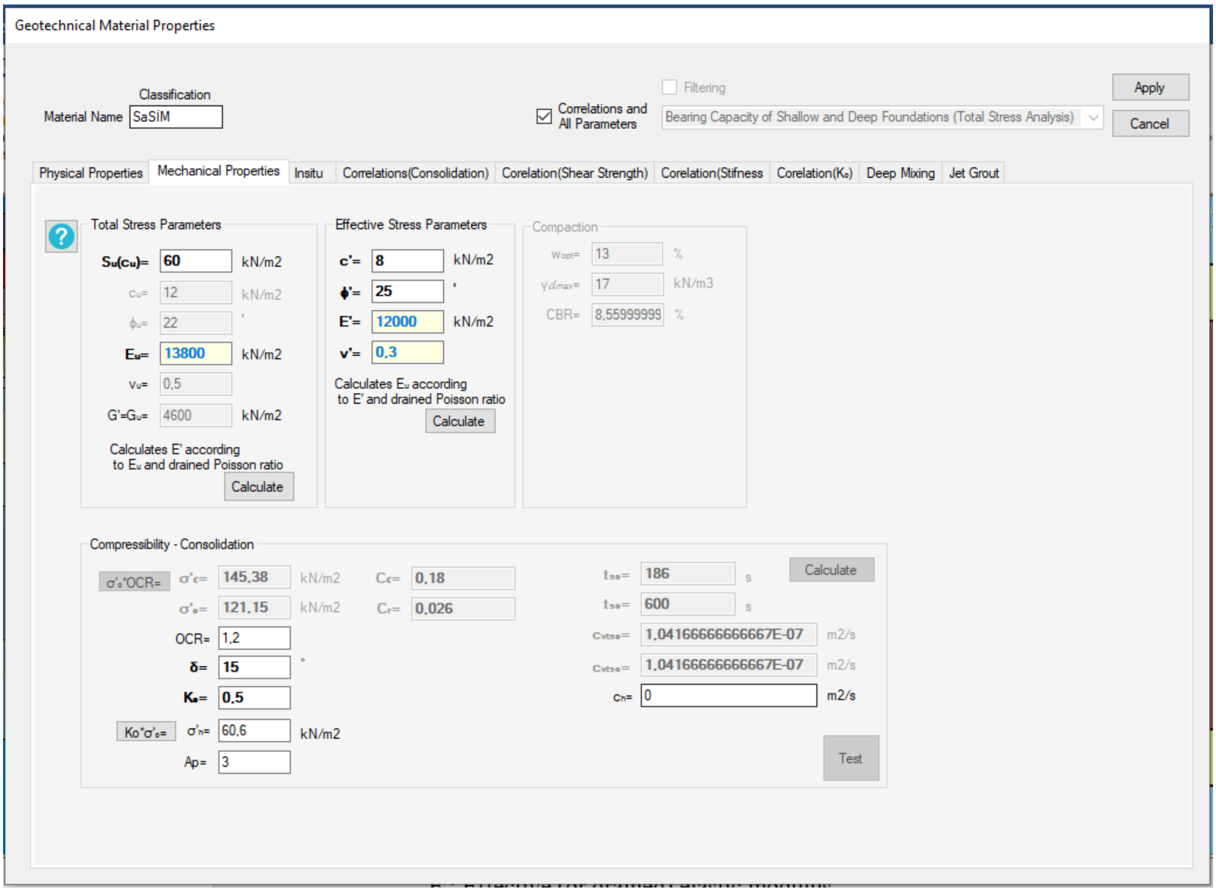


Figure 8. Defining Material Properties (Mechanical Properties Tab)

- Total Stress:

  $S_u(c_u)$: Undrained shear strength

  c_u: Cohesion under CU conditions (total stress)

  ϕ_u: Angle of internal friction under CU conditions (total stress)

  E_u: Undrained elastic modulus

  ν_u: Undrained Poisson's ratio

- Effective Stress:

  $G_u=G'$= Shear modulus

  c': Effective (or drained) cohesion

  ϕ': Effective (or drained) angle of internal friction

  E': Effective (or drained) elastic modulus

  ν': Effective (or drained) Poisson's ratio

- Consolidation Properties:

  σ'_c: Preconsolidation pressure

  σ'_0: Vertical effective stress

  OCR: Overconsolidation ratio

  δ: Wall-soil friction angle

  K_0: Coefficient of earth pressure at rest

  σ'_h: Horizontal effective stress

  A_p: Cohesion influence coefficient

  C_c: Compression index

  C_r: Recompression index

a_v: Coefficient of compressibility

m_v: Coefficient of volume compressibility

t_{50}: Time for 50% consolidation

t_{90}: Time for 90% consolidation

c_{vt50}: Coefficient of consolidation calculated with t_{50}

c_{vt90}: Coefficient of consolidation calculated with t_{90}

c_h: Horizontal coefficient of consolidation

- Compaction Properties:

  w_{opt}: Optimum moisture content

  γ_{dmax}: Maximum dry unit weight in compaction

  CBR: California Bearing Ratio

2.3. In Situ (Field Properties)

The in situ test properties of the soil at specific depths are entered in the **In Situ** tab of the program. For the Standard Penetration Test (SPT), the SPTN value and correction factors (**E_m, C_b, C_s, C_r**) are entered.

By clicking the **Calculate** button, the values of **C_N, N_1, N_{60}, and $N_{1,60}$** are automatically calculated (Figure 9). The values entered here can be used in correlations such as estimating the deformation modulus of coarse-grained soils.

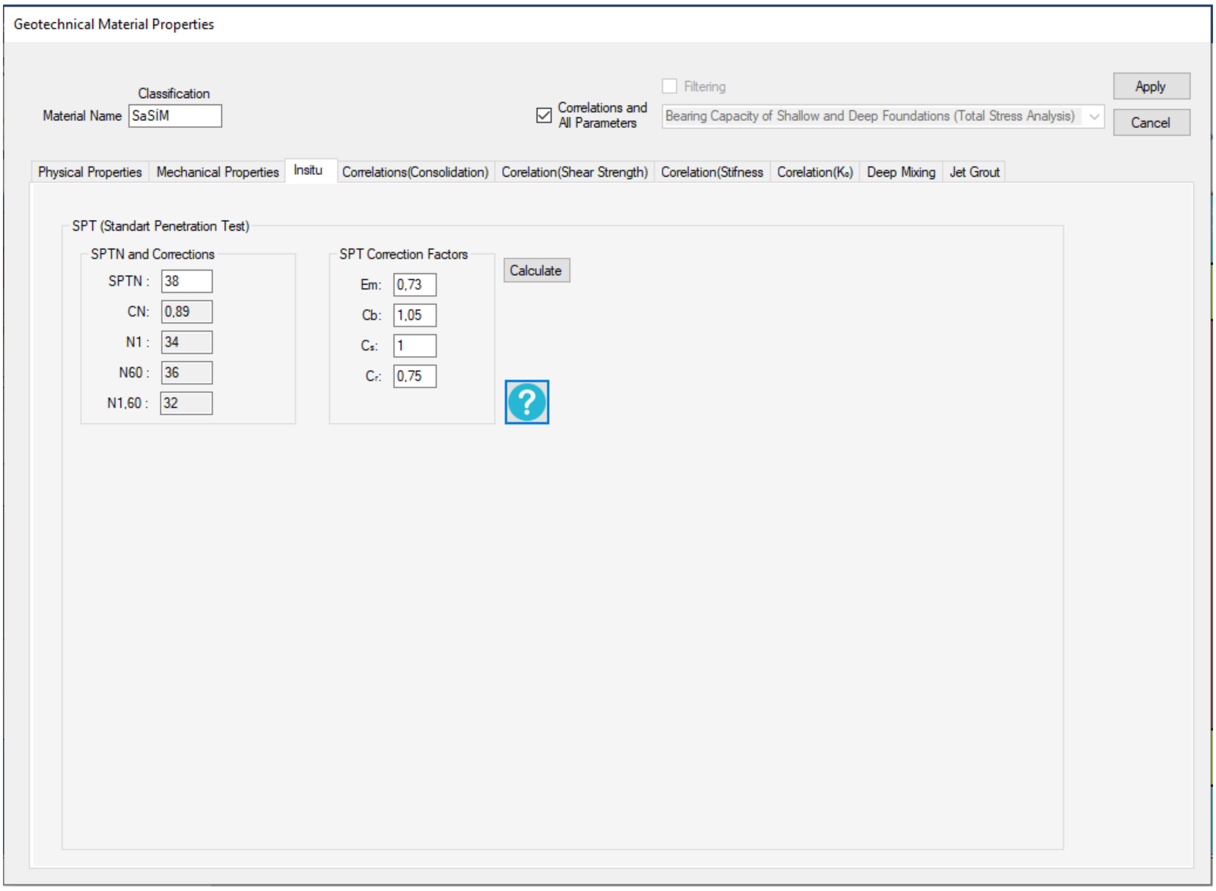


Figure 9. In Situ Tab

SPT Parameters:

SPTN: Number of blows for the second and third 15 cm of penetration (i.e., for the 30 cm after the initial 15 cm)

C_N: Overburden correction factor

N_1: C_N x SPTN

N_{60}: Blow count corrected to 60% standard energy

E_m: Hammer efficiency

C_b: Borehole diameter correction factor

C_s: Sampler correction factor

C_r: Rod length correction factor

2.4. Correlations

SETAF2018 allows users to calculate physical and mechanical properties using various literature-based correlations, particularly when sufficient experimental data is not available or when approximate values of soil parameters are needed. These properties can be calculated using the **Calculate** button.

The user can then use the **Apply** button to transfer the calculated values directly into the corresponding input fields in the **Physical** and **Mechanical Properties** tabs, thereby completing the data entry process.

In the **Correlations (Consolidation)** tab, one or more correlation-based calculations can be performed for **C_c**, **C_r**, **OCR**, and **σ'_c** values (Figure 10).

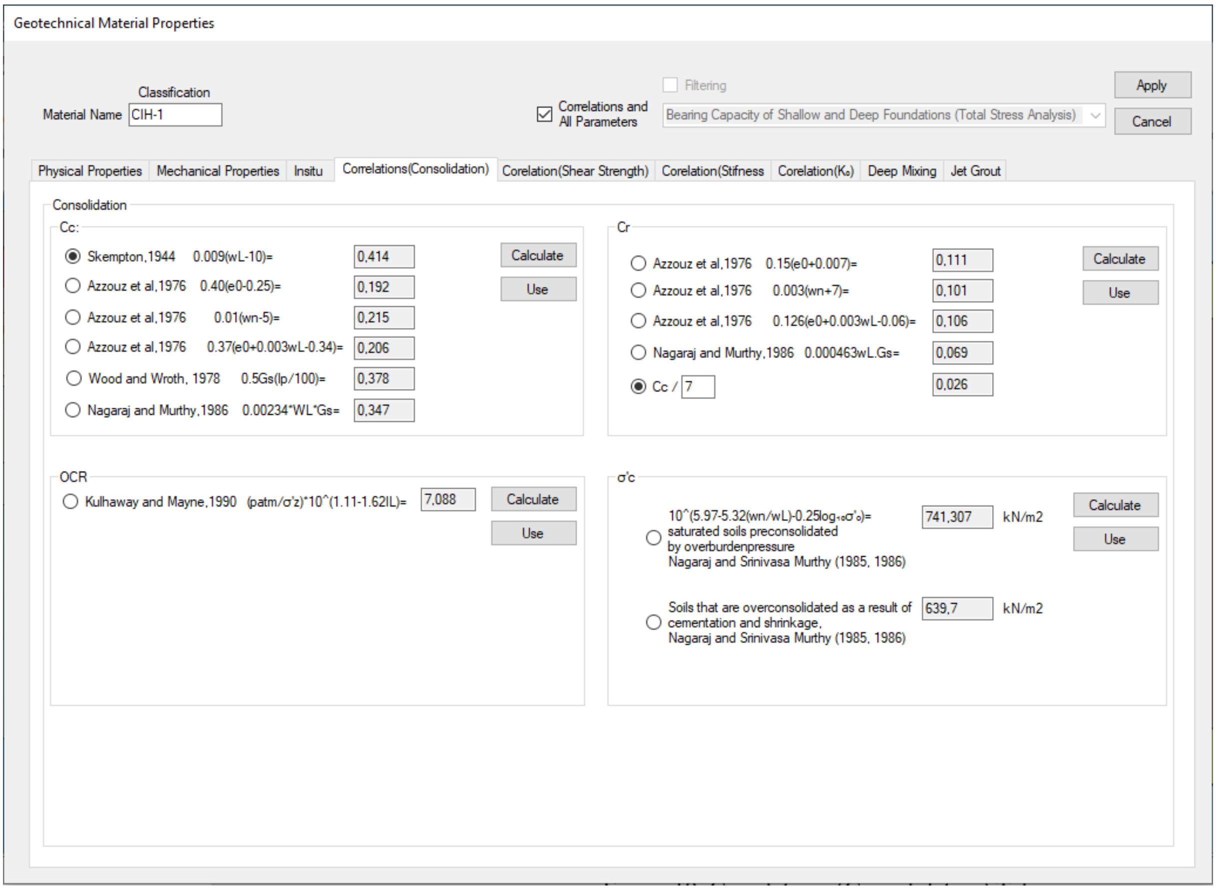


Figure 10. Correlations (Consolidation) Tab

In the **Shear Strength** tab, various correlations are available for estimating **undrained shear strength (S_u)** and **effective shear strength parameters (c', ϕ')** (Figure 11).

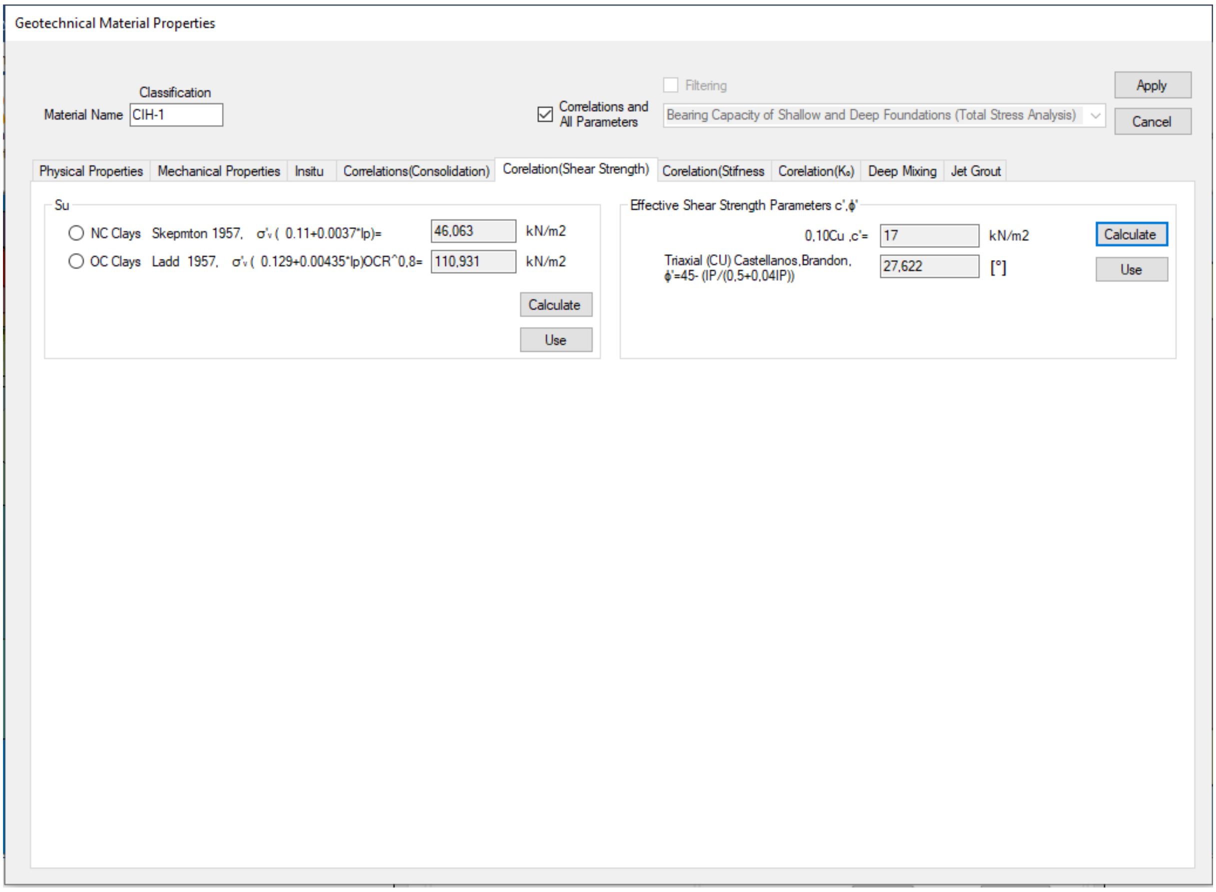


Figure 11. Correlations (Shear Strength) Tab

In the **Correlations (Stiffness)** tab, multiple correlation options are provided to estimate the **undrained elastic modulus (E_u)**, **effective elastic modulus (E')**, and **shear modulus (G)** (Figure 12).

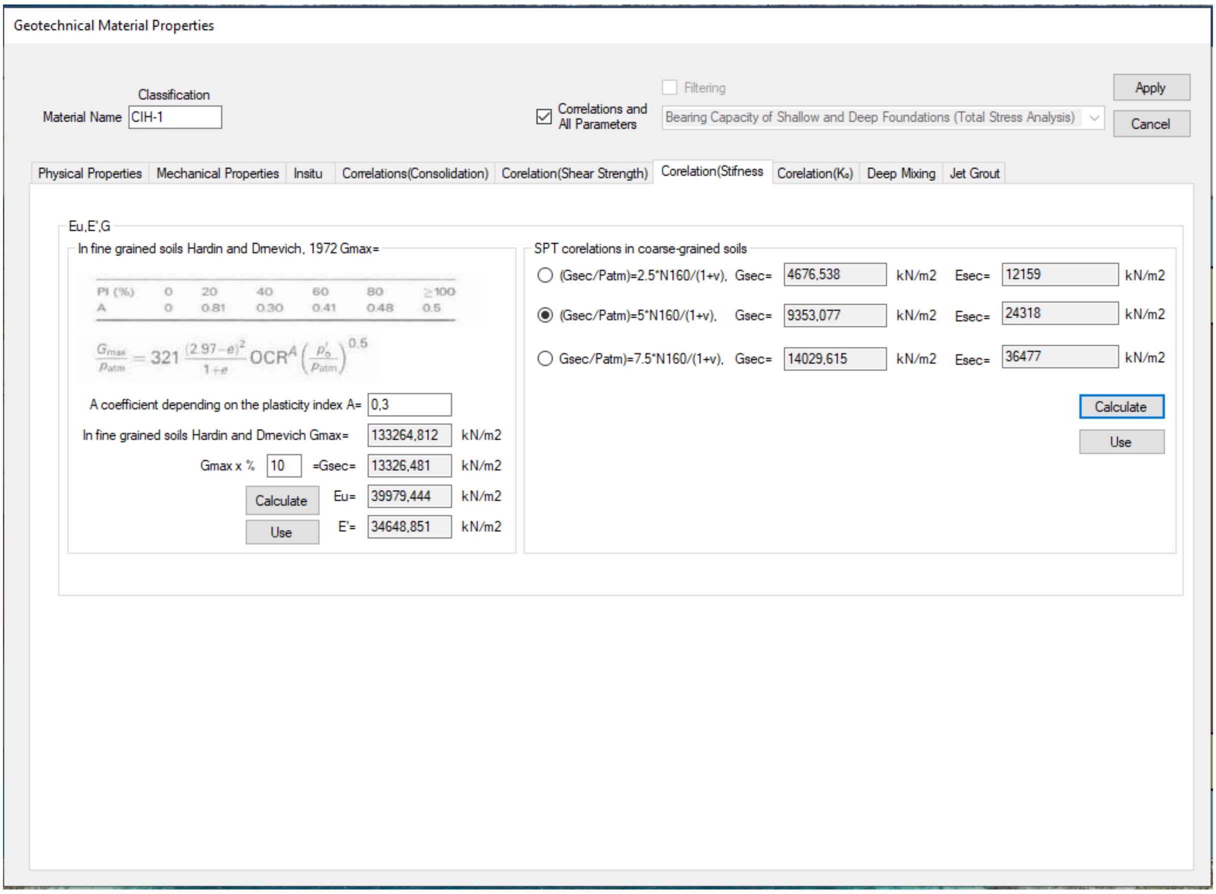


Figure 12. Correlations (Stiffness) Tab

In the **Correlations (K_0)** tab, the coefficient of earth pressure at rest (**K_0**) can be estimated using various correlations available in the literature.

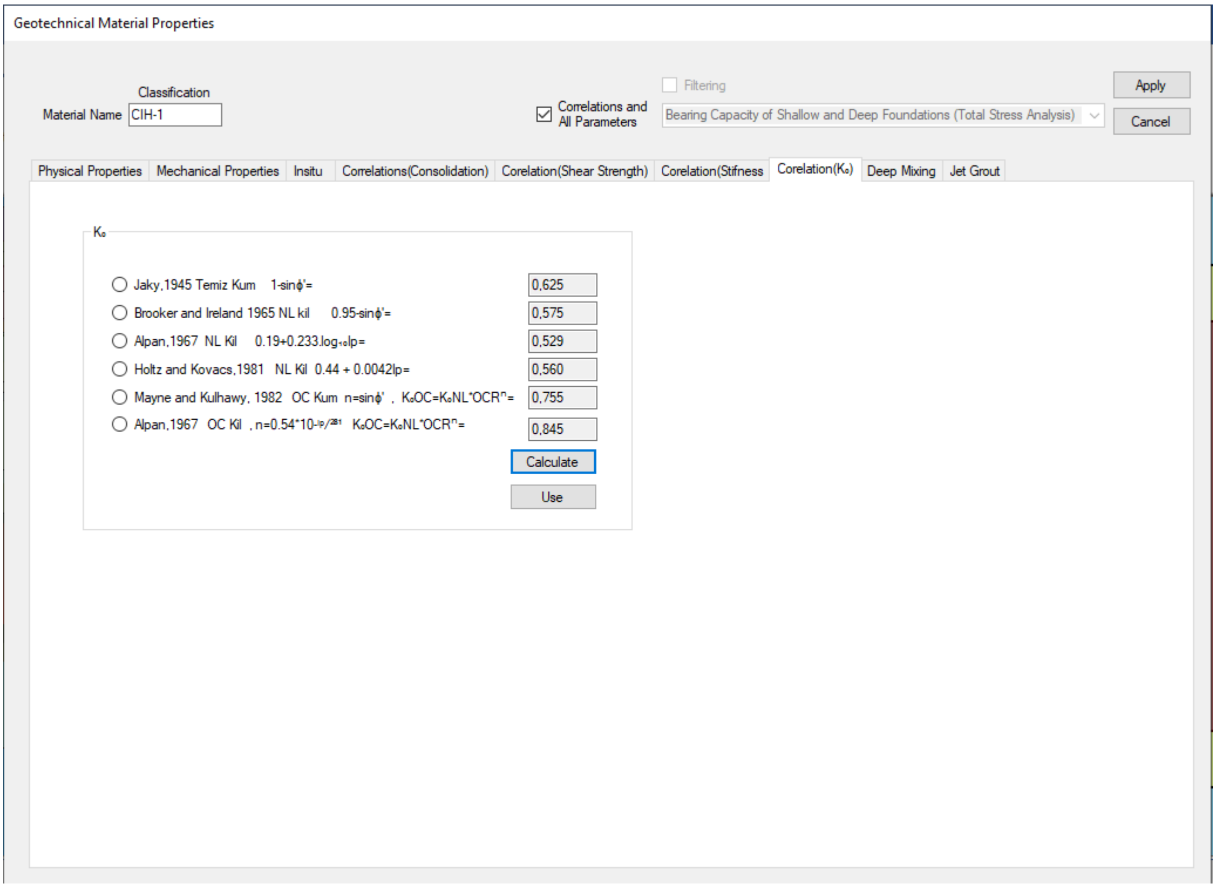


Figure 13. Correlations (K_0) Tab

2.5. Deep Mixing

In cases where the soil is improved using the **deep mixing method**, the properties of the resulting **mixture (column)** are defined in the **Deep Mixing** tab.

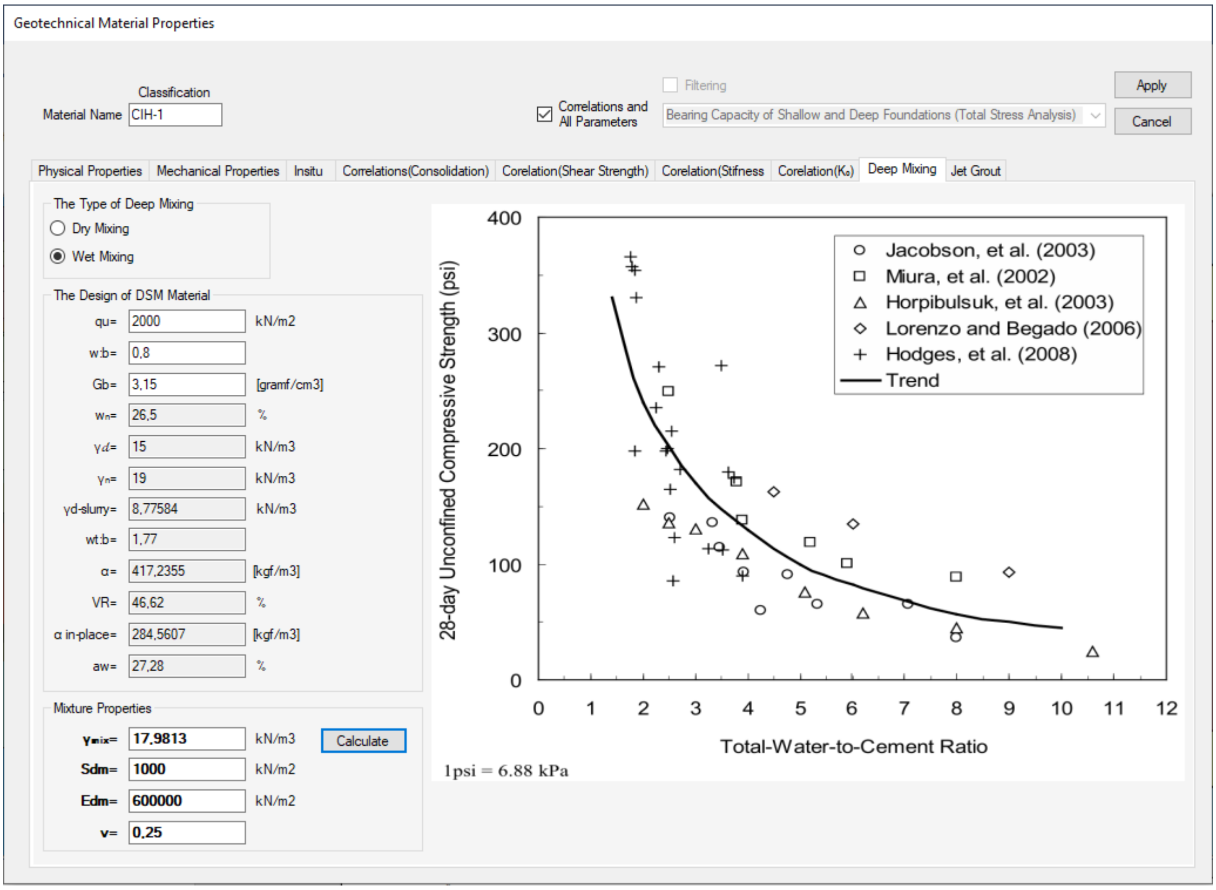


Figure 14. Deep Mixing Tab

Mixture Properties:

γ_{mix}: Natural unit weight of the mixture

S_{dm}: Shear strength of the mixture (for the case $\phi = 0$)

E_{dm}: Elastic modulus of the mixture

ν: Poisson’s ratio of the mixture

The values of γ_{mix}, S_{dm}, and E_{dm} can be automatically calculated using the **Calculate** button. E_{dm} and S_{dm} are calculated based on the **unconfined compressive strength (q_u)**. ρ_{mix} and the **amount of binder in the mixture** are determined to ensure a proper material design.

The required amount of slurry in the mixture is calculated based on the **target unconfined compressive strength (q_u)**, the **water-to-binder ratio (w:b)**, and the **specific gravity of the binder (G_b)**.

Additional Parameters:

q_u: Unconfined compressive strength

w:b Water-to-binder ratio of the slurry

G_b: Specific gravity of cement

w_n: Natural water content of the existing soil

γ_d: Dry unit weight of the existing soil

γ_n: Natural unit weight of the existing soil

$\gamma_{d\text{-}slurry}$: Dry unit weight of the slurry

w_t:b: Total water-to-binder ratio

α: Binder factor

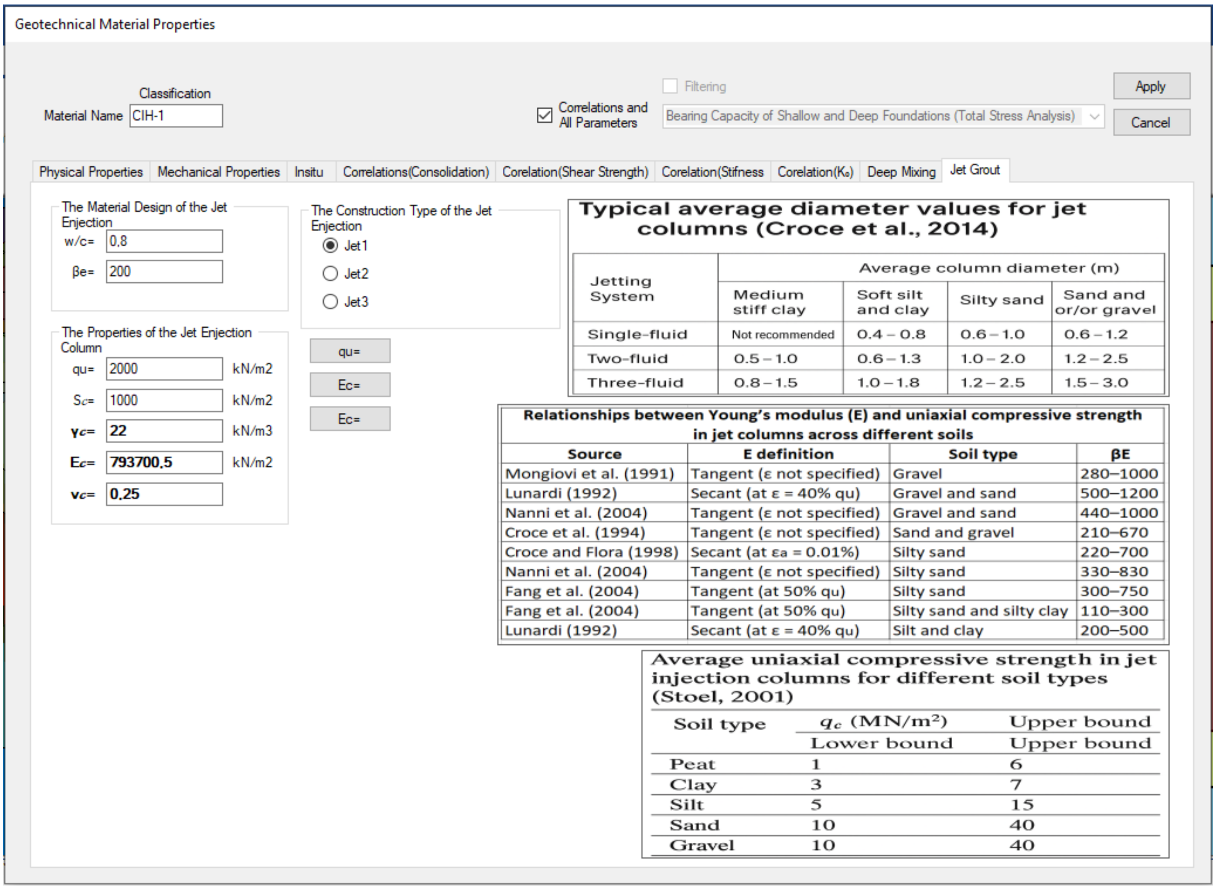
VR: Volume ratio

$\alpha_{in\text{-}place}$: Binder factor in-place

a_w: Binder content

2.6. Jet Grouting

The engineering properties of the column formed by mixing the defined soil using the **jet grouting method** are calculated.



Şekil 15. Jet Grouting Tab

The properties of the jet grout column can be defined based on either the **water-cement ratio** or the **β_e coefficient**. A reference table is provided for determining the β_e coefficient.

Parameters:

q_u: Unconfined compressive strength

w/c: Water-cement ratio of the injection mixture

S_c: Shear strength of the jet grout column (assumed ϕ=0, Tresca model)

γ_c: Unit weight of the jet grout column

E_c: Elastic modulus of the jet grout column

ν_c: Poisson's ratio of the jet grout column

2.7. Soil Properties Used in Analysis and Calculations

In **SETAF2018**, the physical and mechanical soil properties required for analyses and calculations, along with their intended uses, are listed below:

γ_{sat} (Natural Unit Weight)

Used in the calculation of effective stresses and for generating stress diagrams.

γ_n (Natural Unit Weight)

A key parameter in determining effective stresses. It is used to compute stress distributions within the soil profile and to generate stress diagrams.

e_0 (Initial Void Ratio)

Represents the initial (natural) void ratio of the soil. It is directly used in consolidation settlement calculations and serves as a primary input in consolidation analyses when entered in the relevant section.

S_u (c_u) - Undrained Shear Strength

Applied in total stress ($\phi = 0$) bearing capacity analyses for silty or clayey soils. Required for determining shallow foundation bearing capacity under total stress and shaft resistance of deep foundations using the α-method.

c_u - Cohesion (Total Stress Analysis)

The cohesion value used in total stress (c-ϕ) analyses for calculating the bearing capacity of shallow foundations. It is entered in the same interface and incorporated into bearing capacity formulas automatically.

ϕ_u - Angle of Shear Resistance (Total Stress Analysis)

Used to calculate shallow foundation bearing capacity in clayey or silty soils using total stress analysis (c-φ assumption). Defined under "Mechanical Properties" > "Total Stress Parameters."

E_u - Undrained Elastic Modulus

Represents the elastic behavior of the soil under undrained conditions. Primarily used to compute elastic deformations during short-term loading. Required input for accurate settlement prediction and stress distribution.

c′ - Cohesion (Effective Stress Analysis)

Represents the cohesion of the soil under drained (long-term) conditions. Entered under "Mechanical Properties" > "Effective Stress Parameters." Used in:

- Shallow foundation bearing capacity calculations
- Shaft resistance estimations in deep foundations (β-method)
- Accurate estimation of earth pressures in excavation support systems

ϕ′ - Angle of Internal Friction (Effective Stress)

The internal friction angle of soil under drained conditions. The software uses this parameter to:

- Calculate shallow foundation bearing capacity (effective stress analysis)
- Estimate shaft resistance for deep foundations using the β-method
- Active and passive earth pressures in wall/excavation analyses are determined using this parameter, which is defined in the **"Effective Stress Parameters"** section of the **"Mechanical Properties"** tab.

E' - Elastic Modulus (Drained Conditions)

Used to calculate elastic deformations in drained soil layers. Critical for determining the amount of deformation under loading.

v' - Poisson's Ratio (Drained Conditions)

Used to compute stress increases and elastic deformations in drained layers. Expresses the ratio of strain in one direction to strain in perpendicular directions.

σ'_c (Preconsolidation Pressure)

Represents the maximum effective stress the soil has experienced. Used in consolidation settlement analyses.

K_0 (Coefficient of Earth Pressure at Rest)

Used in calculating horizontal effective and total stresses, generating stress diagrams, and computing earth pressures at rest in wall analyses. Represents the ratio of horizontal to vertical stresses.

C_c (Compression Index)

Used in consolidation settlement calculations. Indicates the compressibility of the soil.

C_r (Recompression Index)

Used in consolidation settlement analyses. Represents the behavior of the soil under reloading conditions.

t_{50} (Time for 50% Consolidation)

Used in the calculation of the coefficient of consolidation. Represents the time required for 50% consolidation.

t_{90} (Time for 90% Consolidation)

Used in the calculation of the coefficient of consolidation. Represents the time required for 90% consolidation.

$C_{v,t50}$ - Coefficient of Consolidation (from t_{50})

Used for time-based consolidation settlement analysis and for generating settlement vs. time graphs. Represents the consolidation coefficient derived from the t_{50} value.

$C_{v,t90}$ - Coefficient of Consolidation (from t_{90})

Used for time-based settlement calculations and time-settlement graphing. Derived from the t_{90} value.

δ (Wall-Soil Friction Angle)

Used in wall analyses. Represents the friction angle between the wall and the adjacent soil.

A_p (Cohesion Influence Coefficient)

Used in wall analyses during the calculation of the horizontal subgrade reaction modulus. Reflects the influence of soil cohesion on the wall.

3. BOREHOLES

The **SETAF2018** software enables a detailed definition of borehole data obtained within the scope of geotechnical projects.

- **Borehole Management:** Users can define any number of borehole locations and related data for the project site. There is no limitation on the number of boreholes that can be added.
- **Layer Definition:** Within each borehole profile, an unlimited number of soil and rock layers can be defined. This allows for accurate modeling of even complex subsurface conditions.
- **In-Situ Test Profiles:** The software supports the definition of depth-based profiles for Standard Penetration Test (SPT) results (N-values) and Menard-type Pressuremeter Test (MPM) data for each borehole. These datasets serve as essential inputs for geotechnical analysis.

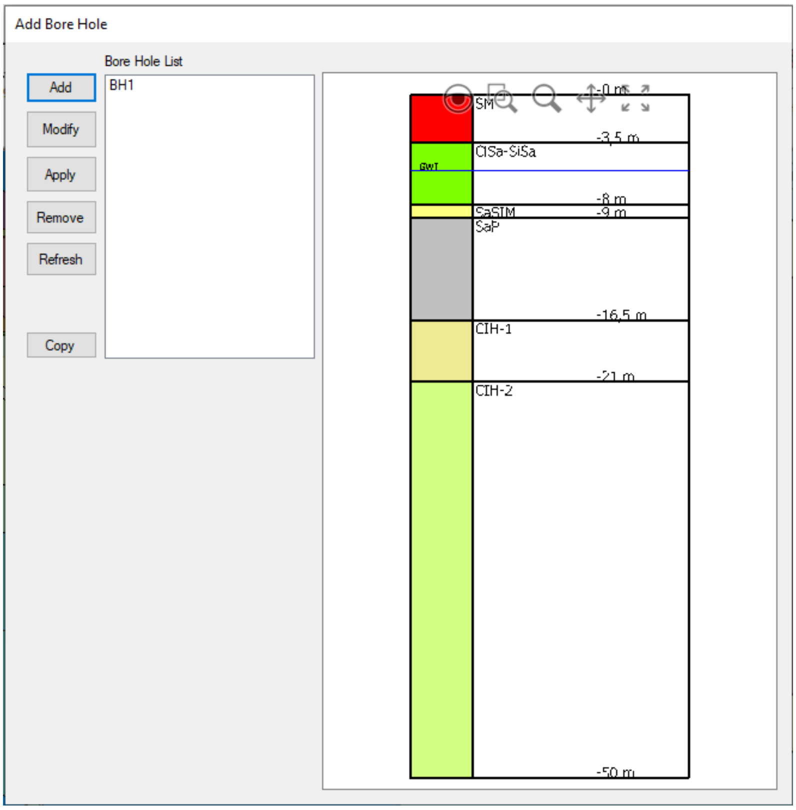


Figure 16. Adding a Borehole

Defining Basic Borehole Information

When adding a new borehole or editing an existing one, basic identifying information for each borehole must be entered. These include:

- **Borehole ID/Name:** An alphanumeric identifier used to uniquely distinguish each borehole within the project.
- **Coordinates:** Includes the planimetric position of the borehole on-site (usually X and Y coordinates) and the elevation of the borehole collar (Z coordinate or surface elevation).
- **Groundwater Level (GWL):** It refers to the depth or elevation of the groundwater table, determined during or after drilling operations. This information is essential for assessing hydrogeological conditions and for calculating effective stresses in geotechnical analyses.

A visual example showing how to enter this basic information in the software interface is provided in Figure 17.

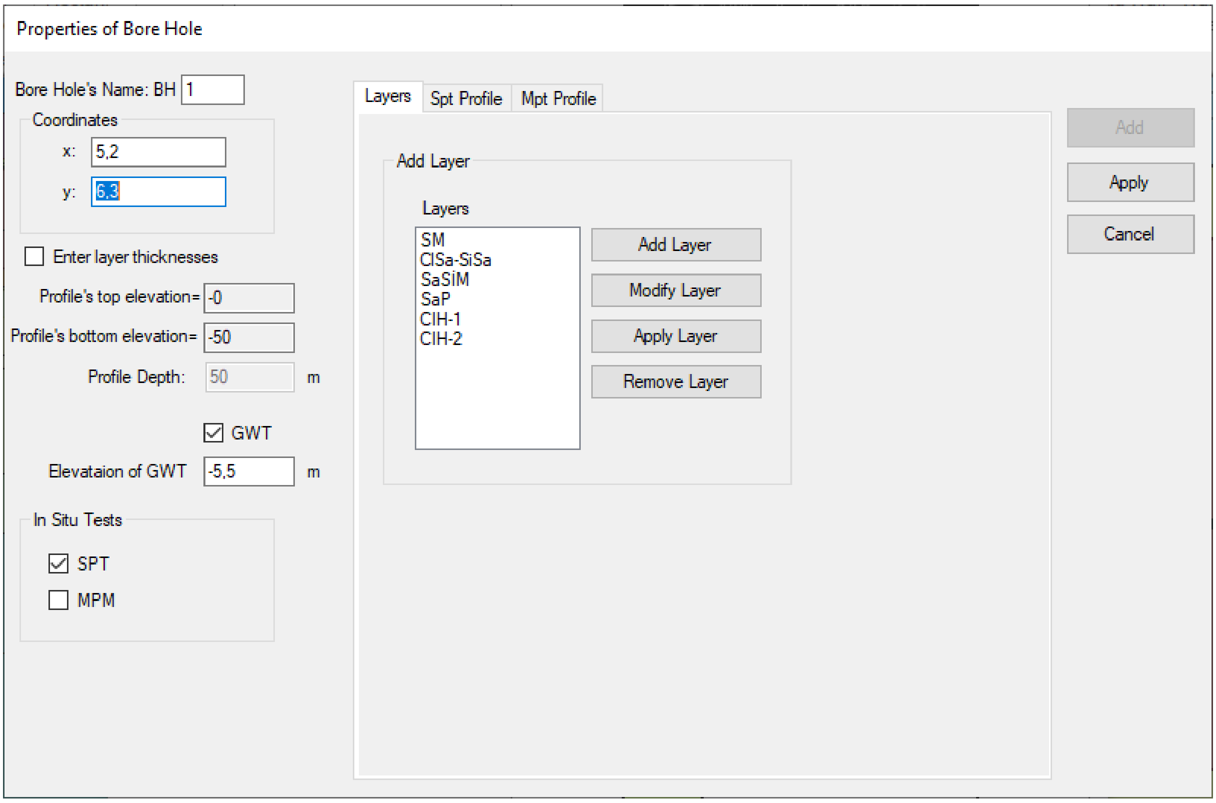


Figure 17. Borehole Properties

3.1. Definition of Soil Layers

In the program, various properties of each layer can be entered, including name, description, top and bottom elevations, artesian pressure, capillarity (if silt is present), drainage conditions, number of sublayers, and consolidation properties. For consolidation characteristics, the distribution of excess pore water pressure with depth can be selected (Figure 18). One of the general application options—(a), (b), or (c) is used for this purpose.

The following properties are defined for each soil layer in the software:

- Layer name
- Description
- Top elevation

- Bottom elevation
- Artesian pressure
- Capillarity (if silt is present)
- Drainage conditions
- Number of sublayers
- Consolidation properties

In the consolidation section, depth-dependent distributions of excess pore water pressure are selected (Figure 18). One of the general application options (a), (b), or (c) can be used. Detailed explanations of these options are provided below.

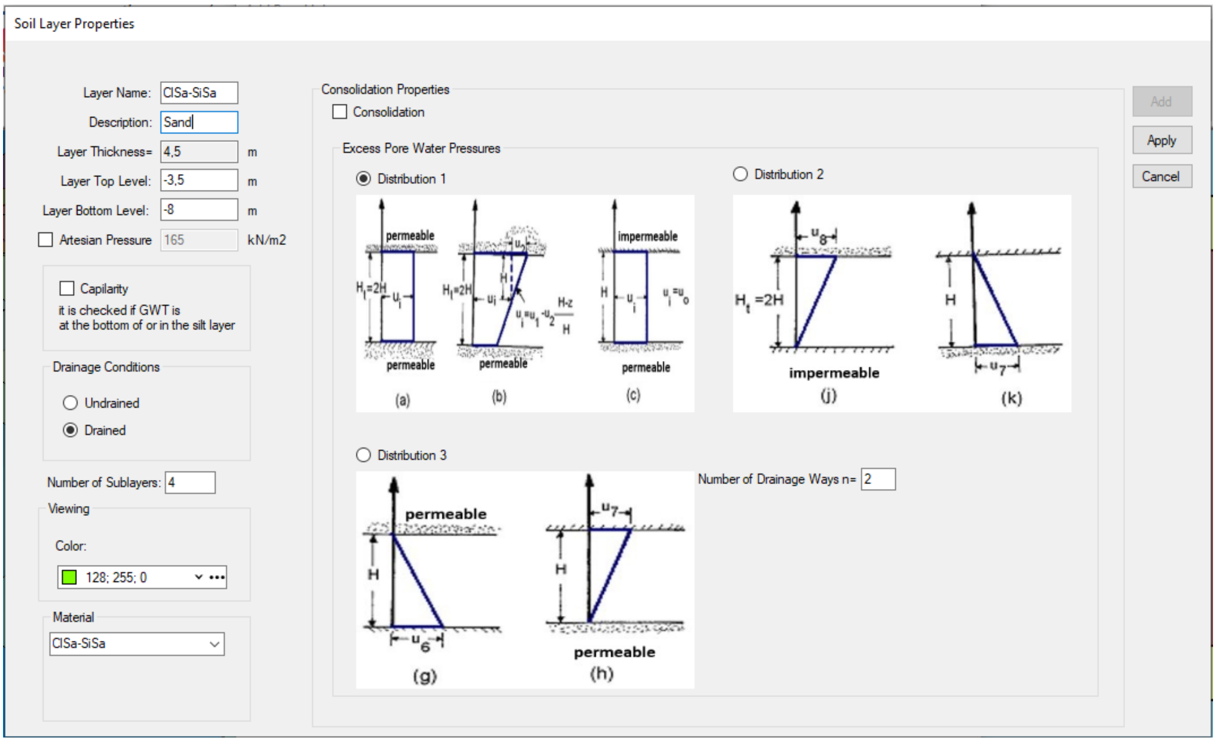


Figure 18. Layer Properties

Layer elevations are entered based on the global coordinate system. Elevation values are defined such that the upward direction is positive and the downward direction is negative. Continuity between layer elevations must be maintained. For example, if the first layer is defined between 0 and -5 meters, the second layer must begin at -5 meters.

For silty layers located above the groundwater level (GWL), the capillarity feature can be defined. The number of sublayers indicates how many divisions will be considered in the settlement analysis.

3.2. Defining the SPT Profile

The results of the Standard Penetration Test (SPT) conducted in the relevant borehole are entered into the program as a **depth–SPTN profile**. Additionally, **SPT correction factors** are also defined separately (Figure 19).

SPT test data from the selected borehole are input into the system in the form of a depth-based SPTN profile. The corresponding SPT correction factors are also specified. Figure 19 illustrates how to enter both the SPT profile and the correction factors into the software.

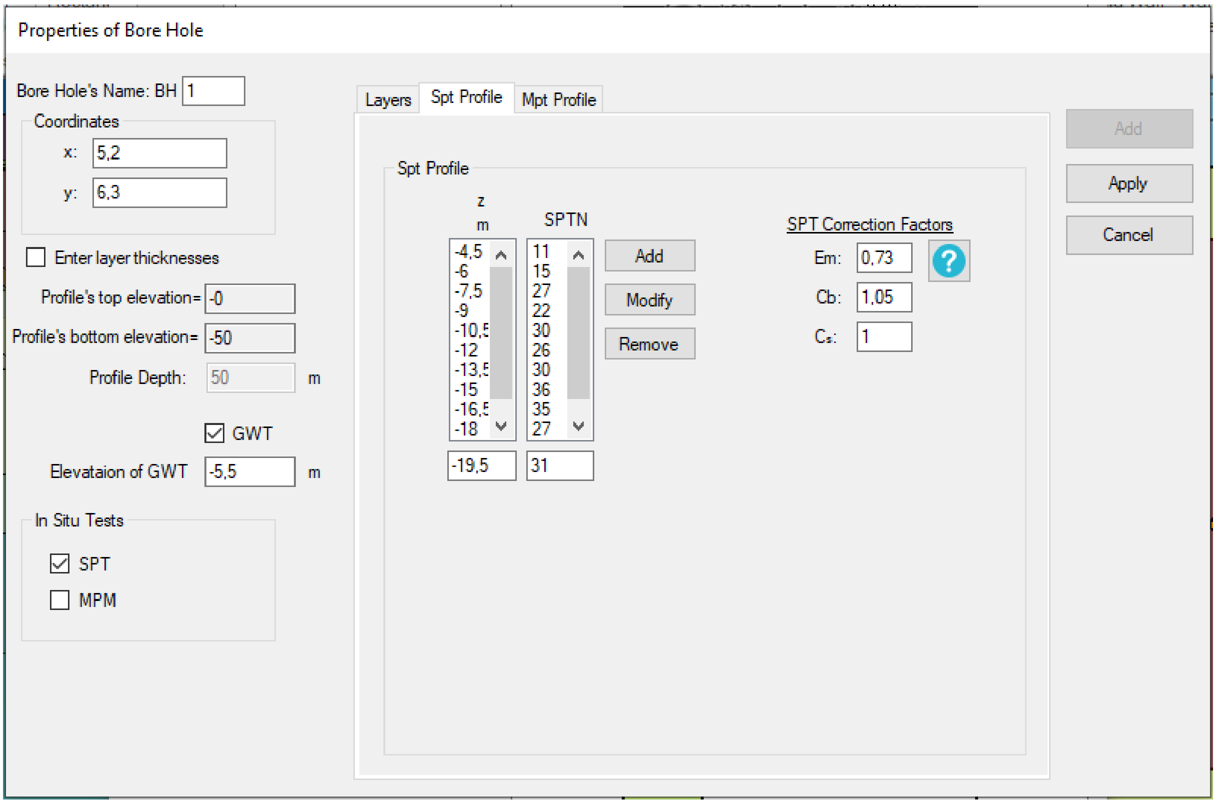


Figure 19. Borehole Properties – SPT Tab

3.3. Defining the MPM Profile

The results of the **Menard Pressuremeter Test (MPM)** conducted in the relevant borehole are entered into the program as a **Depth–Menard Modulus (EM)** and **Limit Pressure (PL)** profile. Figure 20 illustrates how to input the MPM profile into the software.

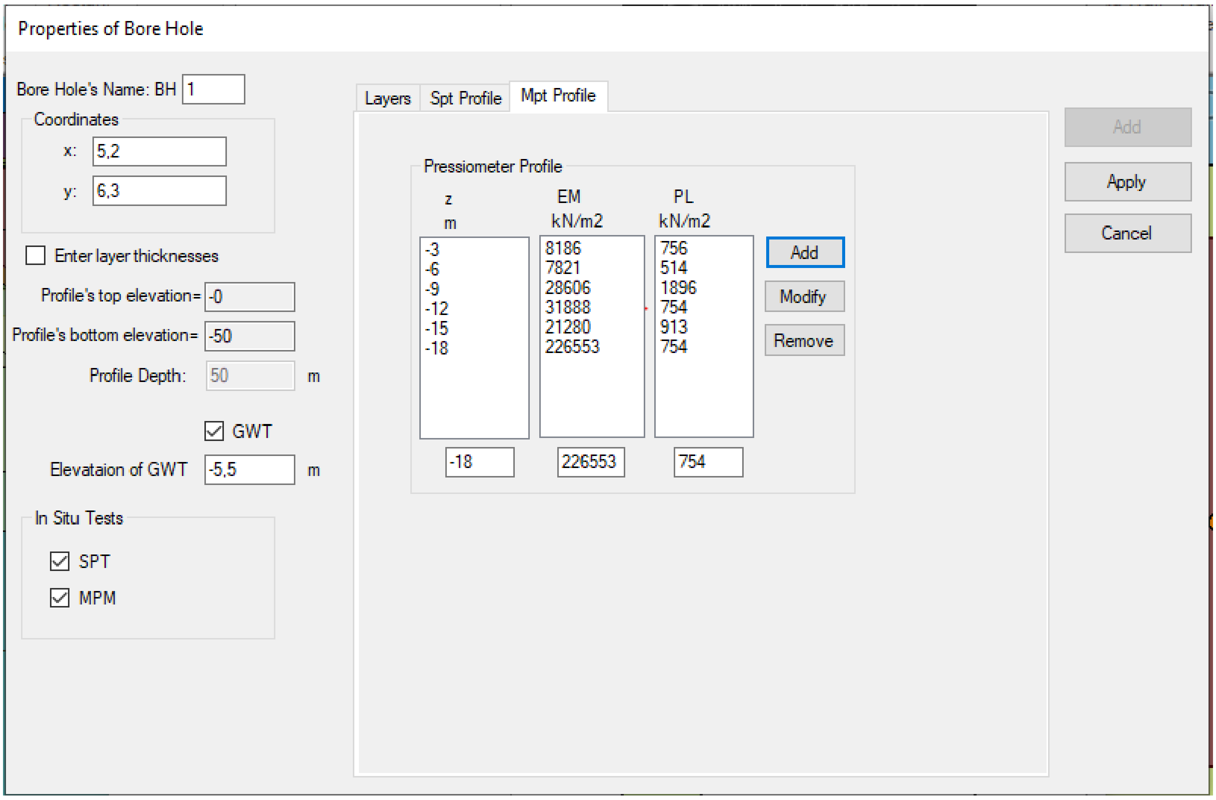


Figure 20. Borehole Properties – MPT Tab

E_M: Menard modulus

P_L: Limit pressure

4. FOUNDATIONS

In **SETAF2018**, users can create models that include any number of foundations at specified coordinates. Foundation geometries can be defined as **rectangles, polygons, or circles**. Foundations are positioned by entering their coordinates and bottom elevation data.

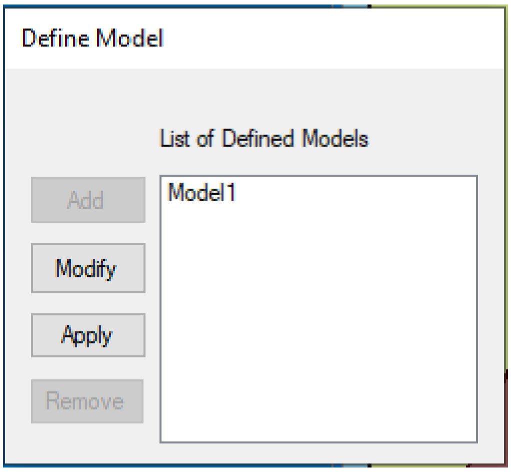


Figure 21. Adding a Foundation Model

The model can include as many **settlement points** and **foundations** as needed, located at any coordinates. The interface for defining settlement points and foundations is shown in Figure 22.

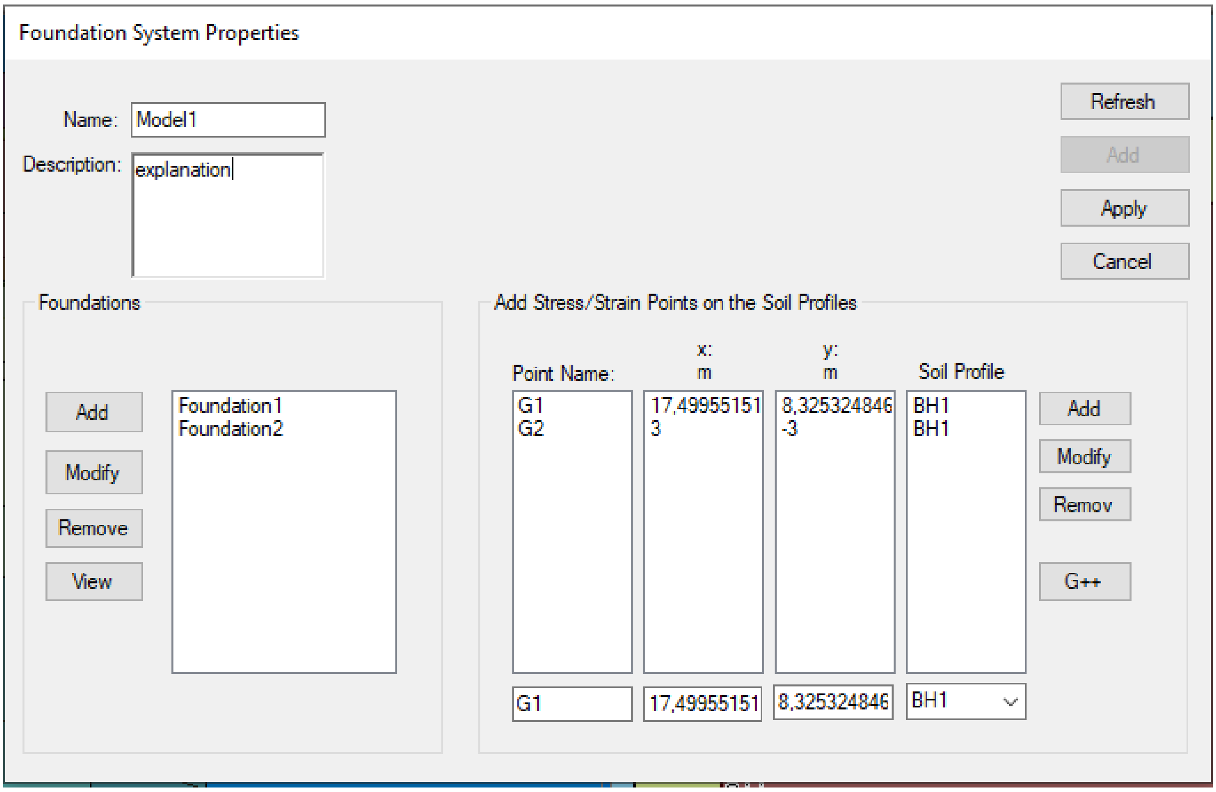


Figure 22. Foundation System Properties

By using the **G++** button, a settlement point is added at the **geometric center** of each defined foundation. This point serves as a **reference location** for the settlement analysis of the foundation.

4.1. Rectangular Foundation Geometry

In the program, the foundation is defined by entering the (x, y) coordinates of its four corner points and its bottom elevation (z). These values are defined relative to the 3D coordinate system (x, y, z) (Figure 23).

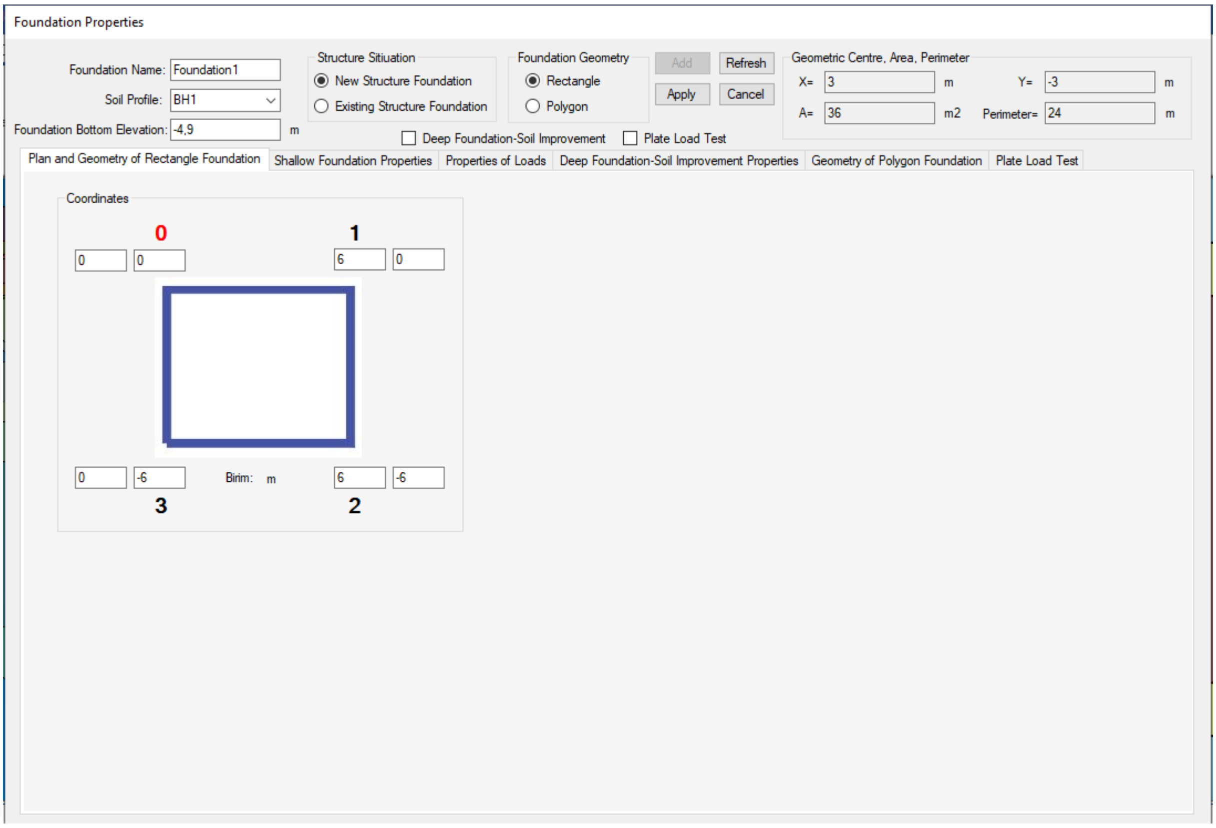
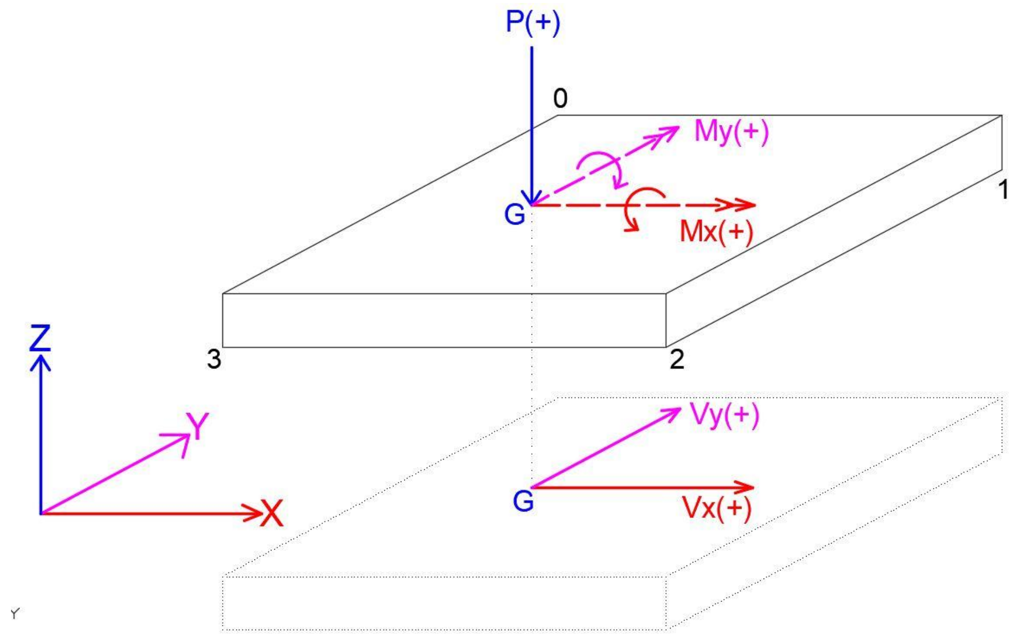


Figure 23. Plan and Geometry Tab

4.2. Loads

In the **Loads** tab, the user enters the axial force acting on the foundation (P), moments in the X and Y directions (M_x, M_y), horizontal forces in the X and Y directions (V_x, V_y), foundation base inclination, and ground surface slope. Using the **Calculate** button, assuming a rigid foundation, the base pressures at each corner and the net base pressure (q_{net}) are calculated based on the input values of P, M_x, and M_y. The value of $\mathbf{q_{net}}$ is calculated according to the **D_f depth** specified in the **Shallow Foundation Properties** tab.



P: Axial load

M_x: Moment about the global X-axis (moment vector in the X direction)

M_y: Moment about the global Y-axis (moment vector in the Y direction)

V_x: Horizontal force in the X direction

V_y: Horizontal force in the Y direction

Base Inclination: Angle between the foundation base and the horizontal (in degrees)

Ground Slope: The angle (in degrees) between the ground surface adjacent to the footing and the horizontal.

q_{avg}: Average base pressure

q_{net}: Net base pressure

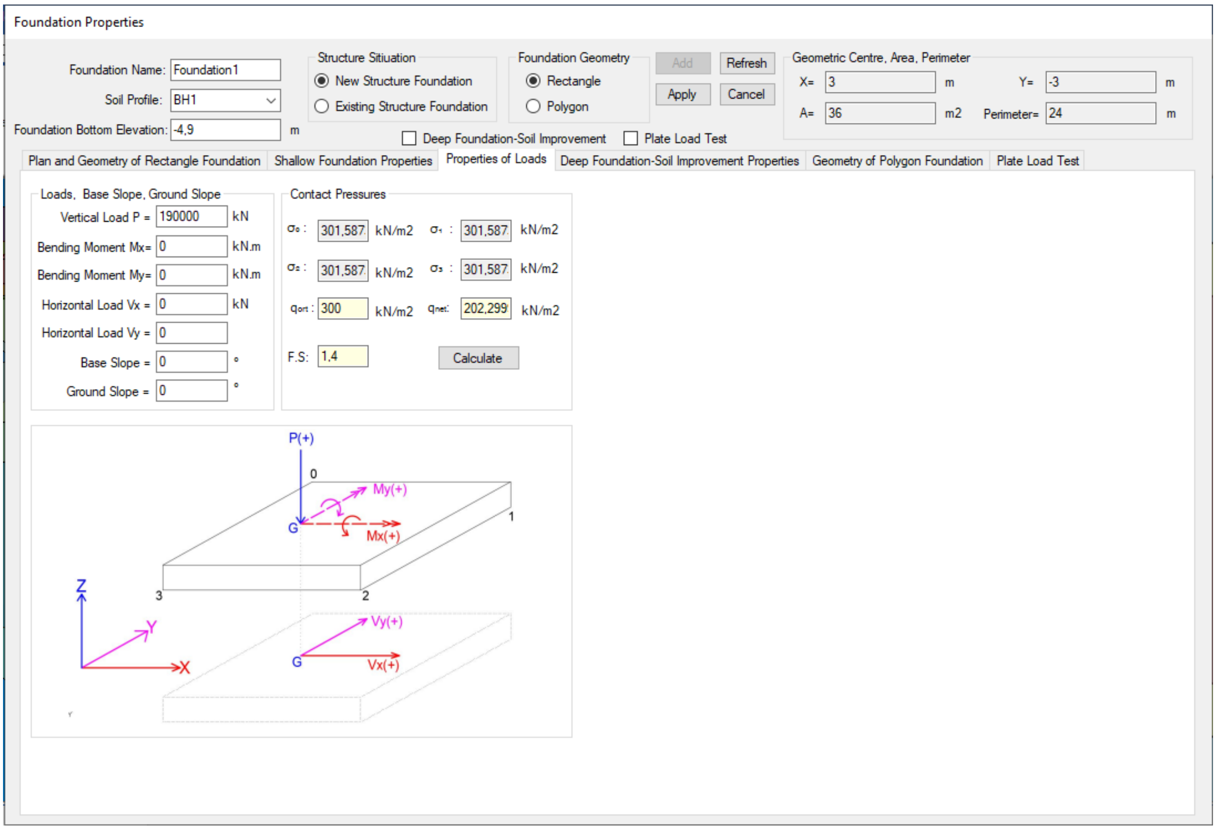


Figure 24. Loads Tab

γ_{Rv}: Safety factor for shallow foundation bearing capacity. This value corresponds to the strength reduction factor γ_{Rv} in the Turkish Seismic Code (TBDY).

When entering loads, the units of **P [F]** and **q_{avg} [F/L²]** must be compatible. The **q_{avg}** represents the average base pressure generated by the axial load P.

4.3. Shallow Foundation Properties

Values entered in the **Shallow Foundation Properties** tab are used in the calculation of shallow foundation bearing capacity and do **not** affect the geometric model of the foundation. In this tab, the user specifies the foundation shape (strip, rectangle, circle), the short side length or diameter (B), the long side length (L), and the embedment depth (D_f), **which are used in bearing capacity calculations**.

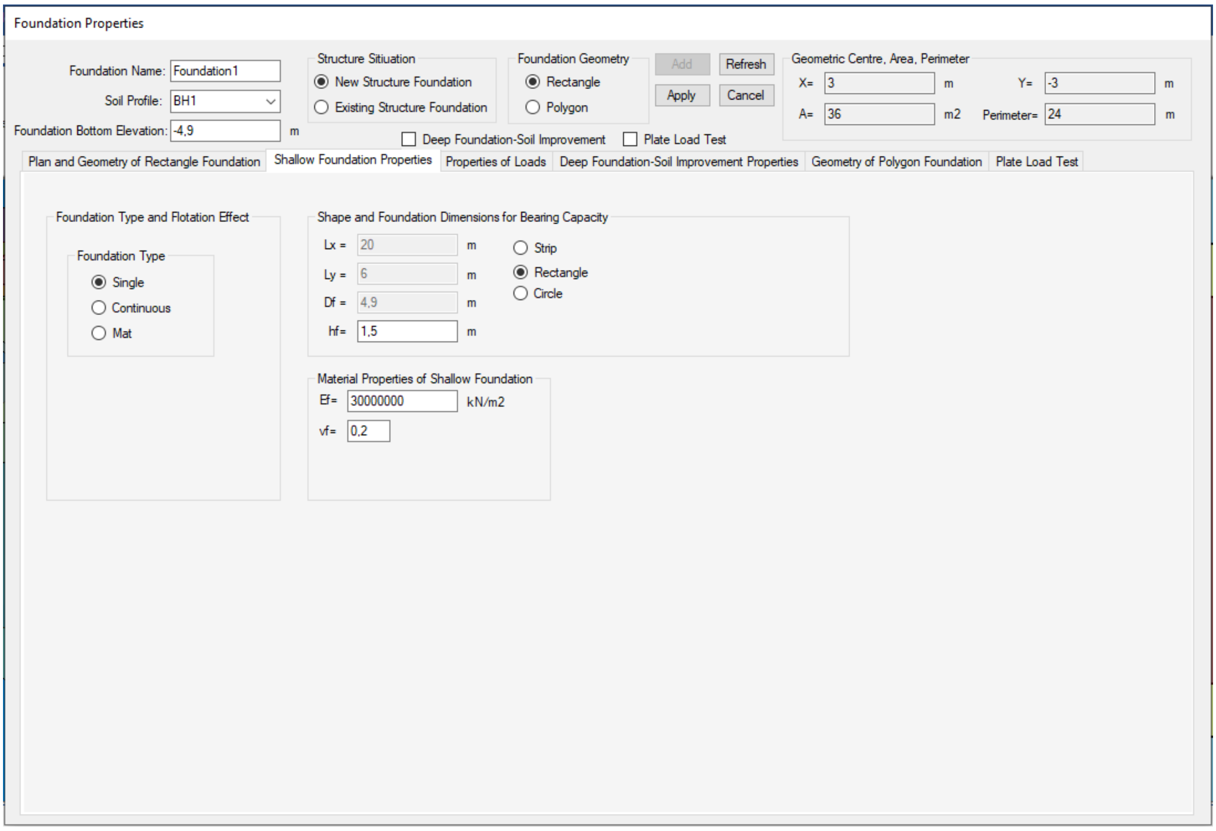


Figure 25. Shallow Foundation Properties Tab

4.4. Deep Foundation – Ground Improvement Properties

In the **Deep Foundation Properties** tab, the **number of rigid columns** and their **spacing** are defined specifically for **rectangular foundations**. In addition, for **all foundation types**—rectangular, polygonal, and circular—parameters such as **column type**, **diameter**, **length**, and **cross-sectional area** can be specified (Figure 26).

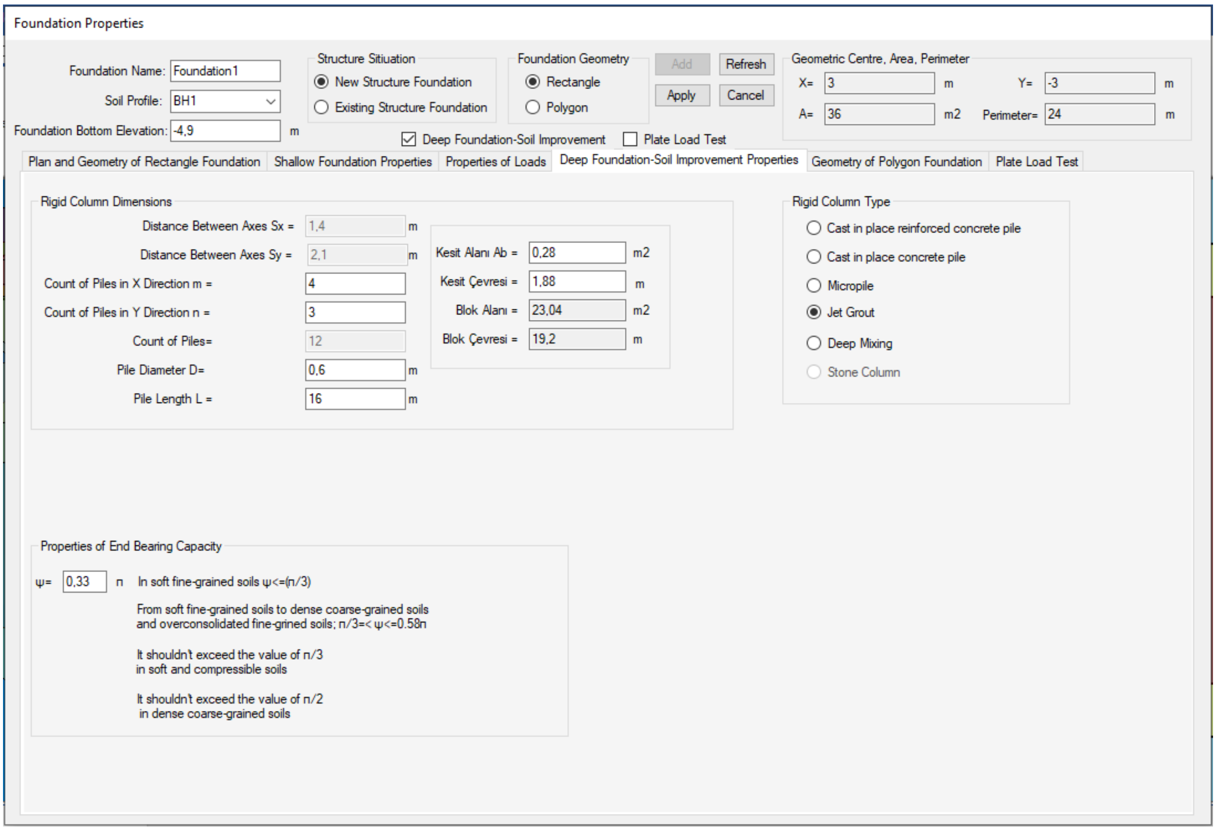


Figure 26. Deep Foundation Properties Tab

4.4.1. Adding Rigid Columns to a Rectangular Foundation

The spacing between column centers in the X and Y directions is defined as $\mathbf{S_x}$ and $\mathbf{S_y}$. These values must not be less than **2 × D**, where **D** is the column diameter. Based on the number of columns defined in the X direction (**m**) and Y direction (**n**), the spacing is automatically calculated so that the distance between the edge columns and the edge of the foundation equals **1 × D**.

4.5. Polygon-Shaped Foundation Geometry

When the **Polygon** geometry option is selected, the "Rectangular Foundation Plan and Geometry" tab is disabled, and the **Polygon Foundation Geometry** tab becomes active. In this tab, closed polygon shapes can be created by defining corner points. Circular foundations can also be designed by specifying a radius. Within the defined polygonal or circular foundations, **voids** (also polygonal or circular) can be created. There is no limit to the number of voids that can be defined.

4.5.1. Adding Rigid Columns to a Polygonal or Circular Foundation

Rigid columns can be defined by entering their coordinates within polygonal or circular foundations. Alternatively, an **automatic column group** can be generated inside a closed area using user-defined **S$_x$** and **S$_y$** spacing values. The resulting column centers will approximately follow the entered spacing values. Column coordinates can be modified afterward.

It is **not** possible to place columns outside the closed foundation area. Unlike rectangular foundations, there is no restriction on the spacing between column centers.

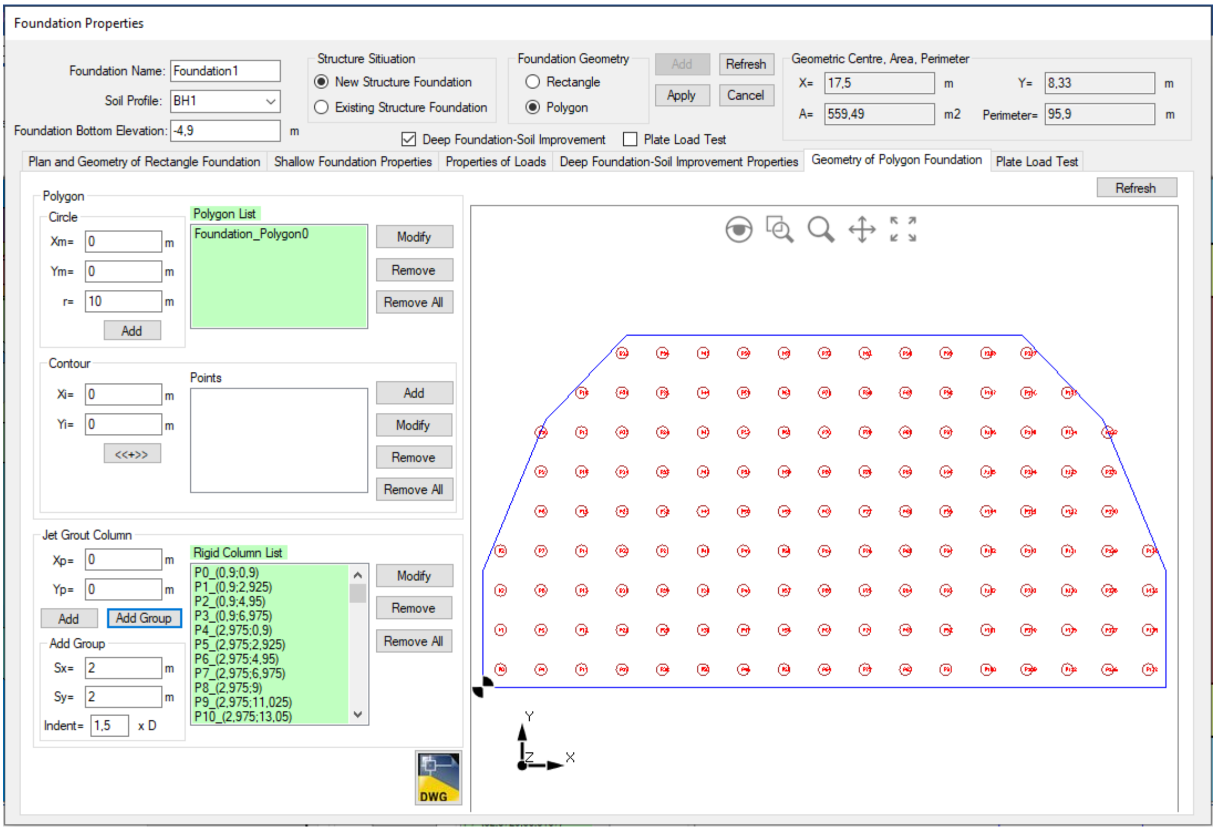


Figure 27. Polygon Foundation Definition and Column Group

The coordinates of the columns in the group can be edited, or selected columns can be deleted. For circular foundations, column groups are arranged in **ring patterns** based on specified spacing values. In this case, **S$_x$** refers to the distance between rings, and **S$_y$** refers to the spacing between column centers along the ring.

5. WALLS

In the program, walls are modeled in 3D using macros referred to as **Wall Groups**. Each macro contains a wall group, and each wall within the group has an associated **section component**. The section component includes subcomponents such as the **soil profile**, **wall geometry**, **terrain**, **anchors**, and **section analysis**.

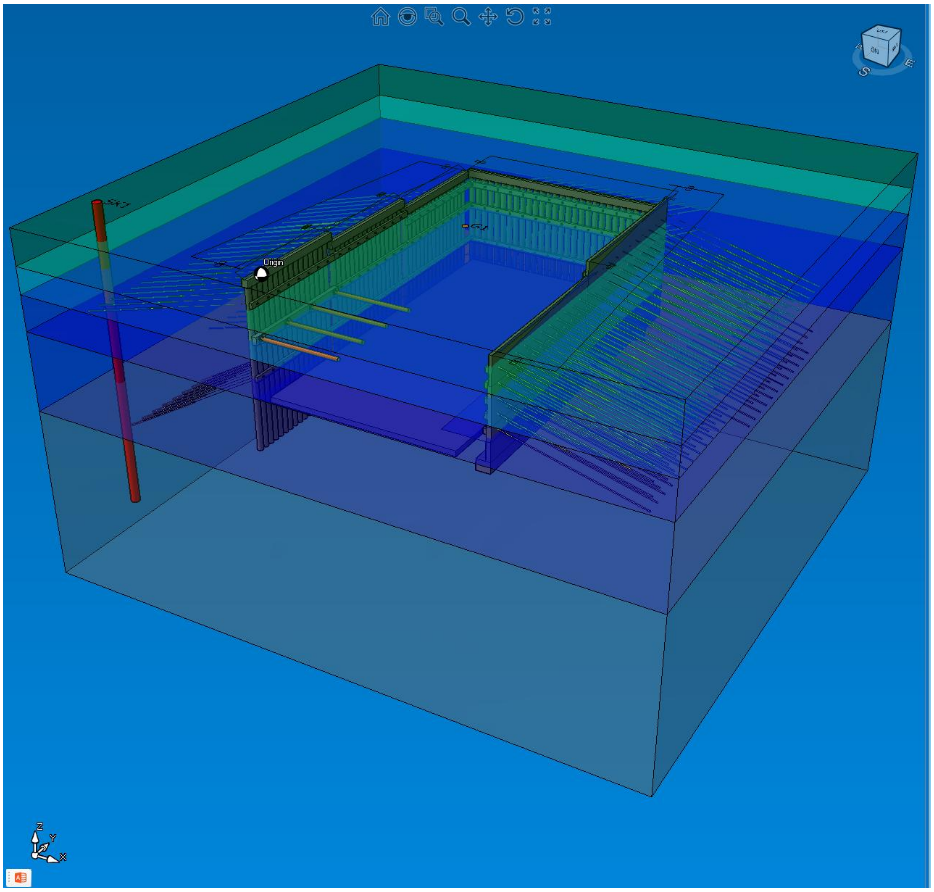


Figure 28. Wall Model

5.1. Wall Groups

To create a wall group, go to **Define > Add Wall Group** to open the **Wall Groups** window (Figure 29). In this window, the following parameters can be configured:

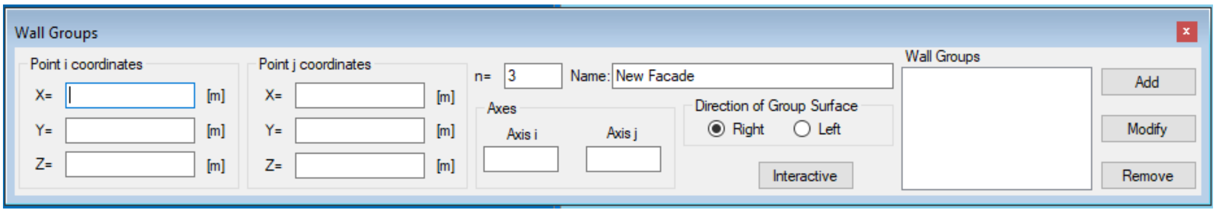


Figure 29. Wall Groups

- **X:** Terrain X-coordinate of the group’s start (i) or end (j) point
- **Y:** Terrain Y-coordinate of the group’s start (i) or end (j) point
- **Z:** Terrain Z-coordinate of the group’s start (i) or end (j) point
- **n:** Number of walls in the group
- **Name:** Name of the wall group
- **Axis i:** Name of the axis at point i
- **Axis j:** Name of the axis at point j
- **Wall Face Direction:** Direction of the earth pressure acting on the group’s face

After entering the parameters, the wall group is added to the model using the **Add** button. Alternatively, users can define the i and j point coordinates interactively by selecting them directly on the screen using the mouse cursor via the **Interactive** button.

When a wall group is created, the program applies the default wall section. If the Z-elevations at points i and j are the same, the default section is used directly. If the elevations differ, the wall sections are adjusted to best match the default configuration.

The **Edit** button opens the **Wall Group Properties** window. The **Delete** button removes the selected wall group from the model.

When the **Define > Selection Mode** option is active, wall groups can be selected by clicking on them in the 2D or 3D view. Double-clicking a selected group also opens the **Wall Group Properties** window. Similarly, pressing the **Delete** key on the keyboard while a group is selected will remove it.

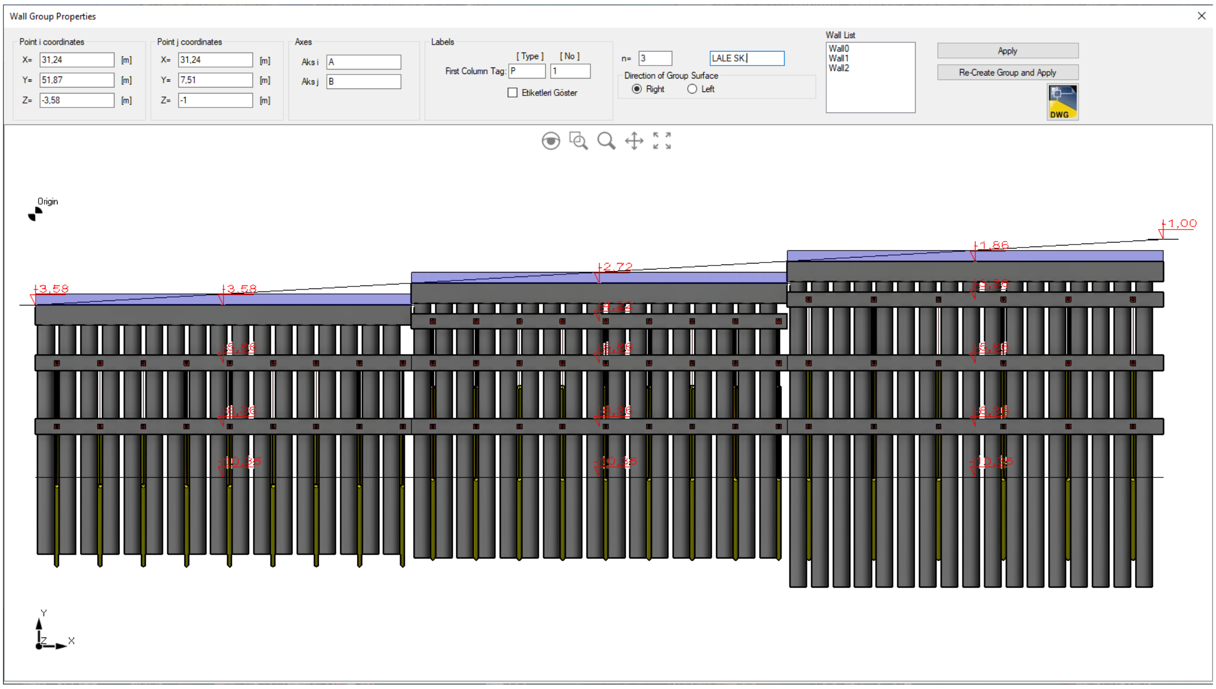


Figure 30. Wall Group Properties

In the **Wall Group Properties** window, the **Apply** button saves the changes. The **Rebuild and Apply Group** button reconstructs the wall group based on the default section settings and applies the changes.

By double-clicking on any wall listed in the wall list, the **Section** window opens, allowing the user to edit the cross-sectional properties of the selected wall.

5.2. Section

In the **Section** window, the following properties are defined:

- **Soil Profile:** The soil profile to be assigned to the section is selected.
- **Wall Type:** The type of wall is chosen, either a **reinforced concrete wall** or a **piled wall**. The **Wall Properties** button opens the window where wall-specific parameters can be edited.
- **Terrain:** The type of terrain behind the wall is selected.

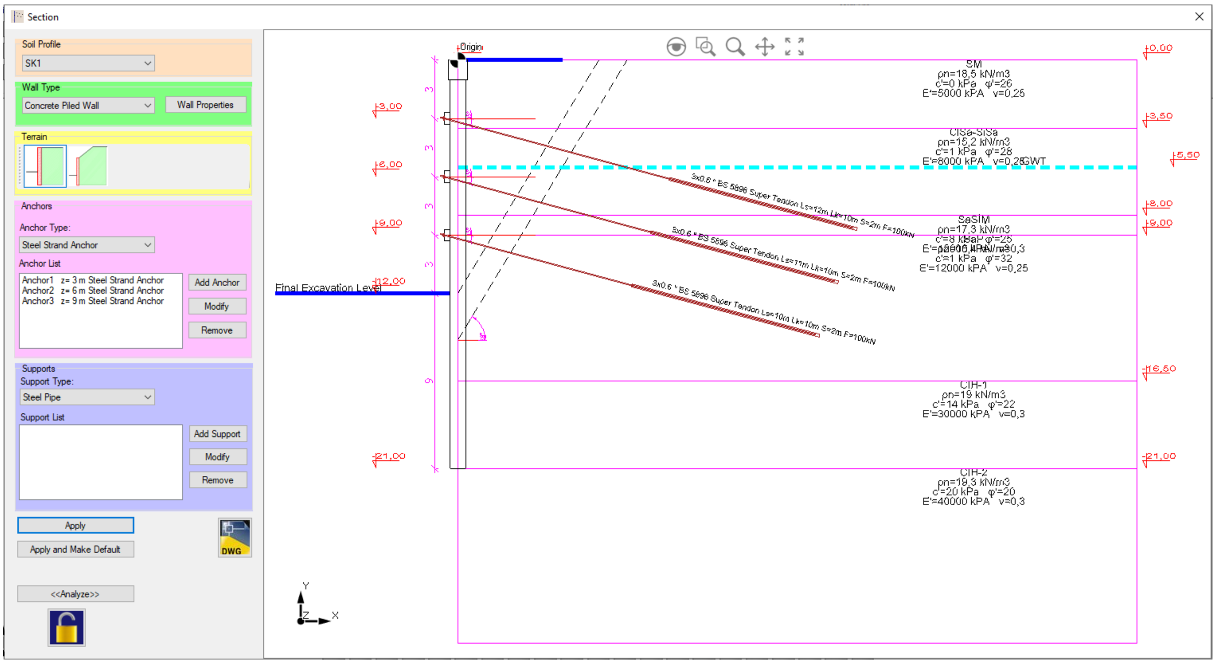


Figure 31. Section

- **Anchors:**

Anchor elements to be used in the wall are configured in this section. The anchor type can be selected as either **Steel Strand Anchor** or **Reinforced Soil Nail**. By clicking the **Add Anchor** button, the defined anchor is added to the list and assigned to the section. To edit an anchor, either double-click it in the list or select it and click the **Edit** button to open the **Anchor Properties** window. The **Delete** button removes the selected anchor from the list.

- **Supports:**

Support elements to be used in the wall are configured in this section. The available support types are **Steel Pipe**, **Steel I-Beam**, or **Reinforced Concrete Rectangular Beam**. By clicking the **Add Support** button, a support element with the defined properties is added to the list and assigned to the section. To edit a support, either double-click it in the list or select it and click the **Edit** button to open the **Support Properties** window. The **Delete** button removes the selected support from the list.

In the **Section** window, the **Apply** button applies the changes to the section. The **Apply and Set as Default** button saves the current section as the default section for future use.

5.2.1. Wall Properties

This window is used to configure the cross-sectional properties of the selected wall type (Figure 32):

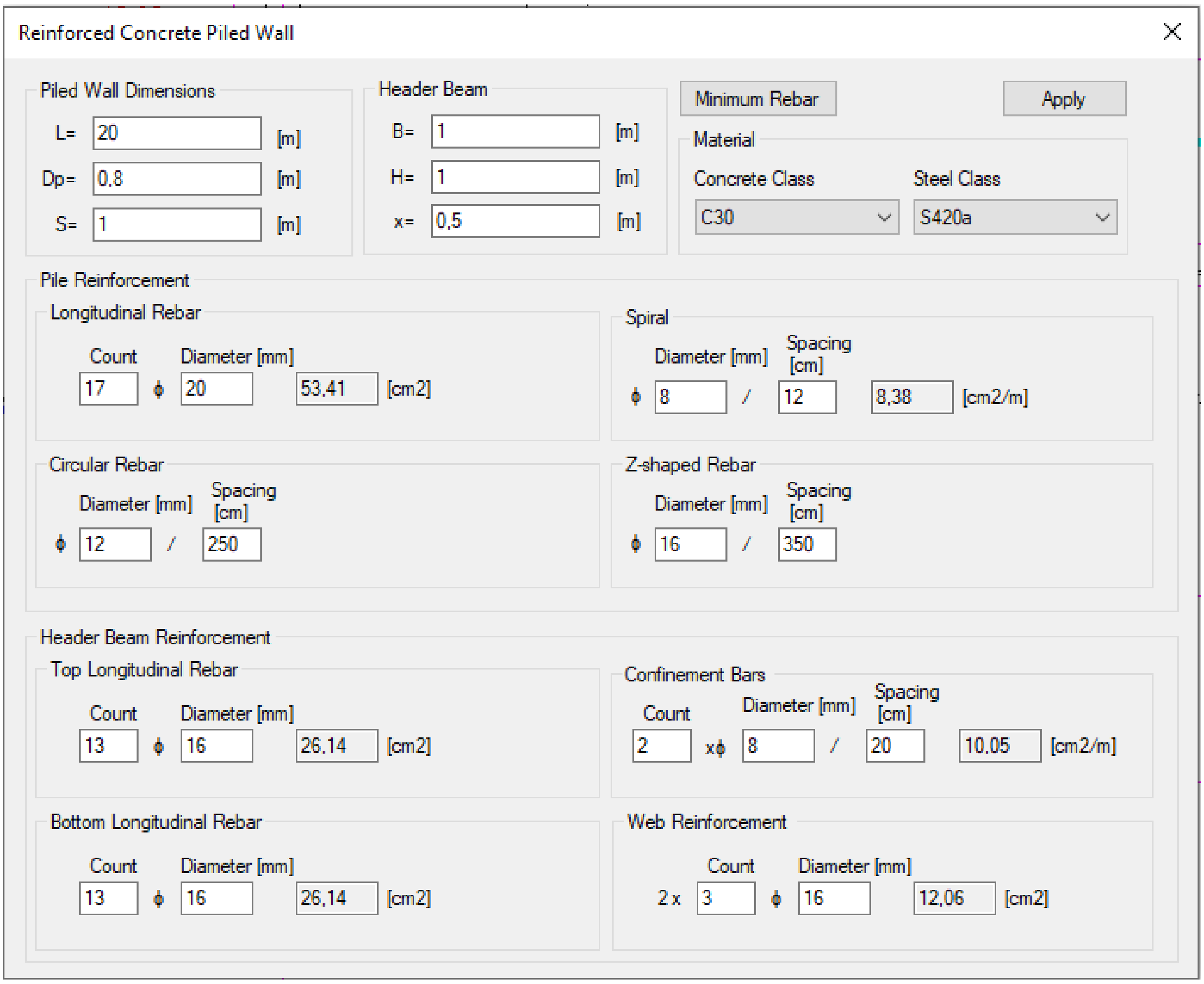


Figure 32. Reinforced Concrete Piled Wall

L: Pile length excluding the cap beam

D_p: Pile diameter

S: Spacing between pile centers

B: Cap beam width

H: Cap beam height

X: Offset distance from the edge of the capping beam to the first pile

The **Minimum Reinforcement** button calculates the minimum required reinforcement for the piles and cap beam according to **TS500 (Design and Construction Rules for Reinforced Concrete Structures)**. The calculated values are shown in the corresponding fields under **Pile Reinforcement** and **Cap Beam Reinforcement**. The user can manually modify the rebar diameter, spacing, and quantity if desired.

In the **Material** section, the **Concrete Class** and **Steel Class** for the reinforced concrete piled wall are selected. Changes are saved using the **Apply** button.

5.2.2. Steel Strand Anchor Properties

In the **Anchors** section of the **Section** window, select **Steel Strand Anchor** as the anchor type and click the **Add Anchor** button to insert a new anchor.

Using the **Modify** button, the **Steel Strand Anchor Properties** window opens (Figure 33), where the following parameters are defined.

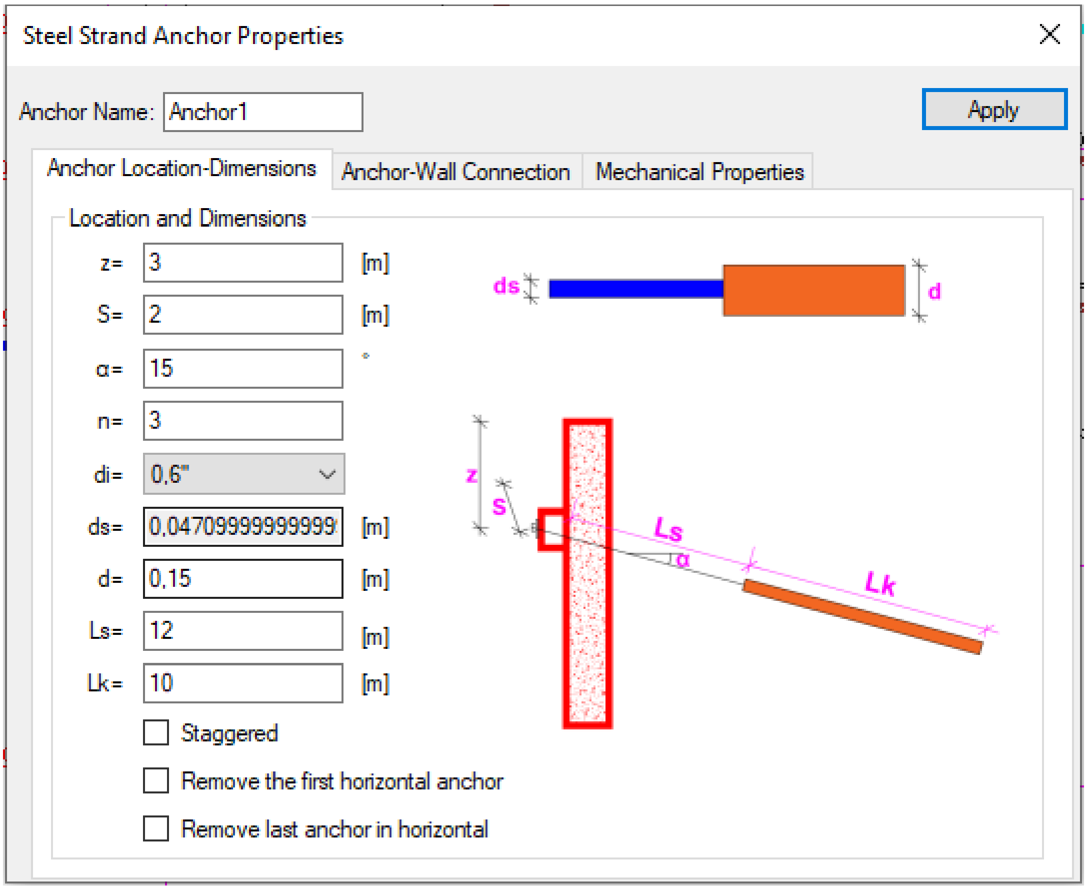


Figure 33. Steel Strand Anchor Properties

Anchor – Placement Dimensions Tab:

- **z:** Anchor elevation
- **S:** Horizontal spacing
- **α:** Angle between the anchor and the horizontal
- **n:** Number of strands
- **d_i:** Diameter of a single strand
- **d_s:** Combined nominal diameter for multiple strands
- **d:** Root zone diameter
- **L_s:** Free length
- **L_k:** Bond (root) length
- **Staggering:** If checked, applies staggering relative to other rows
- **Remove First Anchor Horizontally:** Removes the first anchor in the row if checked
- **Remove Last Anchor Horizontally:** Removes the last anchor in the row if checked

Anchor – Wall Connection Tab:

- **Plate Dimensions Section:**
  - **A:** Plate width
  - **B:** Plate height
  - **H:** Plate thickness
  - **H_r:** Secondary plate thickness for angled plates
- **Plate Type Section:** Select either **Flat** or **Angled** plate
- **L_d:** Length of the anchor extending beyond the plate
- **Anchor Head Section:**
  - **D:** Head diameter

- **H:** Head thickness

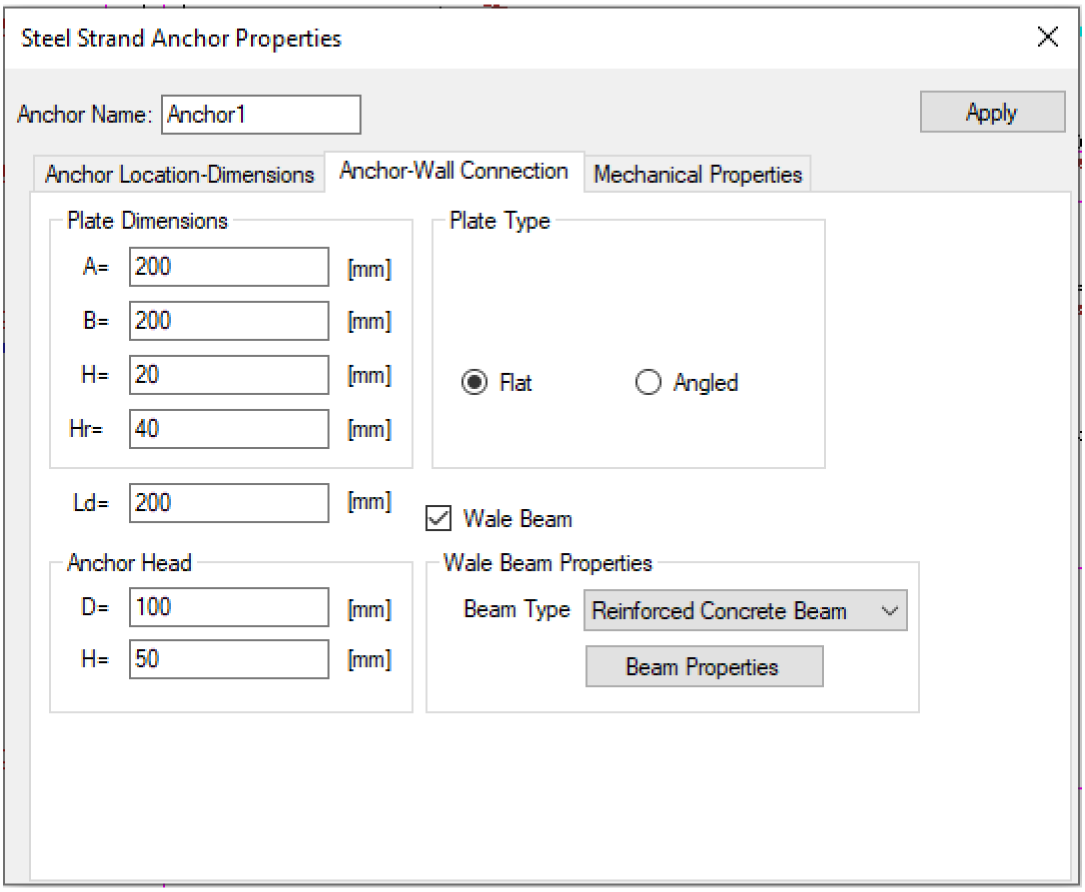


Figure 34. Steel Strand Anchor Properties / Anchor–Wall Connection

- **Waler Beam:** If checked, the **Waler Beam Properties** section is activated. The waler type can be selected as either **Reinforced Concrete Beam** or **Steel U Profile**. The **Beam Properties** button opens the waler configuration window.
    - If **Reinforced Concrete Beam** is selected, the **Dimensions** section allows input of **Width (B)** and **Height (H)** of the beam.
    - In the **Material** section, select concrete and steel grades used for the reinforced concrete waler.
    - The **Minimum Reinforcement** button calculates the required reinforcement according to TS500 and displays it under the **Reinforcement** section. These values can be modified by the user.

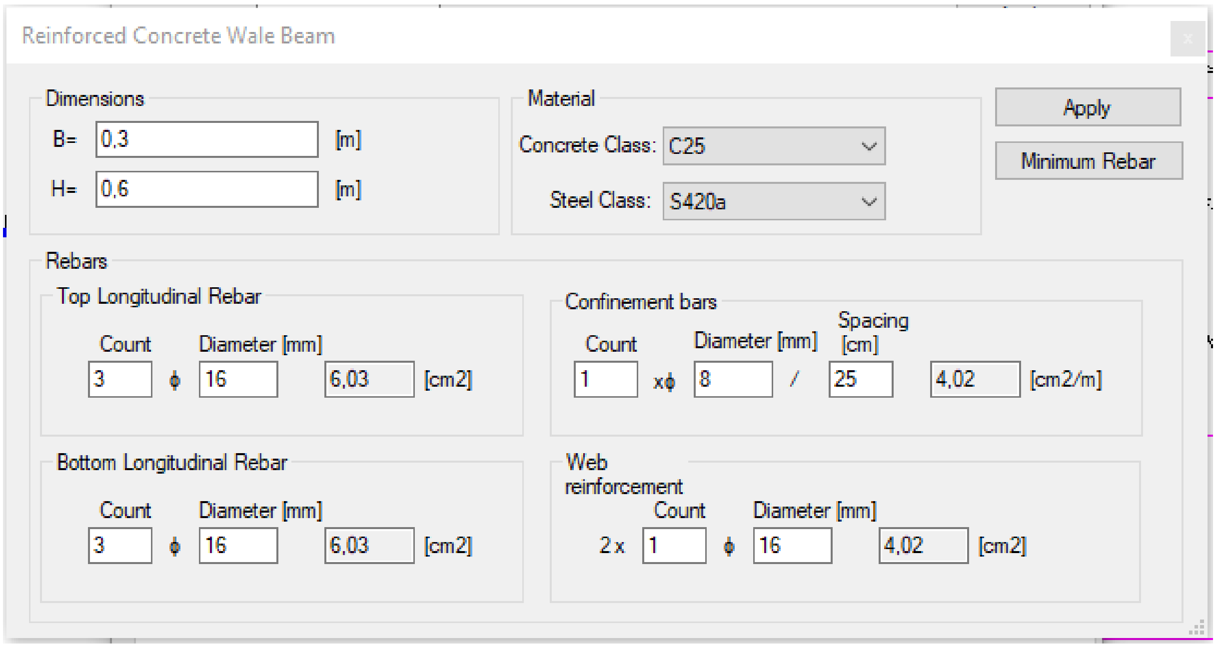


Figure 35. Reinforced Concrete Waler Beam

Mechanical Properties Tab:

- **Mechanical Properties Section:**
  - **F:** Prestressing force
  - **E:** Elastic modulus of the steel strand
  - **ν:** Poisson's ratio of the steel strand
- **Plate Material (Plate, Head) Section:** Select material for the plate and head.
- **Soil–Grout Bond Resistance Section:** Select calculation method (**User-Defined**, **Empirical Data**, **Effective Stress**, or **Total Stress**)
  - **F_s:** Soil–grout bond resistance
  - **f:** Ultimate skin friction
  - **ξ:** Safety factor for empirical method
  - **K_1:** Earth pressure coefficient for anchors constructed with gravity grouting; varies with soil type and density, typically ranging between 1.4 and 2.3. For loose silty or fine sandy soils, it may drop to 1.0–1.15

- **Strand–Grout Bond Resistance Section:** Select calculation method (**User-Defined**, **TS500**, or **ACI 318-11**)
    - **F_k:** Strand–grout bond resistance
    - **f_c:** Compressive strength of grout

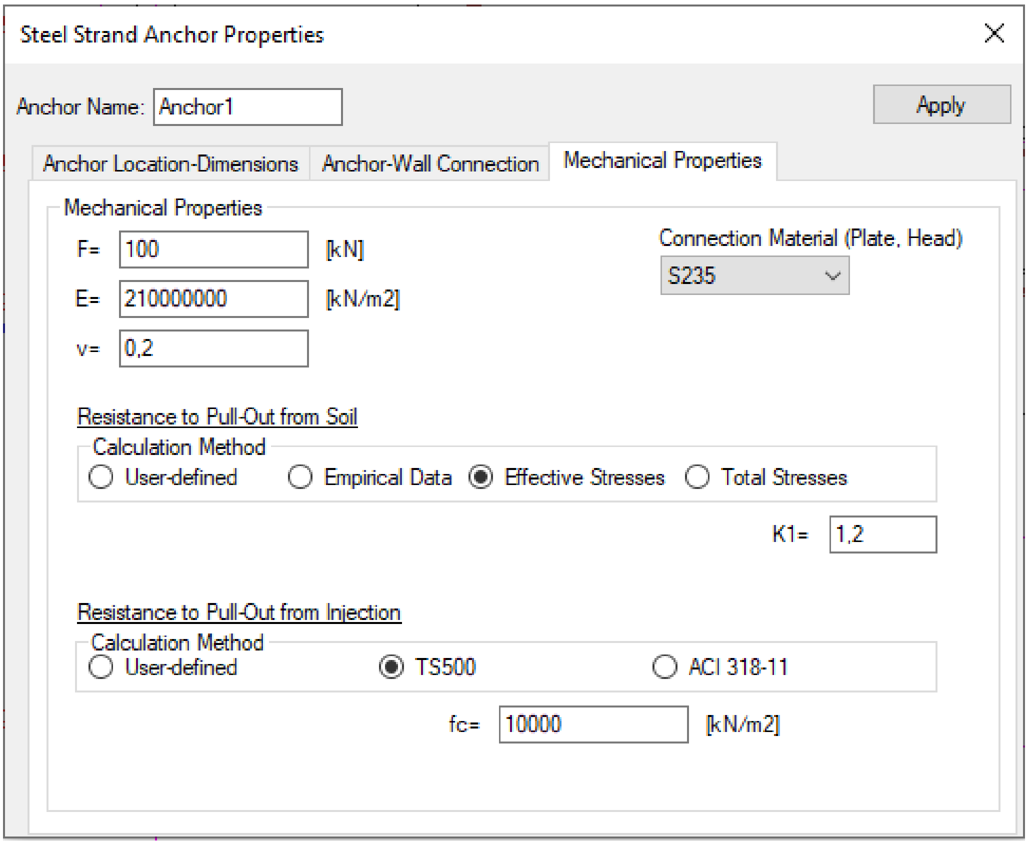


Figure 36. Steel Strand Anchor Properties / Mechanical Properties

Clicking the **Apply** button saves the changes made.

5.2.3. Reinforced Soil Nail Properties

In the **Anchors** section of the **Wall Section** window, select **Reinforced Soil Nail** as the anchor type. Click the **Add Anchor** button to add a new soil nail. Use the **Edit** button to open the configuration window for soil nail properties.

In the opened window, the following parameters are defined:

- **Anchor Name:** The name assigned to the anchor.

- **Nail Placement – Dimensions Tab:**
  - **z**: Depth (vertical distance from the top of the wall to the anchor elevation)
  - **S**: Horizontal spacing
  - **α**: Angle of the nail with respect to the horizontal
  - **n**: Number of nails
  - $\mathbf{d_i}$: Diameter of the reinforcing bar
  - $\mathbf{d_s}$: Diameter of the anchor bar; for multiple bars, this represents their nominal combined diameter.
- **d**: Overall nail diameter
- **L**: Total length of the nail

Additional Options:

- **Staggering**: When checked, nails in the current anchor row are staggered relative to those in adjacent rows.
- **Remove First Anchor Horizontally**: Removes the first anchor in the current row.
- **Remove Last Anchor Horizontally**: Removes the last anchor in the current row.

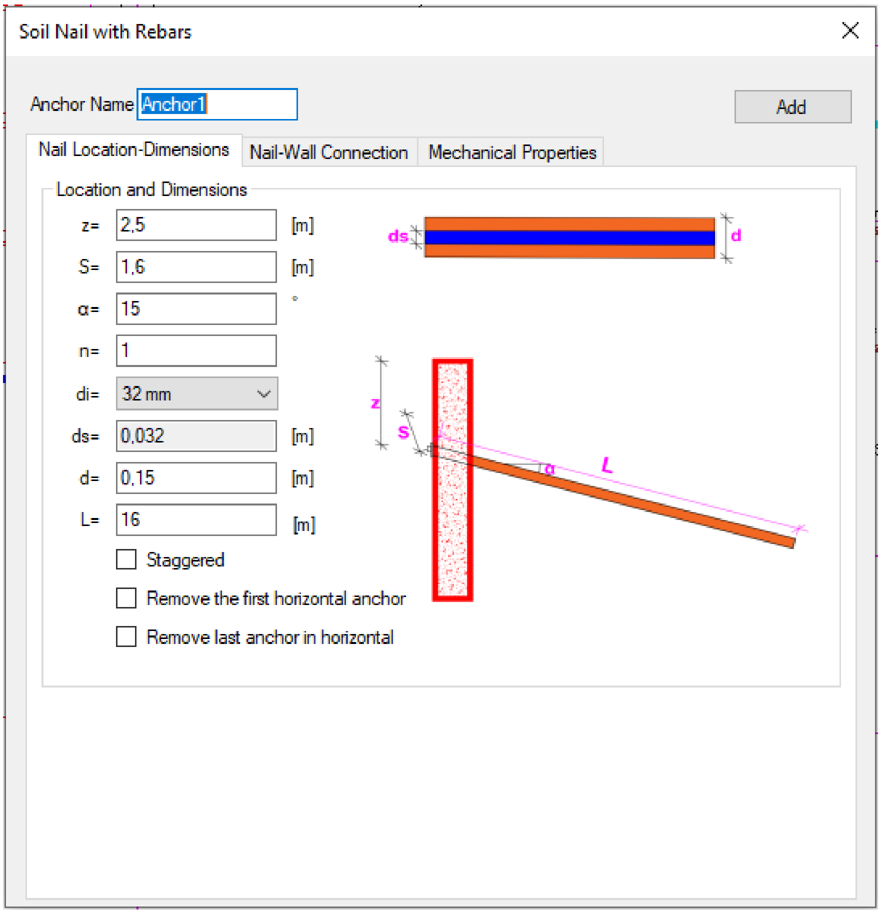


Figure 37. Reinforced Soil Nail

Nail–Wall Connection Tab:

- **Plate Dimensions Section:**
  - **A**: Plate width
  - **B**: Plate height
  - **H**: Plate thickness
  - $\mathbf{H_r}$: Secondary plate thickness (for angled plates)

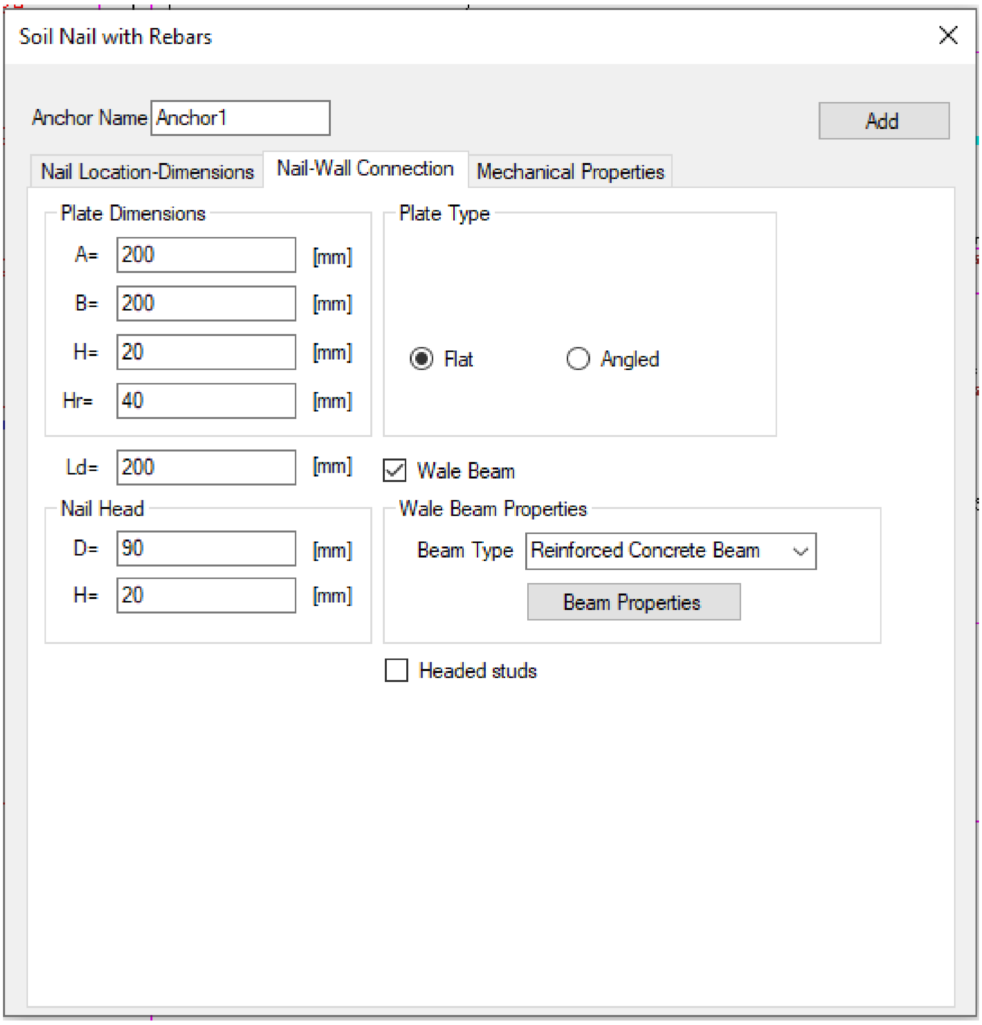


Figure 38. Reinforced Soil Nail / Nail–Wall Connection

- **Plate Type Section:**
  - Choose either *Flat* or *Angled*
  - **L$_d$**: Length of the anchor protruding beyond the plate
- **Nail Head Section:**
  - **D**: Head diameter
  - **H**: Head thickness
- **Waler Beam Option:**

- When the **Waler Beam** checkbox is selected, the **Waler Beam Properties** section becomes active.
- The waler type can be selected as either **Reinforced Concrete Beam** or **Steel U Profile**.

- **Reinforced Concrete Beam Properties (if selected):**
  - **Dimensions Section:**
    - **B**: Waler beam width
    - **H**: Waler beam height
  - **Material Section:**
    - Selection of concrete and steel materials to be used for the waler
  - **Minimum Reinforcement:**
    - The **Minimum Reinforcement** button calculates the minimum required reinforcement based on **TS500**.
    - Calculated values are displayed in the **Reinforcement** section.
    - The user can modify the diameter, spacing, and number of reinforcement bars.

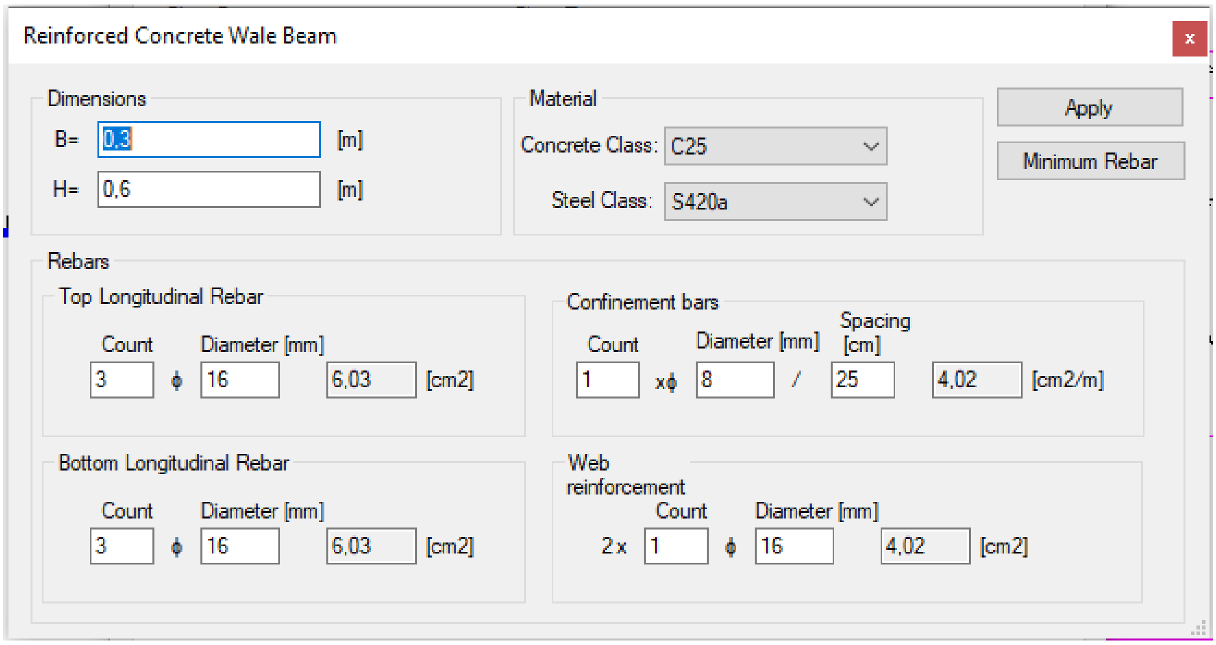


Figure 39. Reinforced Concrete Waler Beam

Mechanical Properties Tab:

- **Mechanical Properties Section:**
  - **Steel Grade of Nail** and **Connection Material (Plate, Head)** are selected.
- **Soil–Grout Bond Resistance Section:**
  - Available calculation methods: **User-Defined**, **Empirical Data**, **Effective Stress**, **Total Stress**, or **Pull-Out Tests**
  - $\mathbf{F_s}$: Soil–grout bond resistance
  - $\boldsymbol{\tau_{bu}}$: Ultimate shaft friction
  - $\mathbf{K_1}$: Earth pressure coefficient for nails installed using gravity grouting
    - Varies between **1.4–2.3** depending on soil type and relative density.
    - For fine sands and silty soils with low relative density, K_1 may drop to values between **1.0 and 1.15**.

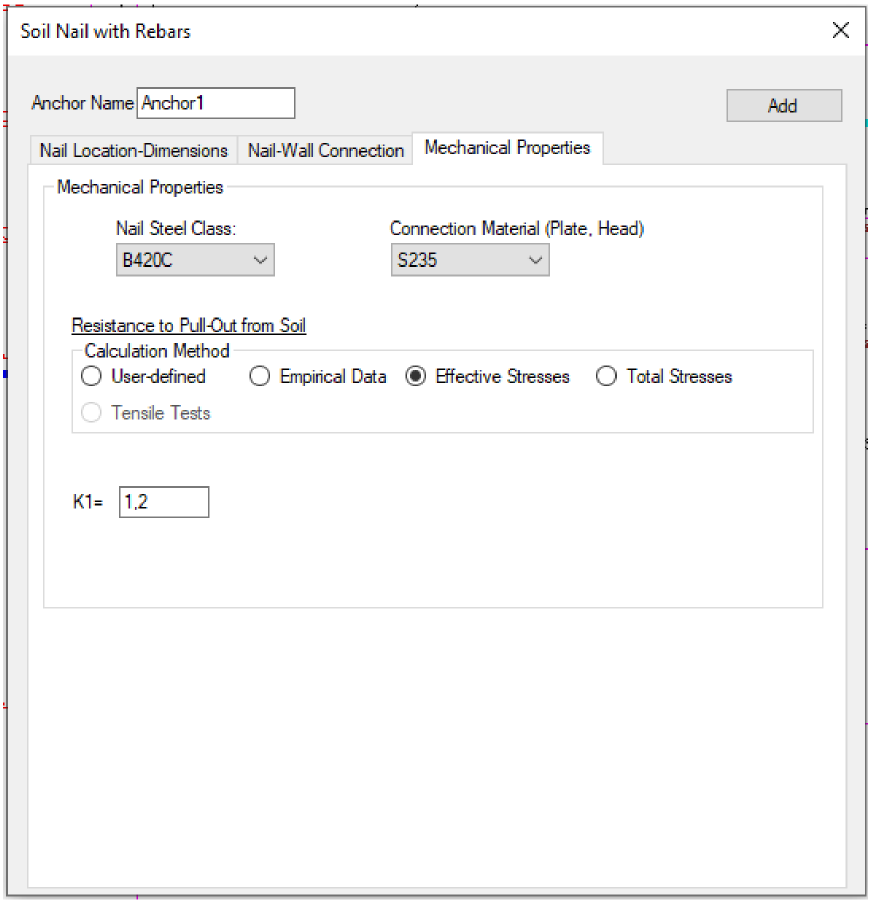


Figure 40. Reinforced Soil Nail / Mechanical Properties

5.2.4. Steel Pipe Support Properties

In the **"Supports"** section of the wall section window, **"Steel Pipe"** is selected as the strut type. Click the **"Add Support"** button to add a new steel pipe strut. To edit the properties of an existing strut, use the **"Edit"** button. The following parameters are defined in the opened window:

- **Support Name:** Name assigned to the support.
- **Placement and Dimensions Section:**
  - **z**: Support installation depth
  - **S**: Horizontal spacing
  - **L**: Support length

- **Angled Support Option (Horizontal):**
    - When selected, enables the input field for horizontal inclination.
    - β: Angle between the support and horizontal
- **Remove First Support Option (Horizontal):**:
    - When selected, removes the first support in the corresponding row.
- **Remove Last Strut Option (Horizontal):**
    - When selected, removes the last support in the corresponding row

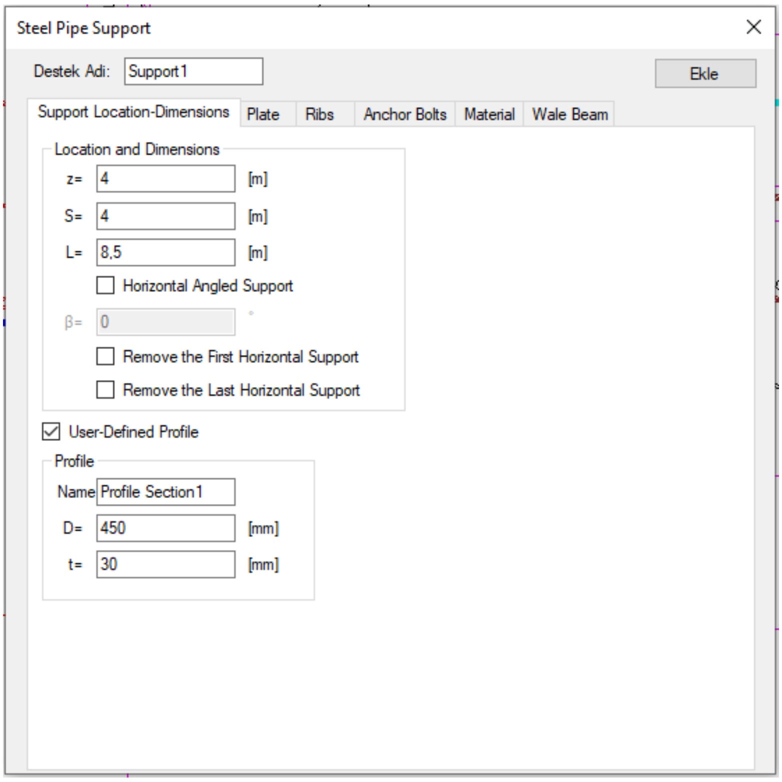


Figure 41. Steel Pipe Support / Support Layout and Dimensions

Profiles Section:

- Select a predefined profile for the steel pipe strut from the dropdown list.
- To manually define a custom profile, check **"User-Defined Profile"**.
- When selected, the following fields are enabled:

- **Name:** Section name of the steel profile
- **D:** Outer diameter of the pipe
- **t:** Wall thickness of the pipe

Plate Tab:

- **Plate Geometry Section:** Choose between square or circular plate geometry.

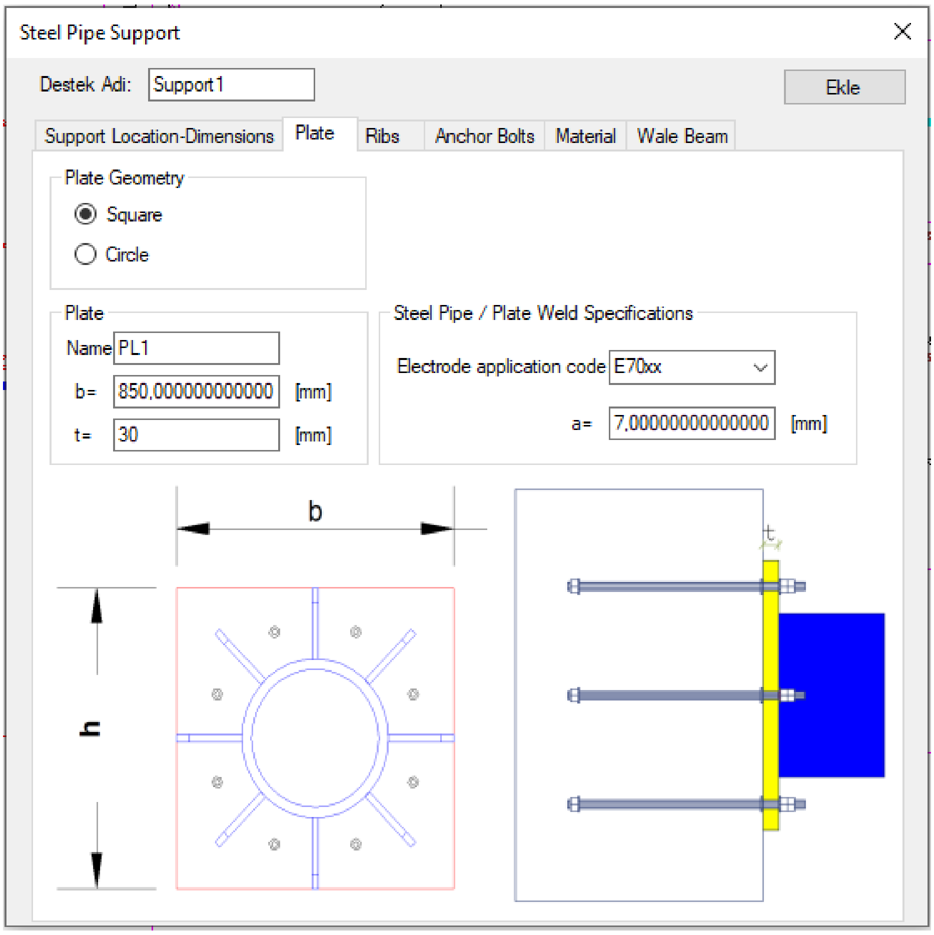


Figure 42. Steel Pipe Support / Plate

Plate Section:

- **Name:** Plate name
- **b:** Plate width
- **t:** Plate thickness

Weld Properties Section (Pipe-to-Plate):

- Select **"Electrode Application Code"** from the dropdown.
- **a:** Weld thickness

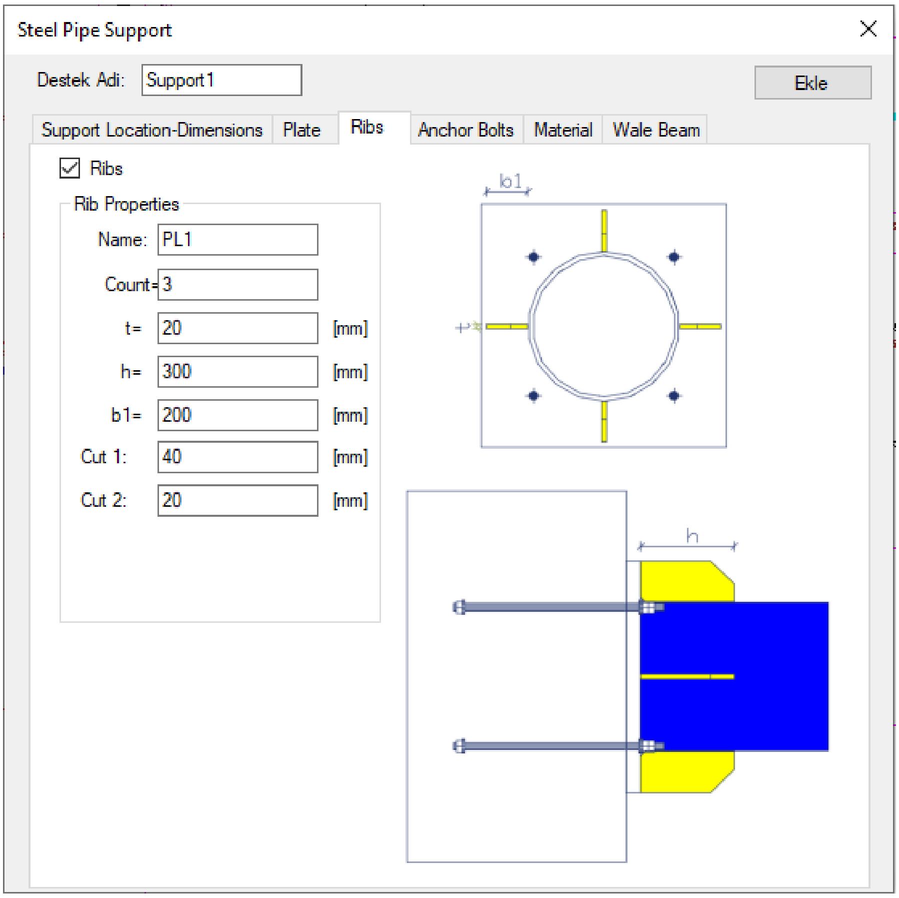


Figure 43. Steel Pipe Support / Ribs

Rib Plates Tab:

- **Rib Properties Section:**
  - **Name:** Rib name
  - **Quantity:** Number of ribs
  - **t:** Rib thickness
  - **h:** Rib height
  - **b1:** Rib width

- **Chamfer1:** Cut dimensions for rib corner 1
- **Chamfer2:** Cut dimensions for rib corner 2

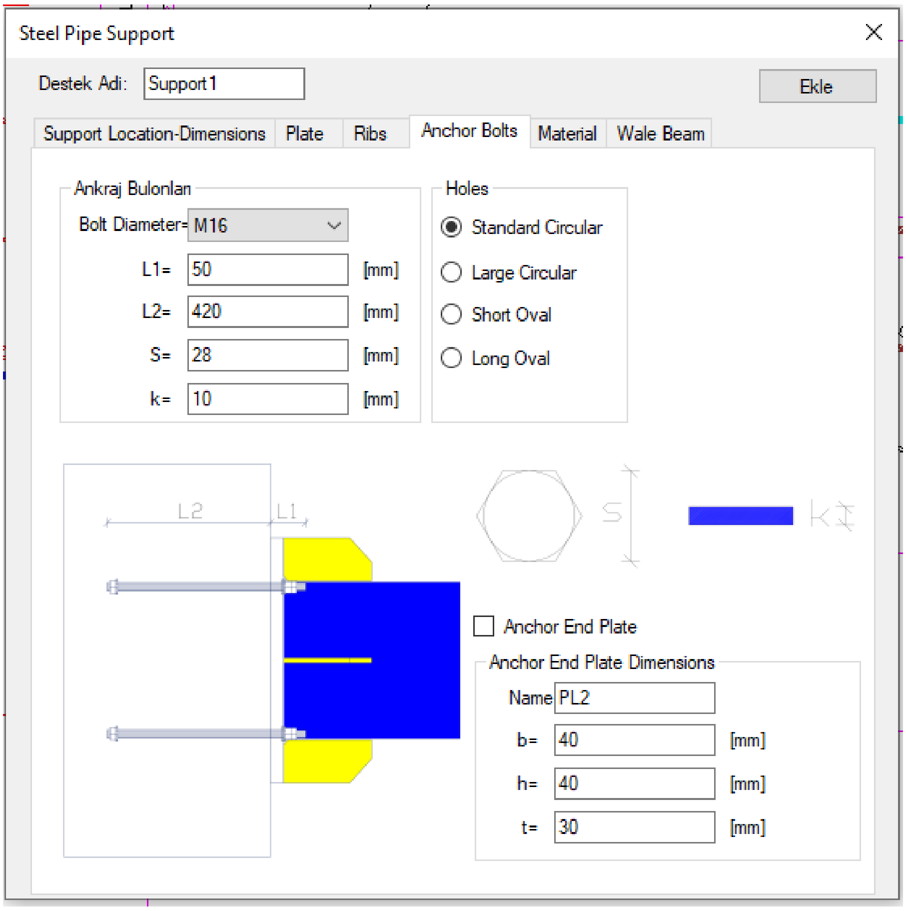


Figure 44. Steel Pipe Support / Anchor Bolts

Anchor Bolts Tab:

- **Anchor Bolt Section:**
  - **Bolt Diameter:** Select from dropdown
  - **L_1:** Length above the plate
  - **L_2:** Length below the plate
  - **S:** Inner diameter of bolt head
  - **K:** Bolt head height

- **Holes (TCY) Section:**
    - **Select hole type:** Standard circular, Large circular, Short oval, or Long oval
- **Anchor End Plate Dimensions Section:**
    - **Name:** End plate name
    - **b:** Plate width
    - **h:** Plate height
    - **t:** Plate thickness

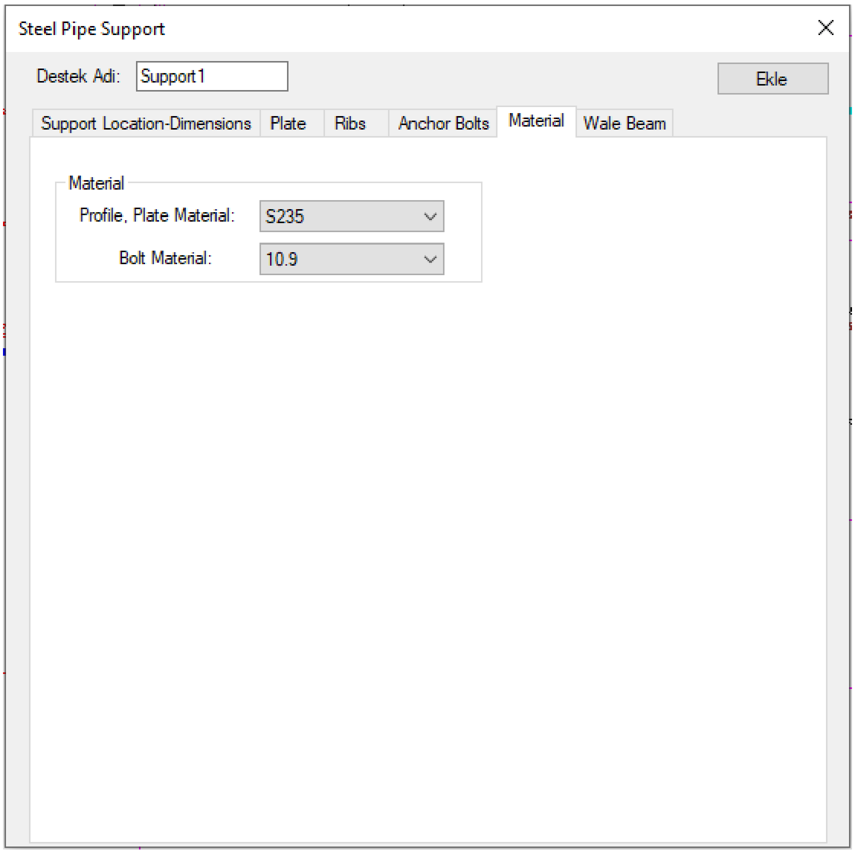


Figure 45.Steel Pipe Support / Material

Material Tab:

- **Material Section:** Select **Profile Material**, **Plate Material**, and **Bolt Material** from dropdown lists.

- **Waler Beam Option:** When selected, the **"Waler Beam Properties"** section is enabled. Choose the waler type as **"Reinforced Concrete Beam"** or **"Steel U-Section"**.

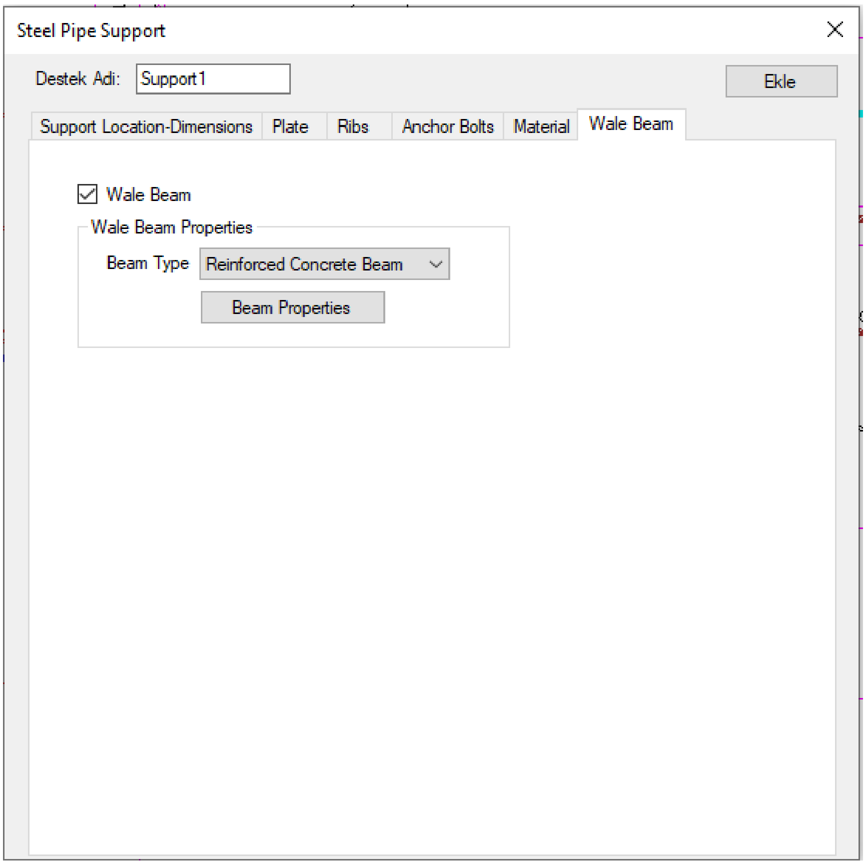


Figure 46. Steel Pipe Support / Waler Beam

- **Reinforced Concrete Waler Properties (when RC Beam is selected):**
  - **Dimensions Section:**
    - **B:** Waler beam width
    - **H:** Waler beam height
  - **Material Section:** Select concrete and steel grades to be used
  - **Minimum Reinforcement:**
    - Press the **"Minimum Reinforcement"** button to calculate reinforcement as per TS500.

- The results are displayed in the **"Reinforcement"** section and can be edited by the user if needed (diameter, spacing, quantity, etc.).

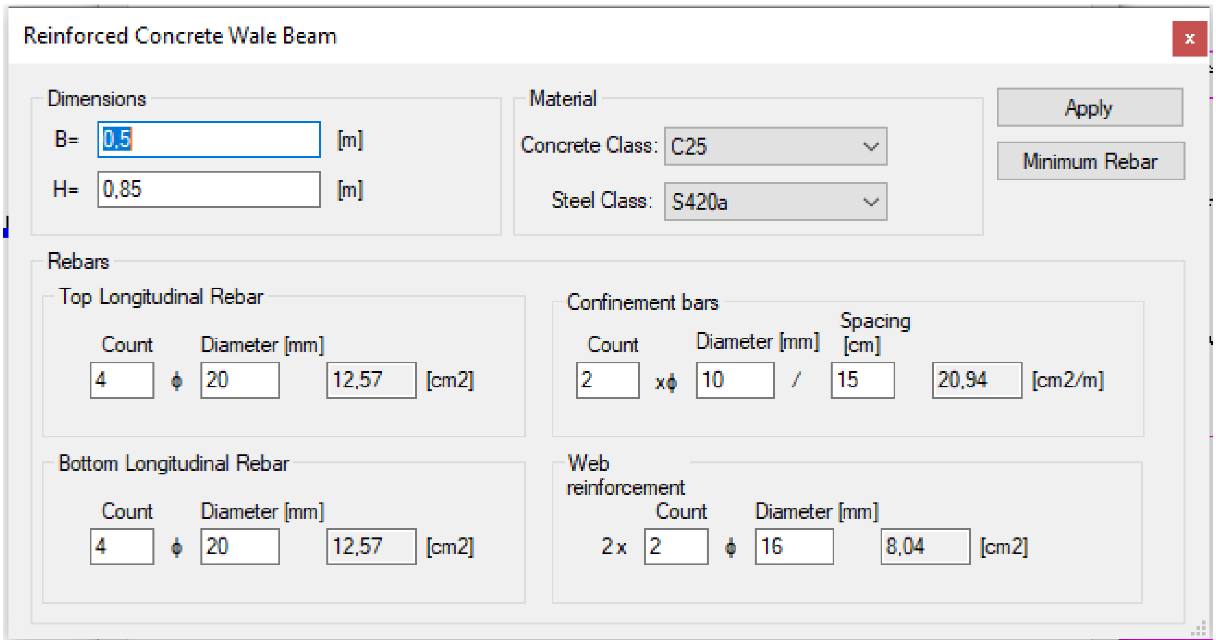
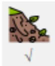


Figure 47. Reinforced Concrete Waler Beam

6. SLOPE STABILITY 

To model slopes, click **Define > Slopes**, which opens the **Slope Analysis List** window.

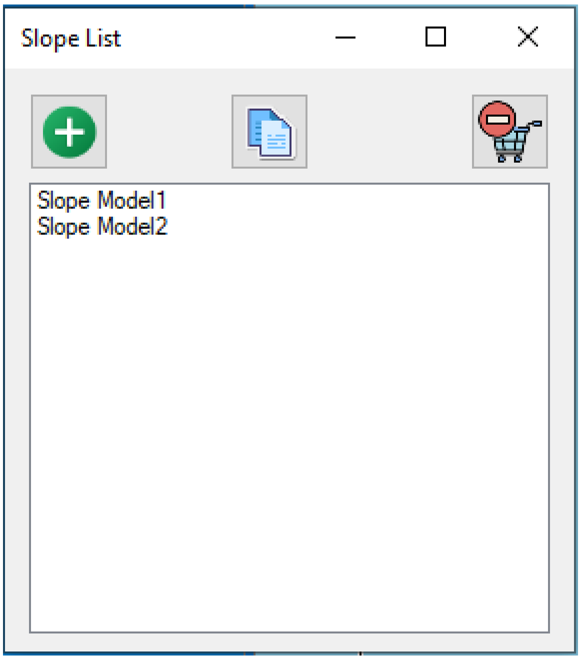


Figure 48. Slope Analysis List

 : Adds a new slope model to the analysis list.

 : Duplicates the selected slope model from the list.

 : Removes the selected slope model from the list.

Double-clicking a model in the **Slope Analysis List** opens its **Slope Model** window.

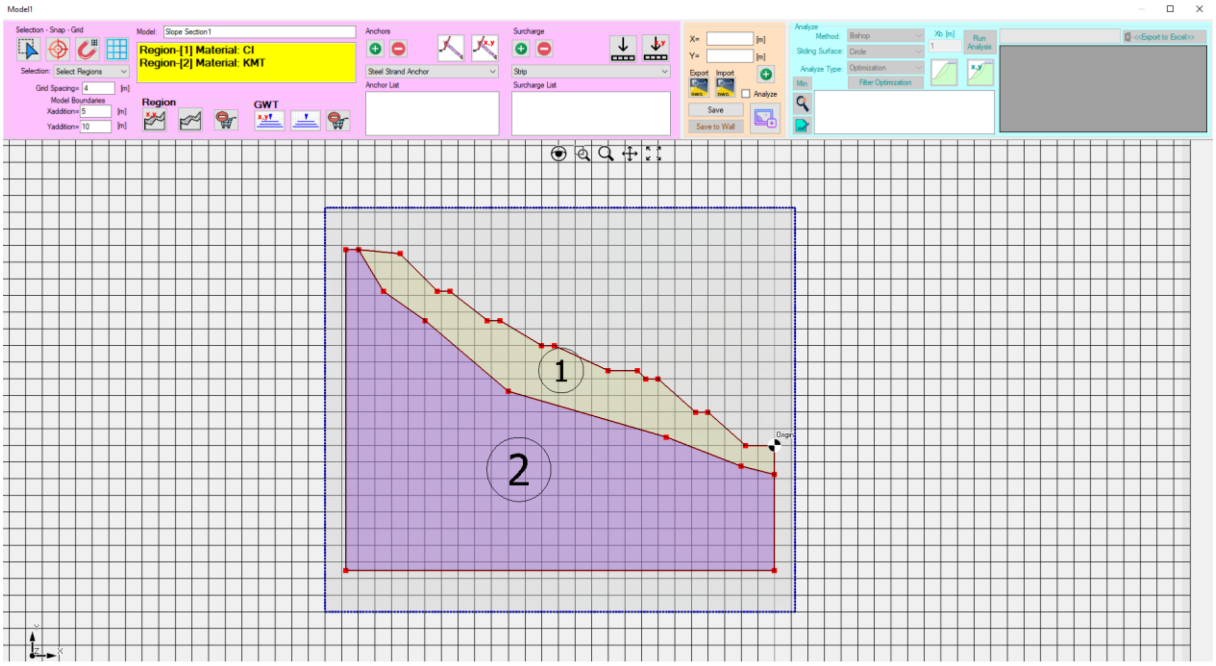


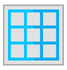
Figure 49. Slope Model

In the **Selection–Snap–Grid** section:

 : Allows you to select objects created in the model window using the cursor on the screen.

 : Enables the feature of capturing objects created in the model window using the cursor on the screen.

 : Enables the feature of capturing grids in the model window using the cursor on the screen.

 : Opens and closes grids in the model window.

Selection Mode: Choose from **Select Areas**, **Select Area Points**, **Select GWL Points**, **Select Anchors**, or **Select Surcharge Loads**, which adjusts the Select button's function.

Grid Spacing: Input intervals for the grid display.

In the **Model Boundaries** section:

- $X_{\text{-Offset}}$: Horizontal offset from the outermost area points
- $Y_{\text{-Offset}}$: Vertical offset from the outermost area points
- Model: Name of the slope model

Below this, in the yellow panel, the numbered regions and assigned materials are displayed.

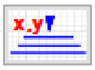
In the **Region** section:

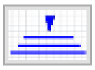
 : Creates a region by entering coordinates.

 (Interactive): Creates a region with cursor input.

 : Removes an existing region.

In the **GWL (Groundwater Level)** section:

 : Creates a GWL by entering coordinates.

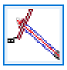
 : Creates a GWL with cursor input.

 : Removes the GWL.

In the **Anchors** section:

 : Adds an anchor.

 : Deletes an anchor.

 : Adds an anchor interactively.

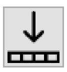
 : Adds an anchor by entering coordinates.

Select anchor type (**Steel Strand Anchor** or **Reinforced Soil Nail**) from the dropdown. Added anchors appear in the **Anchor List**.

In the **Surcharges** section:

 : Adds a surcharge load.

 : Deletes a surcharge load.

 : Adds a surcharge load interactively.

 : Adds a surcharge load by entering coordinates.

Select the surcharge type (**Point**, **Line**, **Strip**, or **Area**) from the dropdown. Added surcharges appear in the **Surcharge List**.

X= [m]
Y= [m]
Coordinates entered in the yellow X and Y fields are used to add areas, GWT, anchors, surcharge loads, and slip surfaces.

: Adds data by entering coordinates.

Export
: Saves the created slope model in .dwg format.

Import
: Imports a cross-sectional slope model created in .dwg format.

Checking the **Analysis** box disables the data entry sections and activates the analysis section.

The "Save" button saves the created slope model.

The **Save to Wall** button saves the analysis results of the wall section checked for external stability into the respective wall's analysis results.

: Used to add images and tables to the geotechnical design report.

In the **Analysis** section:

- The **Method** option allows selecting either the Oms/Fellenius or Bishop method from the dropdown list.
- The **Slip Surface** option allows choosing a circular or polygonal shape.
- The **Analysis Type** option allows selecting either standard or optimization from the dropdown list.

X_b: Slice width

: Interactive slip surfaces can be added.

: Adds a slip surface by entering coordinates.

Min : Selects the minimum factor of safety calculated from the analysis results.

The **Filter Optimization** button removes steps from the optimization results that contain geometric errors or could not be solved.

: Optimization Settings

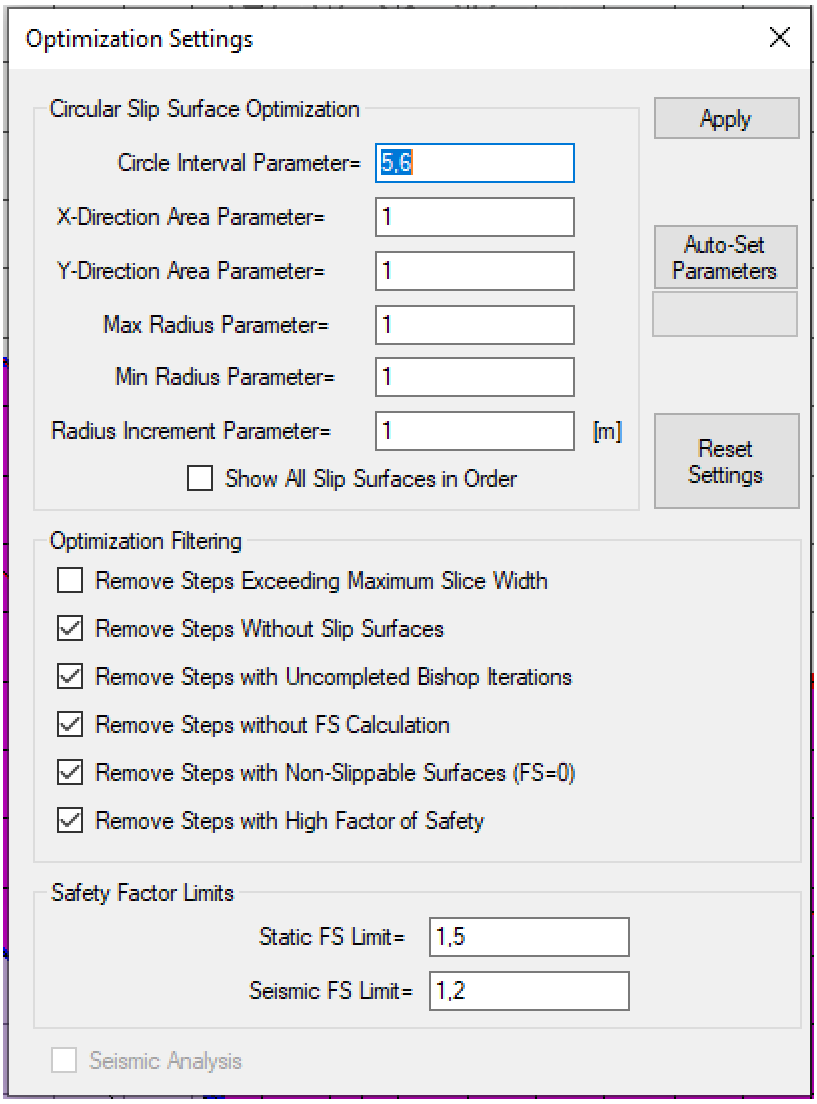


Figure 50. Optimization Settings

In the **Circular Slip Surface Optimization** section, optimization parameters are entered to find the slip circle that yields the lowest factor of safety, based on the initially defined slip circle.

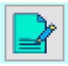
If the **Show All Slip Surfaces Sequentially** option is selected, all slip surfaces identified in the analysis will be displayed one by one on the slope model screen.

The **Set Parameters** button allows the software to automatically determine the parameters for circular slip surface optimization.

In the **Optimization Filtering** section, the properties of slip circles to be removed when clicking the **Filter Optimization** button on the slope model screen are selected.

In the **Factor of Safety Limit Values** section, FS limit values are defined for both static and seismic conditions.

The **Reset Settings** button restores all changes made in the settings window back to their original state.

 : Generates the analysis report.

7. LABORATORY TESTS

By clicking the **Laboratory Tests** button in the **Define** menu, the laboratory tests window opens (Figure 51).

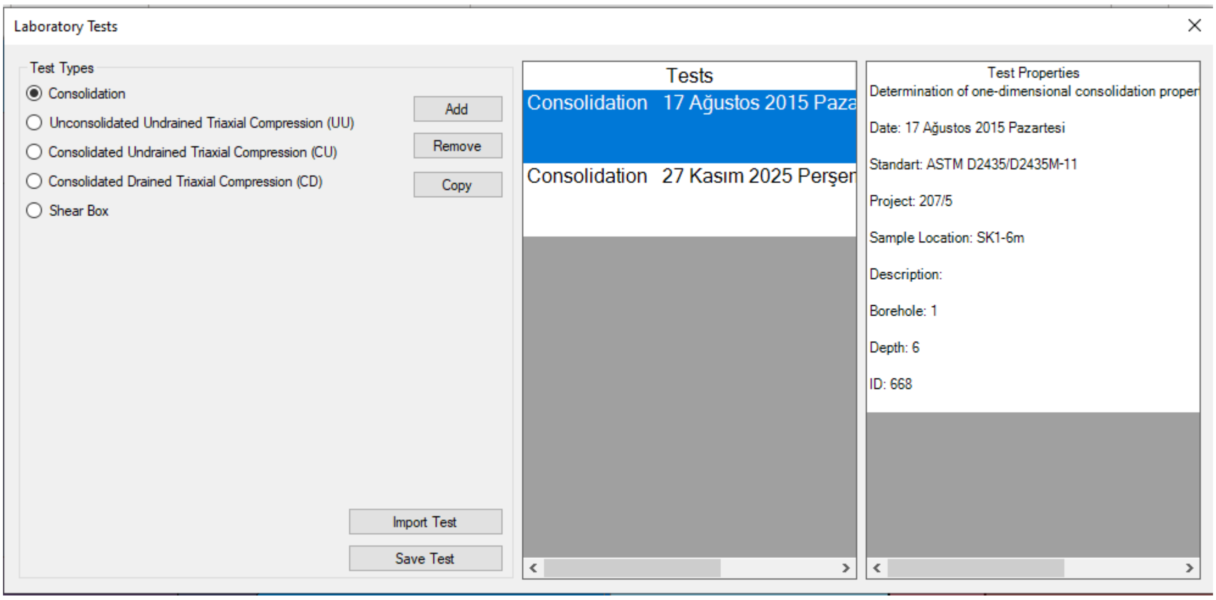


Figure 51. Laboratory Tests

From the **Test Types** section, the laboratory test to be created is selected and added to the **Tests** section by clicking the **Add** button. When the report of the added test is checked, summary information related to the report is displayed in the **Test Properties** section. With the **Delete** and **Copy** buttons, the selected test in the **Tests** section can be deleted or copied.

The prepared test report is saved to the computer with the extension **.slt** by clicking the **Save Report** button. Later, saved reports can be re-imported into the **Tests** section by using the **Import Report** button.

7.1. Consolidation Test

The program generates a test report containing graphs and tables by calculating the consolidation parameters from the data obtained in the laboratory consolidation test.

By double-clicking the consolidation test created in the **Tests** section, the **Consolidation Test** window opens (Figure 52).

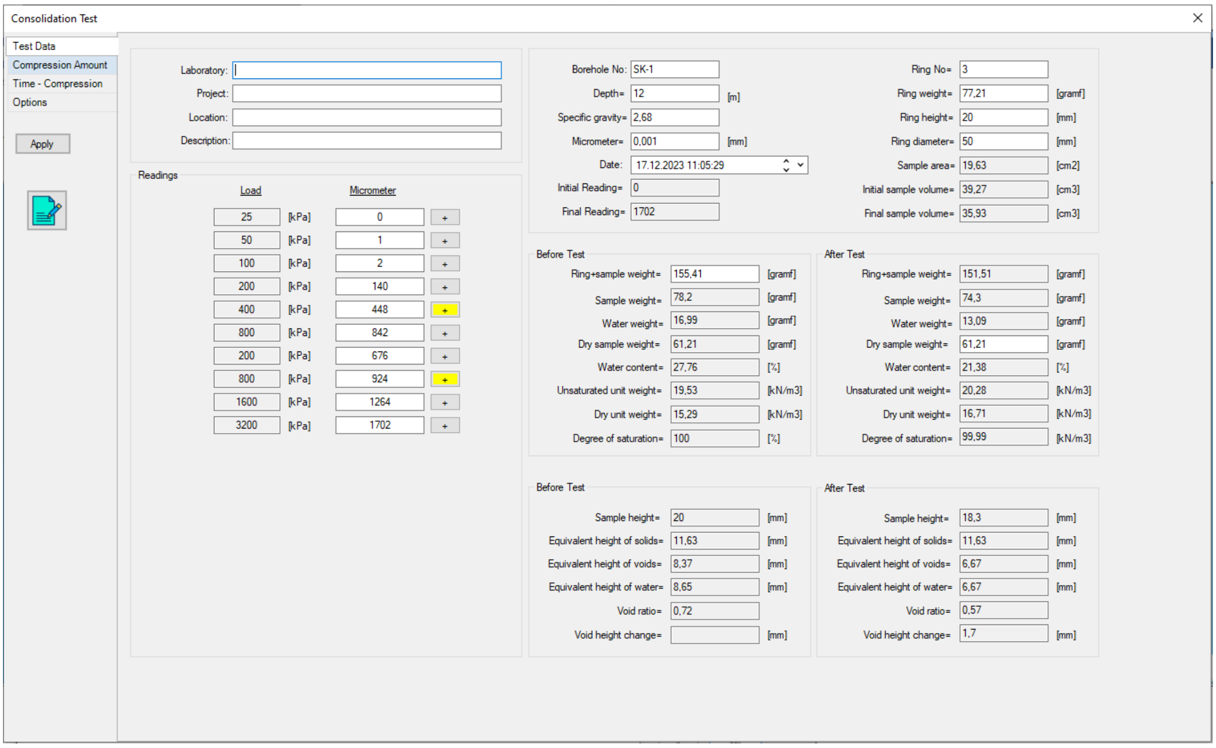


Figure 52. Consolidation Test

In the opened window, under the **Test Data** tab, the fields where test information will be entered are displayed open, while the fields where calculated results will appear remain closed.

In the **Readings** section, micrometer readings corresponding to the load set used in the test are entered. Micrometer readings corresponding to the load values can be quickly added using the adjacent + sign.

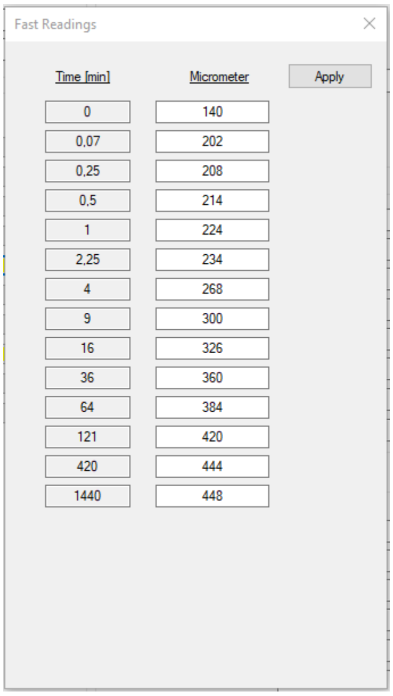


Figure 53. Quick Reading Data

In the load stage, where quick reading data is entered, the related + button in that row is highlighted in yellow.

In the **Compression Amount** tab, a calculation table for the e–log(σ) curve is generated, and Pressure–Void Ratio curves are plotted (Figure 54).

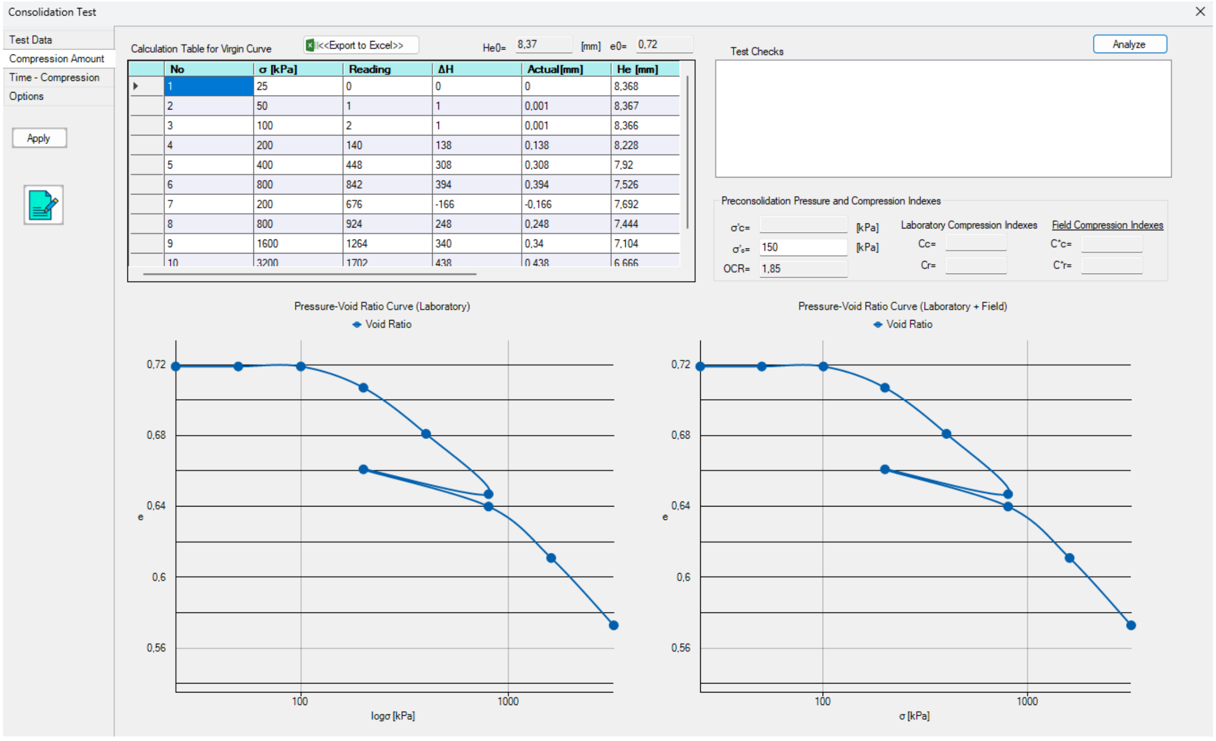


Figure 54. Compression Amount

On the plotted Pressure–Void Ratio (Laboratory) graph, the point with the minimum radius of curvature is visually identified and marked with the cursor. According to this point, the preconsolidation pressure (σ'_c), compression index (C_c), and recompression index (C_r) are calculated on the graph using the **Casagrande method**. The calculated values are displayed in the **Preconsolidation Pressure and Compression Indices** section.

When the effective stress at the sampling depth is entered into the same section, the Pressure–Void Ratio (Laboratory + Field) curve is drawn, and the overconsolidation ratio (OCR) and field compression indices (C_c*, C_r*) are calculated (Figure 55).

By clicking the **Analyze** button, the manual calculations on the graph are removed and the analysis is performed.

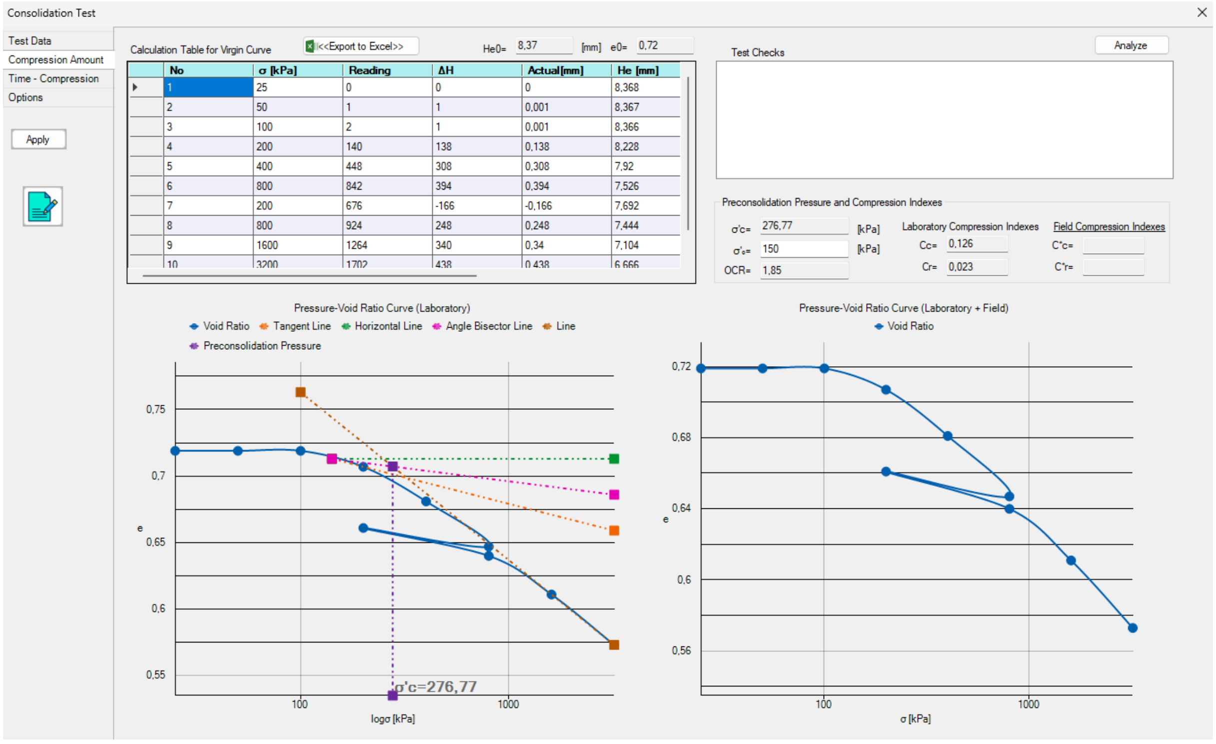


Figure 55. Calculations on Pressure–Void Ratio Curves

In the **Compression Time** tab, for the stresses where quick readings are taken, the times corresponding to 50% and 90% consolidation are calculated (Figure 56).

In the **Load Set** section, the stress values where quick readings are taken are displayed.

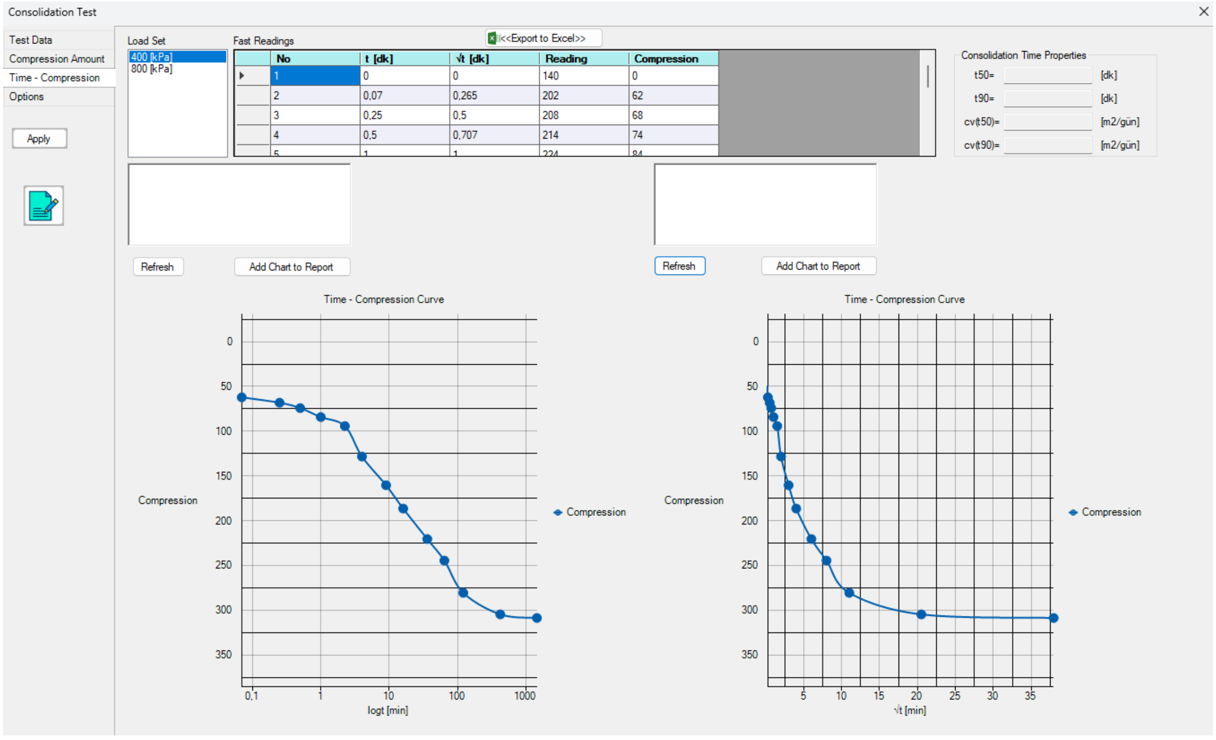


Figure 56. Compression Time

On the Compression–logt graph, the time at which 50% consolidation is completed (t_{50}) is calculated using the logarithmic method (Figure 57).

On the Compression–√t graph, the time at which 90% consolidation is completed (t_{90}) is determined using the square root method (Figure 57).

In the **Consolidation Time Properties** section, the consolidation coefficients calculated from the graphs are displayed.

With the **Refresh** button, calculations on the graphs are reset.

With the **Add Graph to Report** button, the generated graphs are added to the test report.

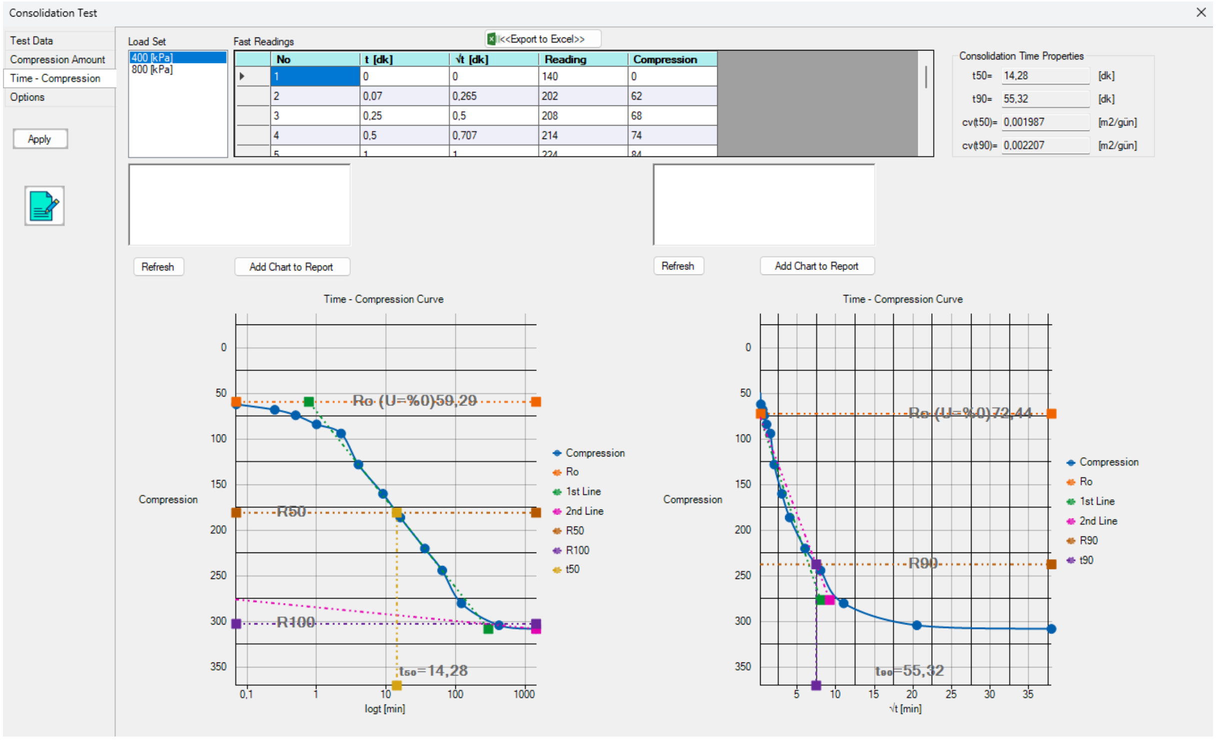


Figure 57. Consolidation Coefficients

In the **Settings** tab, the settings related to the consolidation test are configured (Figure 58).

- In the **Standard** section, the consolidation test standard to be used is selected.
- In the **Load Options** section, the load set suitable for the readings in the **Test Data** tab is determined. One of the existing load sets can be selected, or a new user-defined load set can be created.
- In the **Time Options** section, the quick reading load set in the **Test Data** tab is defined. Similarly, existing sets can be used, or a new definition can be made.

In the **Determination of Preconsolidation Pressure by the Casagrande Method** section, the selection is made on the Pressure–Void Ratio (Laboratory) curve drawn in the **Compression Amount** tab according to the point with the minimum radius of curvature. This selection can be made exactly at the point, just after it, or between two points.

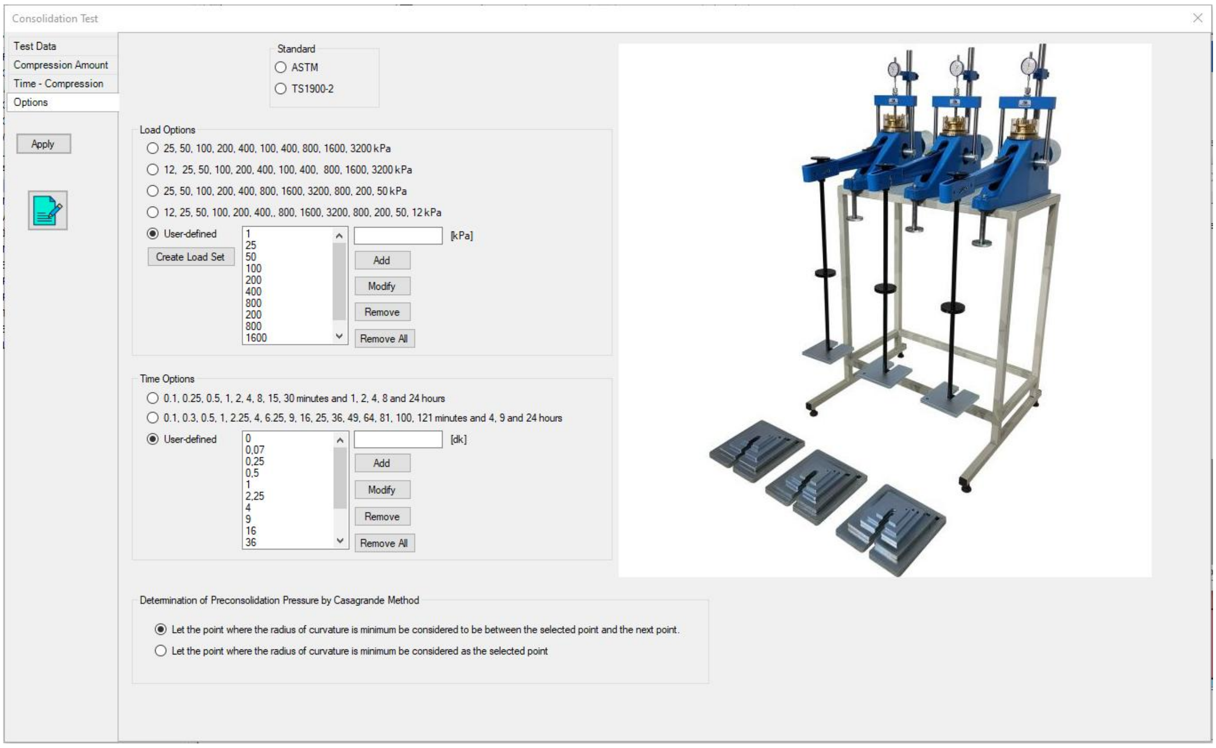


Figure 58. Settings

8. ANALYSES

The program performs **foundation analyses**, **excavation support structure analyses**, **slope/stability analyses**, and **design calculations**.

8.1. Foundation Analyses

By clicking the **Foundation Analysis** button in the **Analysis** menu, the **Foundation Analysis** window opens. In this window, under the **Settlement Analysis** tab, the foundation settlements are calculated at the defined settlement points by considering the stress increments caused by all foundations (Figure 59).

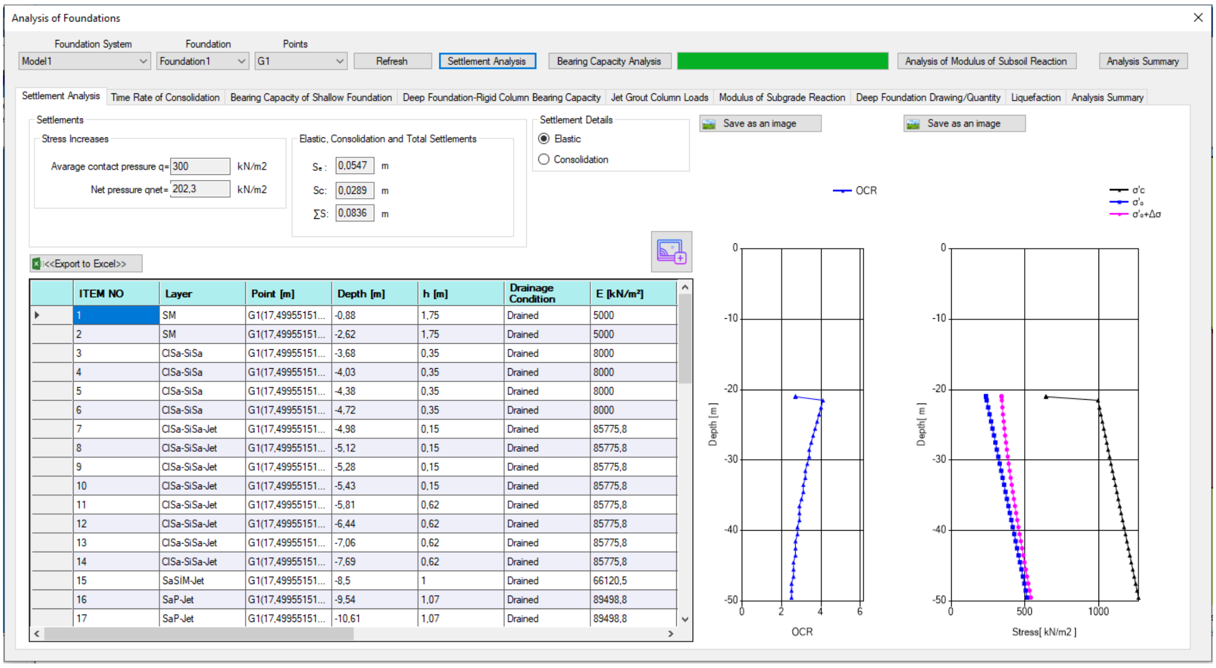


Figure 59. Foundation Analysis (Settlement Results)

In the **Stress Increments** section, the values of **Average Bearing Pressure (q)** and **Net Stress (q_{net})** calculated in the **Loads and Properties** tab of the **Foundation Properties** window are displayed.

In the **Elastic, Consolidation, and Total Settlements** section, the calculated settlement values are shown:

- S_e: Elastic settlement (undrained immediate or drained final settlement)
- S_c: Consolidation Settlement
- ΣS: Total Settlement

With the **Export to Excel** button, settlement analysis results are exported as an Excel file.

In the **Settlement Details** section, it is selected whether the calculation table will be based on elastic or consolidation settlement.

If a consolidation settlement is calculated, the graphs of OCR and stress variation with depth are plotted. These graphs can be saved to the computer with the **Save as Image** button.

 : This button adds the settlement calculation tables and graphs to the geotechnical reports.

In the **Settlement Time** tab, the consolidation settlement–time graphs of the selected point are plotted. For the entered time value, both consolidation settlement and total settlement are calculated.

Definitions:

- t: Consolidation time
- $S_{c(t)}$: Consolidation settlement at time *t*
- $\Sigma S_{(t)}$: Total settlement at time *t*
- $S_{c(t)}$*: Consolidation settlement at time *t* with Terzaghi correction applied
- $\Sigma S_{(t)}$*: Total settlement at time *t* with Terzaghi correction applied

In the **Chart Options** section, the following options are available to customize the appearance of the settlement–time graphs:

- Show Terzaghi Corrections on the Graph
- Show Points
- Show Settlement Values on the Graph

By selecting the relevant options, values can be added to or removed from the graph. The generated graphs can be saved to the computer with the **Save as Image** button.


: The settlement calculation tables and the settlement–time graph can be added to the geotechnical reports.

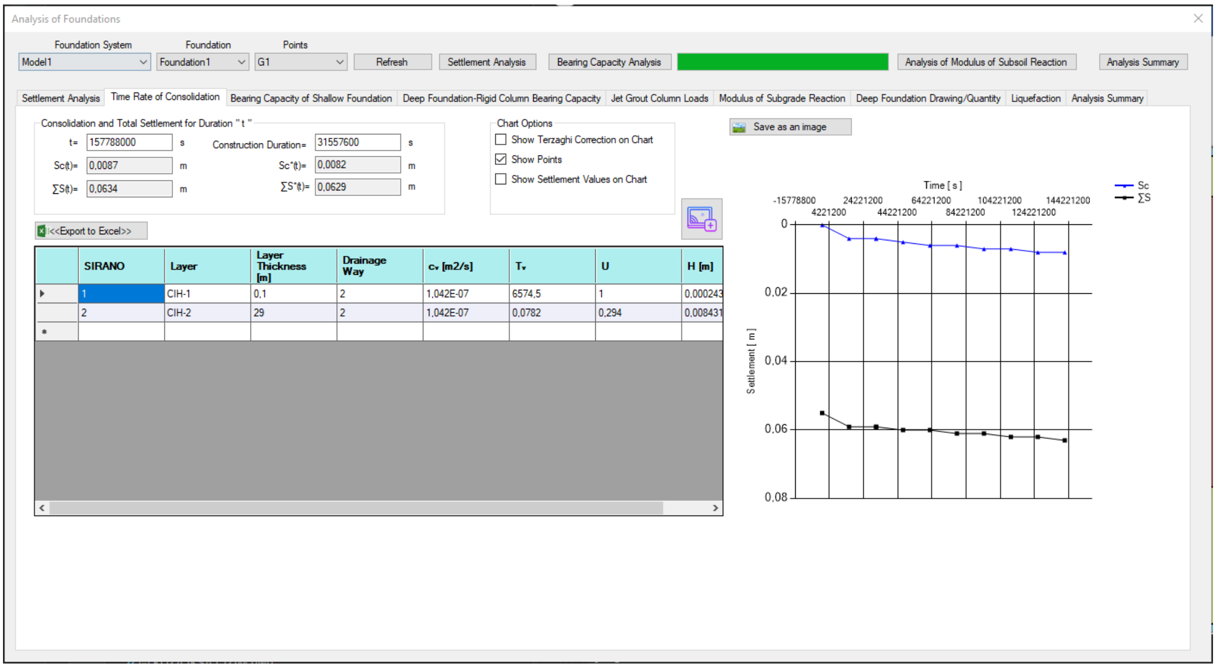


Figure 60. Foundation Analysis (Settlement Time)

In the **Shallow Foundation Bearing Capacity** tab, the **characteristic bearing capacity resistance (q_k)** and the **design resistance (q_t)** are calculated for the selected foundation (Figure 61).

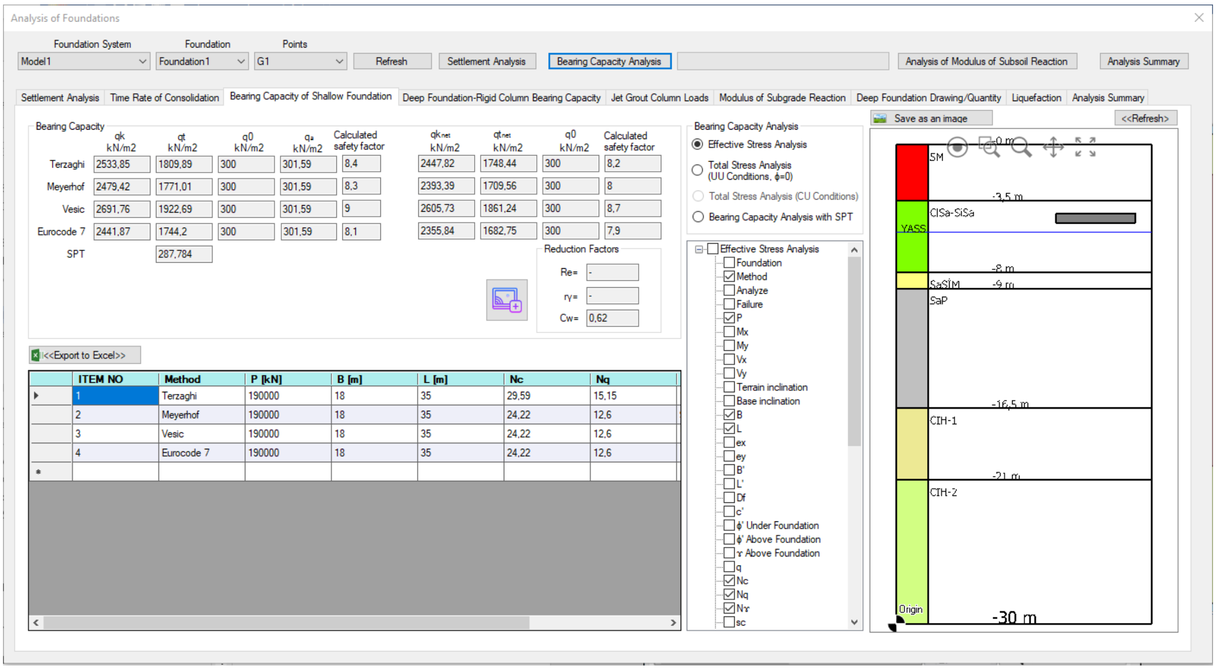


Figure 61. Foundation Analysis (Shallow Foundation Bearing Capacity)

In the **Bearing Capacity Analysis** section, the analysis conditions are selected. The data to be tabulated is marked from the data tree, and with the repeated analysis, these data are displayed in tabular form within the window. The generated table can be saved to the computer with the **Export to Excel** button and opened in Excel.

The main values calculated in the **Bearing Capacity** section are:

- q_k: Characteristic bearing capacity
- q_t: Design bearing capacity
- q_0: Foundation pressure
- q_a: Effective foundation pressure
- q_{knet}: Characteristic net bearing capacity
- q_{tnet}: Net design bearing capacity

The analysis results are automatically displayed in the related data fields.

"With the **Refresh** button, the foundation–soil cross-section is generated. The generated cross-section can be saved to the computer with the **Save as Image** button.

: The bearing capacity calculation tables and the foundation–soil cross-section can be automatically added to the geotechnical reports.

If the selected foundation is a **deep foundation (rigid column)**, the **Deep Foundation–Soil Improvement** tab allows separate bearing capacity calculations for single piles and pile groups (Figure 62).

In the **Single Rigid Column Bearing Capacity** section, the main calculated values are:

- Q_{ks}: Characteristic shaft resistance
- Q_{ku}: Characteristic tip resistance
- Q_{ktv}: Characteristic total bearing capacity
- γ_{Rs}: Partial safety factor for shaft friction resistance
- γ_{Ru}: Partial safety factor for pile tip resistance

- Q_{tv}: Vertical design resistance
- P_{tv}: Vertical design load
- G.S.: Calculated factor of safety

In the **Group Bearing Capacity** section:

- E_g: Group efficiency
- $Q_{tv,gr}$: Vertical design resistance of the group
- $P_{tv,gr}$: Vertical design load of the group
- G.S.: Calculated factor of safety for the group

In the **Analysis Details** section, the options to be included in the analysis are selected. Then, by clicking the **Bearing Capacity Analysis** button, the analysis is initiated. The results are displayed in a table and can be exported with the **Export to Excel** button.

With the **Refresh** button, the foundation–soil cross-section is generated. The generated structure–soil cross-section can be saved to the computer with the **Save as Image** button.

 : The bearing capacity calculation tables and graphs can be included in the geotechnical reports.

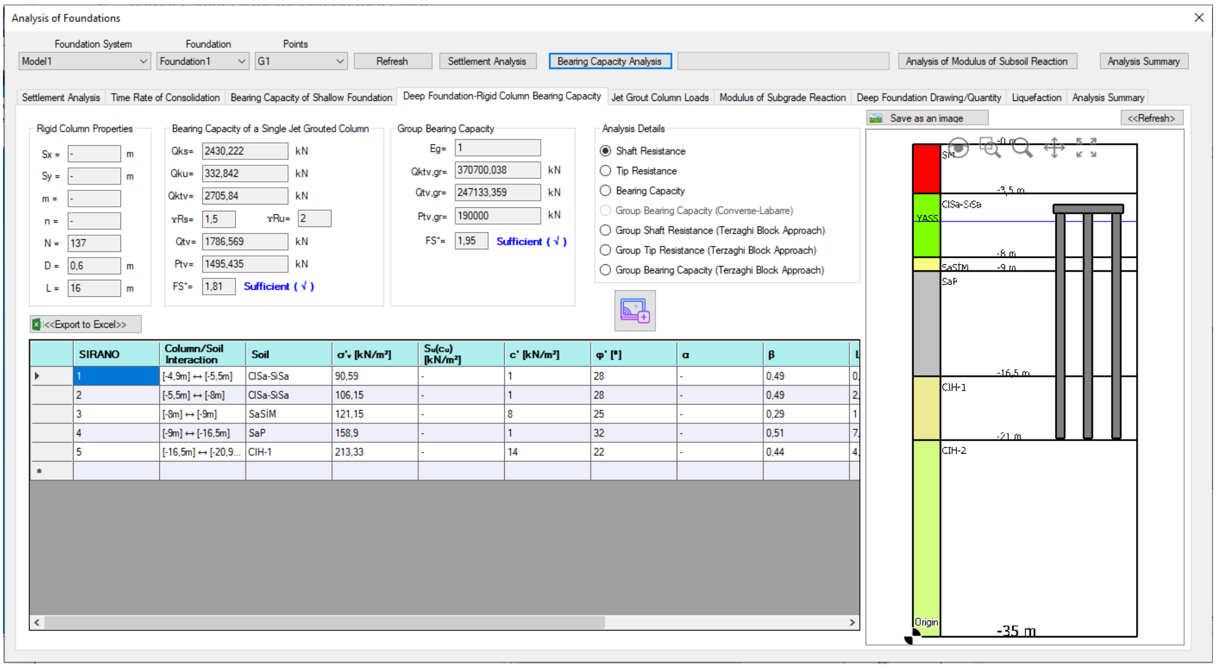


Figure 62. Foundation Analysis (Deep Foundation Bearing Capacity)

In the **Column Loads** tab, the load transfer to the pile system is calculated. On the table, the load distribution for each rigid column is listed, while in the plan view, the pile layout and orientations are graphically displayed (Figure 63).

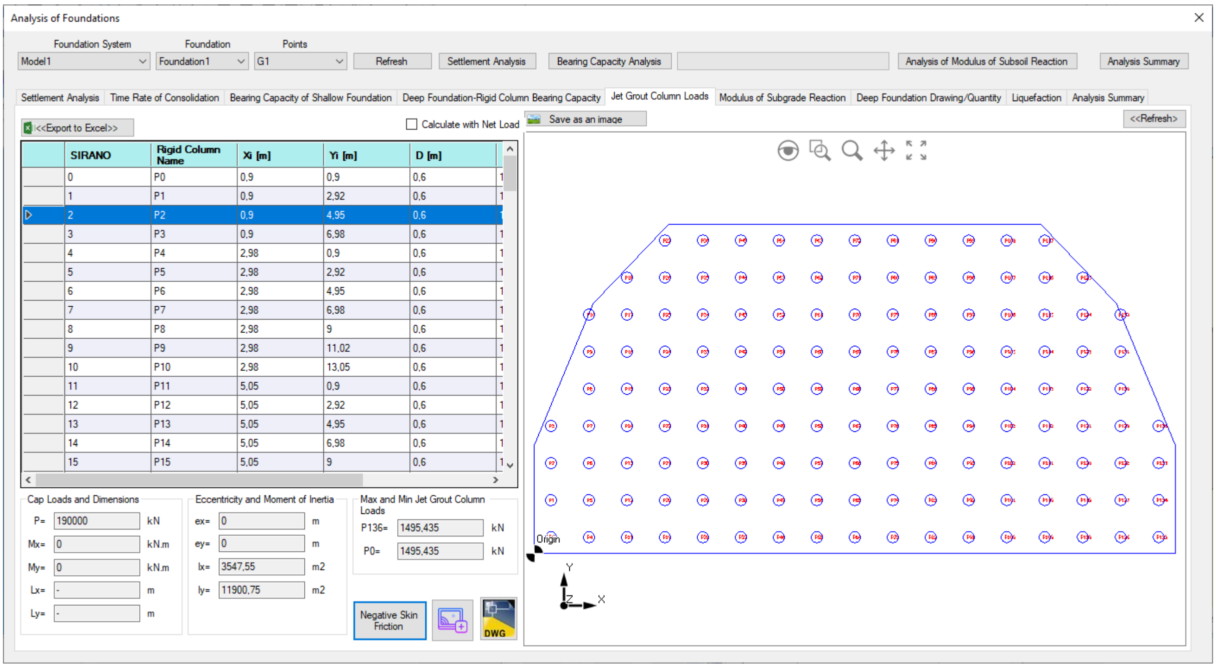


Figure 63. Load Transfer to Piles

When the **Calculate with Net Load** option is enabled, the pile loads are calculated based on net load values. With the **Refresh** button, the foundation–soil cross-section is generated. The generated structure–soil cross-section can be saved to the computer with the **Save as Image** button.

The generated table can be saved to the computer with the **Export to Excel** button and opened in Excel.

Cap Loads and Dimensions Section:

- P: Vertical load
- M_x, M_y: Bending moments in the X and Y directions
- L_x, L_y: Foundation dimensions

Eccentricities and Moments of Inertia Section:

- e_x: Eccentricity in the X direction
- e_y: Eccentricity in the Y direction
- I_x: Moment of inertia in the X direction
- I_y: Moment of inertia in the Y direction

In the **Max–Min Column Loads** section, the numbers and load values of the rigid columns carrying the maximum and minimum vertical loads are displayed.

 : The bearing capacity calculation tables and graphs can be added to the geotechnical reports.

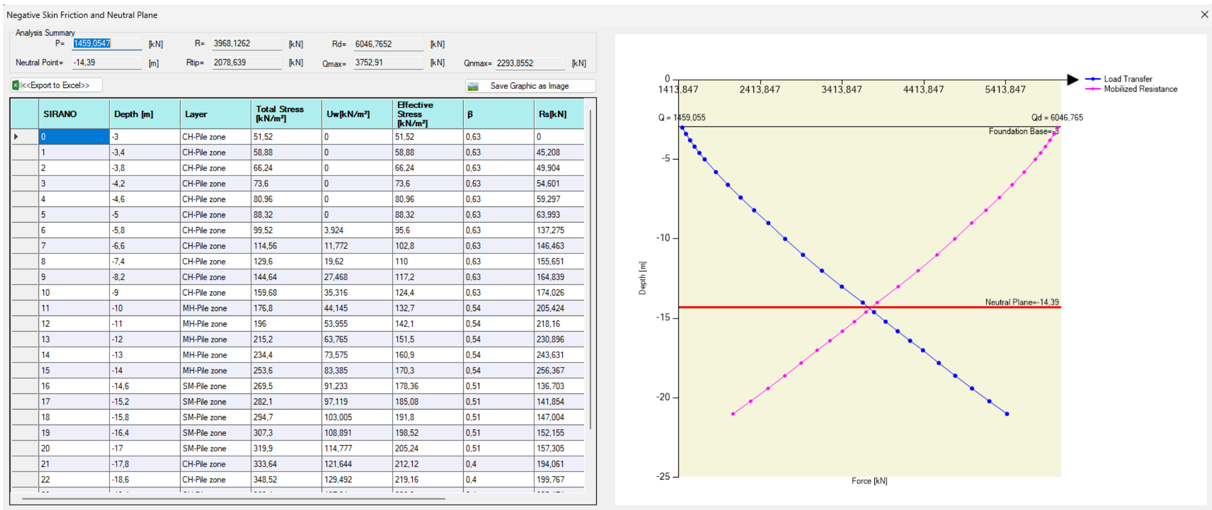
: The rigid column layout can be saved to the computer in **.dwg** format.

Using the **Negative Skin Friction** option, the program generates load transfer curves and determines the location of the neutral plane for a selected pile or improvement column from the list. As a result of this analysis, $\mathbf{Q_{max}}$ (load at the neutral plane) and $\mathbf{P_n}$ values are obtained (Figure 64).

Values Displayed in the Analysis Summary Section:

- P: Total load
- R: Shaft resistance
- $R_{uç}$: Tip resistance
- R_d: Total resistance
- Q_{max}: Load at the neutral plane

The generated table can be saved to the computer with the **Export to Excel** button and opened in Excel. The load transfer curves and neutral plane graph can be saved to the computer with the **Save as Image** button.



SIRANO	Depth [m]	Layer	Total Stress [kN/m²]	Uw[kN/m²]	Effective Stress [kN/m²]	β	Rs[kN]
0	-3	CH-Pile zone	51,52	0	51,52	0,63	0
1	-3,4	CH-Pile zone	58,88	0	58,88	0,63	45,208
2	-3,8	CH-Pile zone	66,24	0	66,24	0,63	49,904
3	-4,2	CH-Pile zone	73,6	0	73,6	0,63	54,601
4	-4,6	CH-Pile zone	80,96	0	80,96	0,63	59,297
5	-5	CH-Pile zone	88,32	0	88,32	0,63	63,993
6	-5,8	CH-Pile zone	99,52	3,924	95,6	0,63	137,275
7	-6,6	CH-Pile zone	114,56	11,772	102,8	0,63	146,463
8	-7,4	CH-Pile zone	129,6	19,62	110	0,63	155,651
9	-8,2	CH-Pile zone	144,64	27,468	117,2	0,63	164,839
10	-9	CH-Pile zone	159,68	35,316	124,4	0,63	174,026
11	-10	MH-Pile zone	176,8	44,145	132,7	0,54	205,424
12	-11	MH-Pile zone	196	53,955	142,1	0,54	218,16
13	-12	MH-Pile zone	215,2	63,765	151,5	0,54	230,896
14	-13	MH-Pile zone	234,4	73,575	160,9	0,54	243,631
15	-14	MH-Pile zone	253,6	83,385	170,3	0,54	256,367
16	-14,6	SM-Pile zone	269,5	91,233	178,36	0,51	136,703
17	-15,2	SM-Pile zone	282,1	97,119	185,08	0,51	141,854
18	-15,8	SM-Pile zone	294,7	103,005	191,8	0,51	147,004
19	-16,4	SM-Pile zone	307,3	108,891	198,52	0,51	152,155
20	-17	SM-Pile zone	319,9	114,777	205,24	0,51	157,305
21	-17,8	CH-Pile zone	333,64	121,644	212,12	0,4	194,061
22	-18,6	CH-Pile zone	348,52	129,492	219,16	0,4	199,767

Figure 64. Determination of the Neutral Plane in a Rigid Column

In the **Subgrade Modulus** tab, the vertical subgrade modulus of a shallow foundation is calculated for the selected foundation model. If the foundation is supported by rigid columns, the subgrade modulus representing the shaft resistance, the subgrade modulus representing the tip resistance, and the horizontal subgrade moduli are also calculated (Figure 65).

Shallow Foundation Subgrade Modulus Section:

- q: Contact pressure
- k_v: Vertical subgrade modulus

Deep Foundation Subgrade Moduli Section:

- k_{vs}: Subgrade modulus representing the shaft resistance
- k_{vp}: Subgrade modulus representing the tip resistance

In the **Shallow Foundation Calculation Method** section, the method used for the calculations is specified.

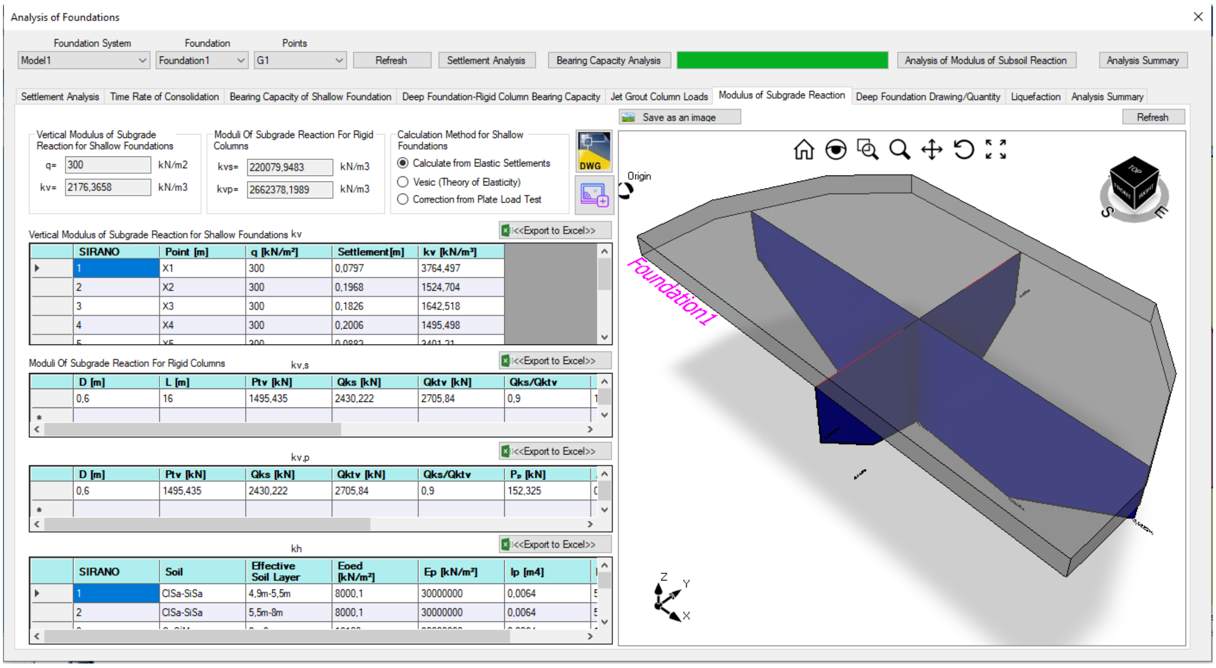


Figure 65. Subgrade Modulus Analysis Results

: Subgrade modulus calculation tables and graphs can be added to the geotechnical reports.

: The drawing generated for the subgrade modulus calculation can be saved to the computer in **.dwg** format.

With the **Refresh** button, a drawing showing the settlements according to the subgrade modulus calculation is created. The generated drawing can be saved with the **Save as Image** button.

The generated tables can be exported with the **Export to Excel** button and opened in Excel.

Pile Reinforcement and Quantity Calculations

In the **Pile Reinforcement** tab, reinforcement and quantity calculations are performed for the piles defined in the system (Figure 66).

In the **Quantity Type** section, the element for which the quantity will be calculated is selected. Then, by clicking the **Quantity** button, the related quantity table is generated.

The generated tables can also be saved to the computer with the **Export to Excel** button and opened in Excel.

 : Quantity tables and graphs can be included in the geotechnical reports.

In the **Material** section, the concrete and steel grade of the pile element is defined.

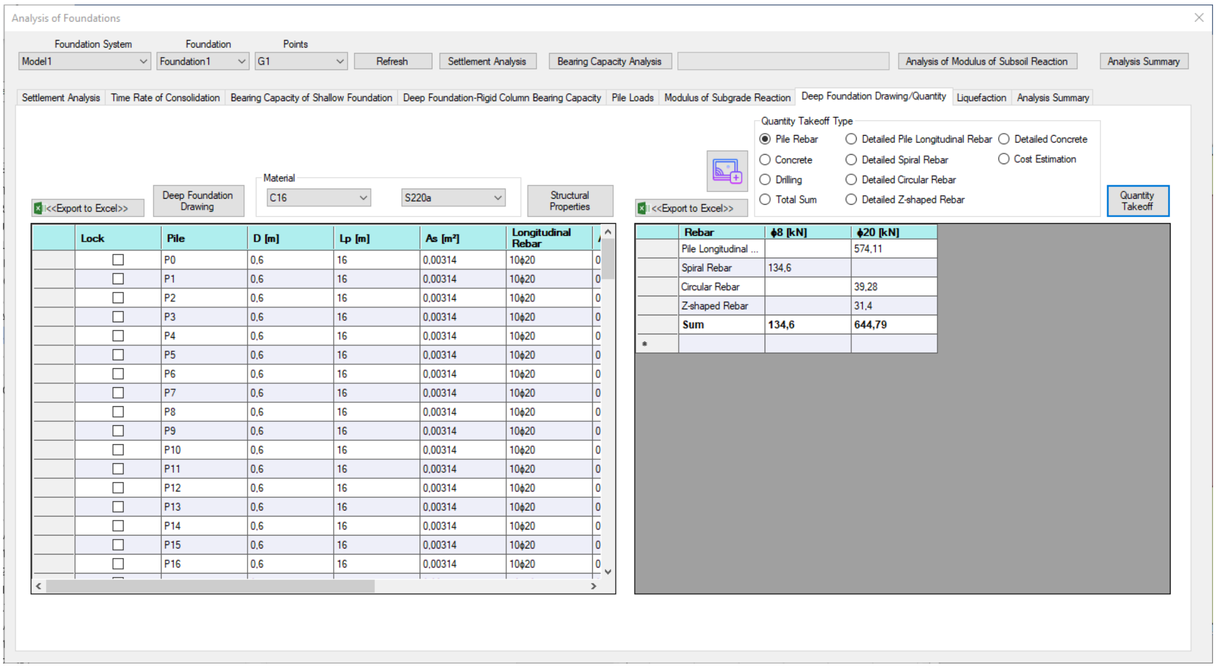


Figure 66. Pile Reinforcement Analysis Results

With the **Reinforcement** button, the rigid columns in the system are reinforced in accordance with **TS500 – Requirements for Design and Construction of Reinforced Concrete Structures**, ensuring that the minimum reinforcement requirements are satisfied. The reinforcement table prepared can be saved to the computer in **.xlsx** format with the **Export to Excel** button.

With the **Pile Foundation Drawing** button, the drawing of the rigid column foundation system is generated.

Liquefaction Analyses

In the **Liquefaction** tab, liquefaction analyses are carried out according to the **SPT profile** of the selected borehole, using the **TBDY Annex 16B** method. The calculation results are prepared as tables and graphs (Figure 67).
Note: Calculations are performed according to the SPT data selected by the user.

Seismic Properties Section:

- M_v: Earthquake magnitude
- SDS: Short-period design spectral acceleration coefficient

In the **Borehole** section, the total depth and **G.W.T. (groundwater table)** information are entered.

Surface Settlement after Liquefaction

In this section, the amount of surface settlement caused by liquefaction is displayed.

In the **SPT Profile and Soil Properties** section, a table of the data forming the basis of the liquefaction analysis is presented.

In the **Liquefaction Evaluation Analysis** section, by clicking the **Liquefaction Triggering Analysis** button, an analysis table showing the results of liquefaction is generated.

If the foundation type is defined as a deep foundation system, or if a soil improvement method is applied beneath a shallow foundation, the **Rigid Column Group Effect** is taken into account when evaluating the influence on liquefaction.

 : This button generates a local report for the liquefaction analysis.

The generated tables can be exported with the **Export to Excel** button and opened in Excel. The generated graphs can be saved to the computer with the **Save as Image** button.

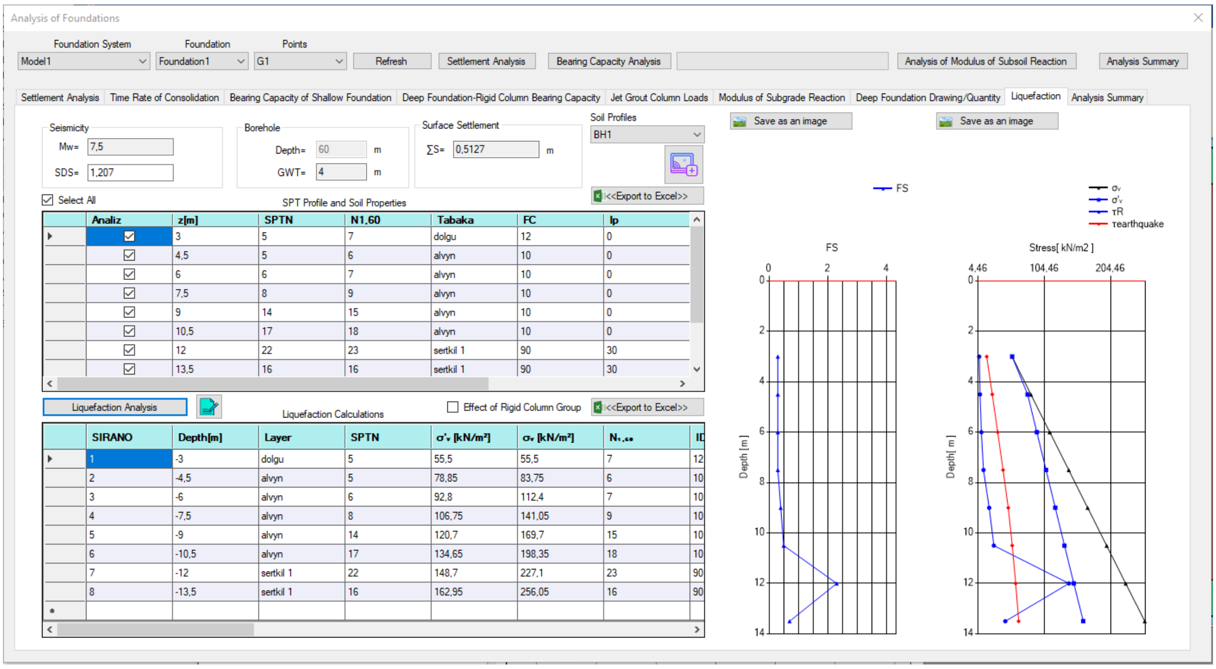


Figure 67. Liquefaction Analysis Results

In the **Analysis Summary** tab, the bearing capacity, settlement, and subgrade modulus analysis results of the created foundation system are generated as summary tables according to the method and stress condition defined by the user (Figure 68).

Within the **Analysis** section, the analysis type, method, and stress condition are selected. The **Foundations** section is used to specify the foundation to be summarized. Finally, the **Settlement Points** section defines the point where the settlement calculation will be performed.

After these definitions are completed, by clicking the **Analysis Summary** button, the analysis summary table is generated.

The generated tables can be exported with the **Export to Excel** button and opened in Excel.

 : The analysis summary tables can be included in the geotechnical reports.

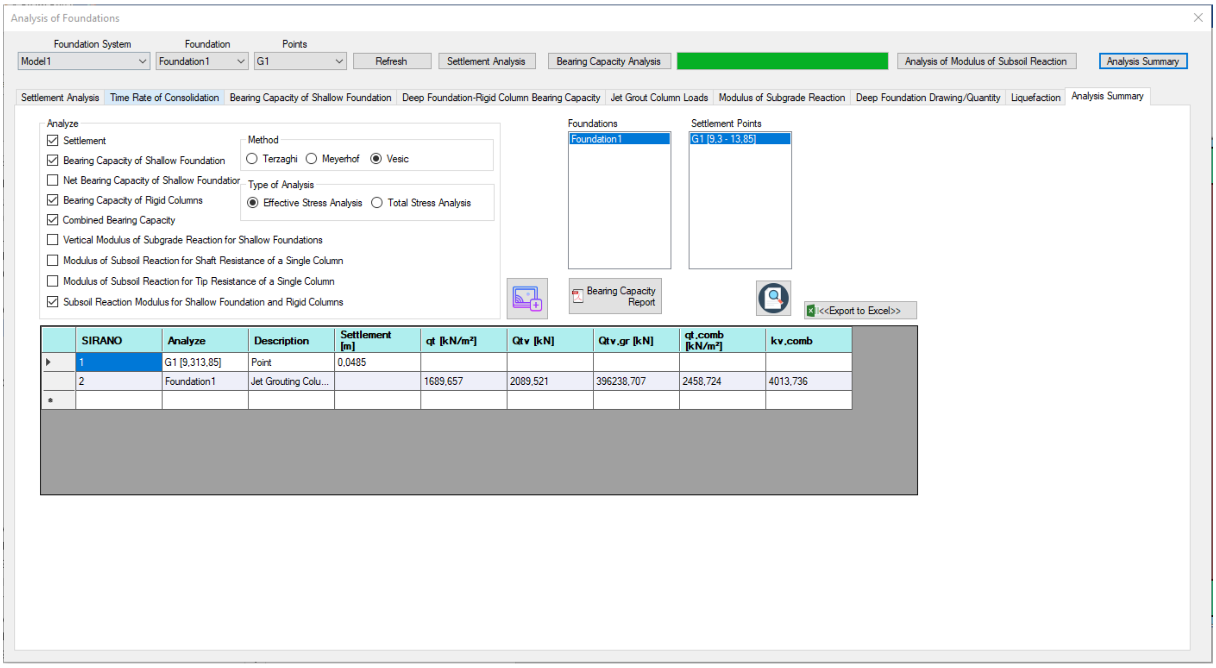


Figure 68. Analysis Summary

8.2. Excavation Support Wall Analyses

By clicking the **Analysis** button in the **Section** window, the **Excavation Support Wall Analysis** window opens. In this window, the excavation support wall analysis and related checks are performed (Figure 69).

Construction Stages Section:

 : **Adds** a new construction stage to the analysis list.

 : **Deletes** the selected construction stage from the list.

Construction Stage List Creation Section:

 : Automatic construction stage adding tool

h_i: Additional excavation depth for anchors and support levels

h_s: Excavation depth increment at the final stage

Other Sections:

- In the **Wall Type (Plate)** section, the **Wall Properties** button is used to define the dimensions and reinforcement of the section to be analyzed.

- In the Terrain section, the ground condition behind the wall is selected.
- In the **Excavation** section, the excavation depth is defined.
- In the **G.W.T. (Groundwater Table)** section, the groundwater level behind the wall is entered.

Anchors and Supports

- In the **Anchors** section, the anchor elements present in the analyzed section are listed. By double-clicking a selected anchor, the window for editing the anchor properties opens.
- Similarly, in the **Supports** section, the support elements are listed. Double-clicking a support element opens the window for editing its properties.

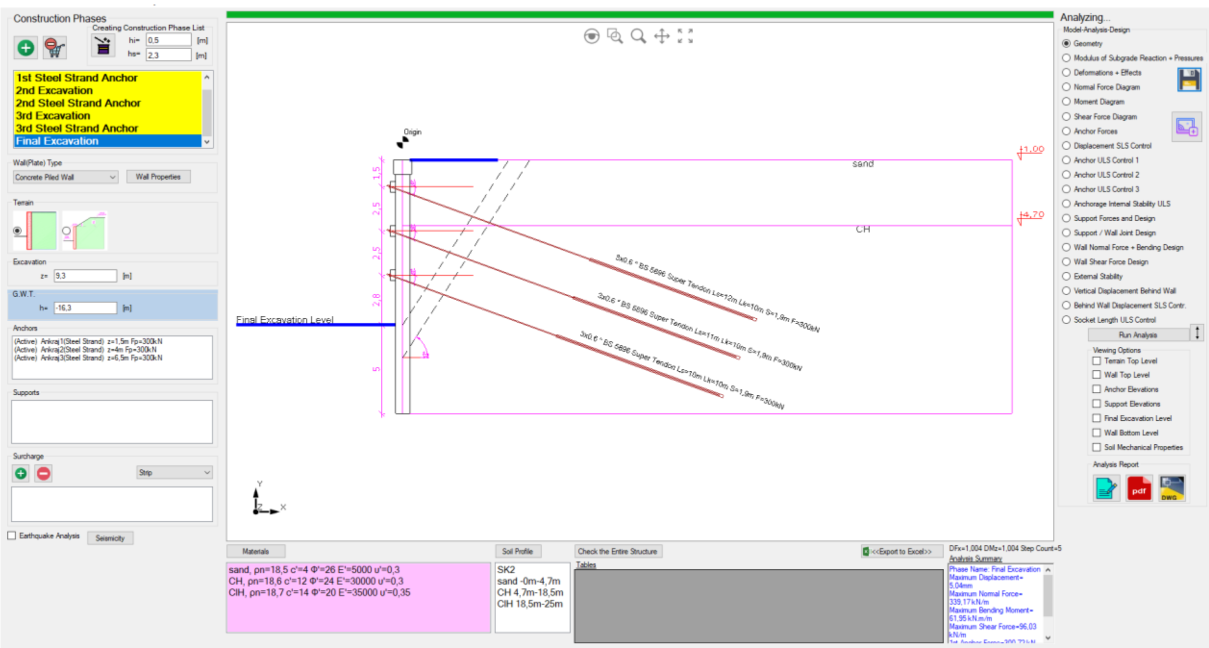


Figure 69. Excavation Support Wall Analysis Window

In the **Surcharge** section, the surcharge loads behind the wall are defined. From the dropdown list, the surcharge type is selected based on point, line, strip, or area data.

 : **Adds** a new surcharge load to the list (Figure 70).

 : **Deletes** the selected surcharge load from the list.

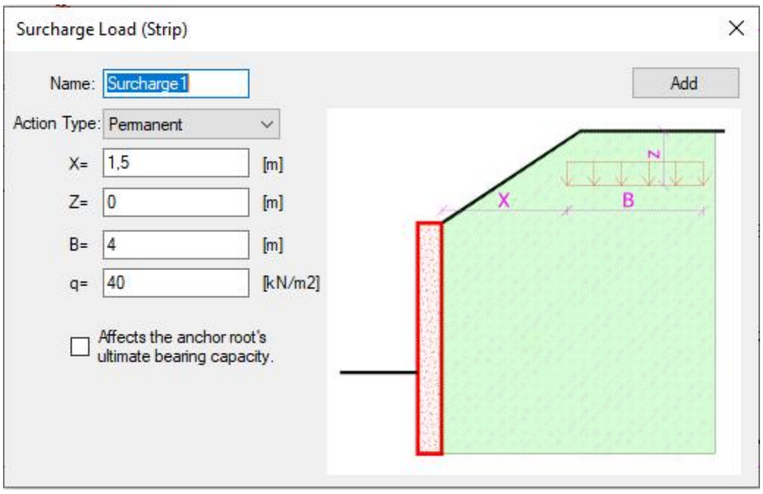


Figure 70. Surcharge Load

Name: Name assigned to the surcharge load

Action Type: Selected as permanent or variable depending on the load condition

X: Horizontal distance of the load from the wall

Z: Vertical distance of the load from the ground surface

B: Load width

q: Load value

When the **Affects the ultimate bearing capacity of the anchor root** option is enabled, the added surcharge load influences the ultimate bearing capacity of the anchor root.

The surcharge load is added to the system with the **Add** button.

Seismic Analysis

When the **Seismic Analysis** option is checked, the analysis is performed for the construction stage under earthquake conditions.

By clicking the **Seismicity** button, the window for entering the required parameters opens (Figure 71).

Earthquake Parameters

- **SDS**: Short-period design spectral acceleration coefficient

- **r**: Horizontal equivalent acceleration reduction factor based on support type
- **k_h**: Horizontal equivalent acceleration coefficient
- **k_v**: Vertical equivalent acceleration coefficient

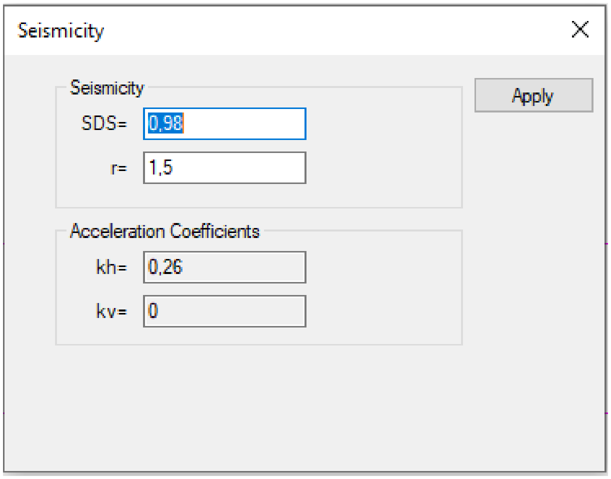


Figure 71. Seismicity

Auxiliary Windows and General Tools

- The **Materials** button opens the *Add Geotechnical Material* window, where the materials used in the analysis can be modified.
- The **Soil Profile** button opens the *Add Borehole* window, allowing the soil profile properties to be edited.
- The **Check Entire Structure** button verifies possible deficiencies and compliance with design code requirements in the completed retaining system.
- In the **Tables** section, the calculation tables generated as a result of the analysis are displayed. These tables can be saved to the computer with the **Export to Excel** button and opened in Excel.
- In the **Analysis Summary** section, the maximum displacements, internal forces, and anchor/support forces corresponding to the selected construction stage are shown.

Graphical Analysis and Design Visualization

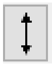
In the **Analysis (Model–Analysis–Design)** section, the following analysis results can be visualized graphically on the screen:

- **Geometry**: Displays the model geometry.
- **Yatak Katsayısı + Basınçlar**: Shows active, passive, and at-rest earth pressures.
- **Deformation & Effects**: Displays the deformation of the retaining section and the acting pressures on the system.
- **Axial Force Diagram**: Distribution of axial forces.
- **Moment Diagram**: Distribution of bending moments along the wall.
- **Shear Force Diagram**: Distribution of shear forces.
- **Anchor Forces**: Displays the forces acting on anchors.
- **Displacement SLS Check**: Serviceability limit state (SLS) displacement check.
- **Anchor ULS Check 1**: Bond strength check between the grouted body and soil interface.
- **Anchor ULS Check 2**: Tendon tensile failure check.
- **Anchor ULS Check 3**: Bond strength check between the tendon and grouted body interface.
- **Anchor Internal Stability (ULS)**: Displays internal stability blocks and results for all construction stages.
- **Support Forces and Design**: Displays support forces and the calculated design values.
- **Support / Wall Connection Design**: Displays the connection design results table.
- **Wall Axial Force + Bending Design**: Section and reinforcement verification.
- **Wall Shear Force Design**: Reinforcement verification based on shear force.

- **External Stability**: Transfers the model to the slope analysis screen for external stability checks.
- **Vertical Displacement Behind Wall**: Settlement graph behind the wall.
- **Vertical Displacement SLS Check Behind Wall**: Settlement control table.
- **Socket Length ULS Check**: Provides the minimum required socket length for the wall.

Other Functions

- The **Run Analysis** button starts the wall analysis.

 : **Calculates** the socket length.

- In the **Display Options** section, depending on the selected data fields, the ground surface level, wall top level, anchor and support levels, final excavation level, wall bottom level, and soil mechanical properties are displayed on the screen.

 : **Saves** the analysis model.

 : **Adds** images and tables to the geotechnical project design report.

8.3. Slope Stability Calculations

All calculation and analysis steps related to slope stability are explained in detail in **Section 6 – "Slope Stability"**.

9. REPORTS

In **SETAF2018**, three types of reports can be generated: **general**, **local**, and **geotechnical** reports. General reports include all data and analysis results presented in tables and graphs. Local reports display the performed analyses or design calculations together with all the equations used.

With the reporting tool available in the program, geotechnical reports can be created using predefined report templates.

Through this tool, the user can write custom text sections and include all data, analysis, and design tables within the geotechnical report to be generated. Images from both inside and outside the program can also be incorporated into these reports.

In **SETAF2018**, three types of reporting are available:

- **General Reports**: Present all data and analysis results accompanied by tables and graphs.
- **Local Reports**: Show the analyses or design calculations in detail, including all equations used.
- **Geotechnical Reports**: The program's reporting tool is used to create reports based on templates.

With the help of this reporting tool, the user can:

- Write their own explanatory text sections,
- Add all data, analysis, and design tables,
- Insert images obtained from within the program or from external sources.

9.1. Foundation General Report

By clicking the **Selectable Reports** button in the **Report** menu, the general report window for foundation analyses opens (Figure 72).

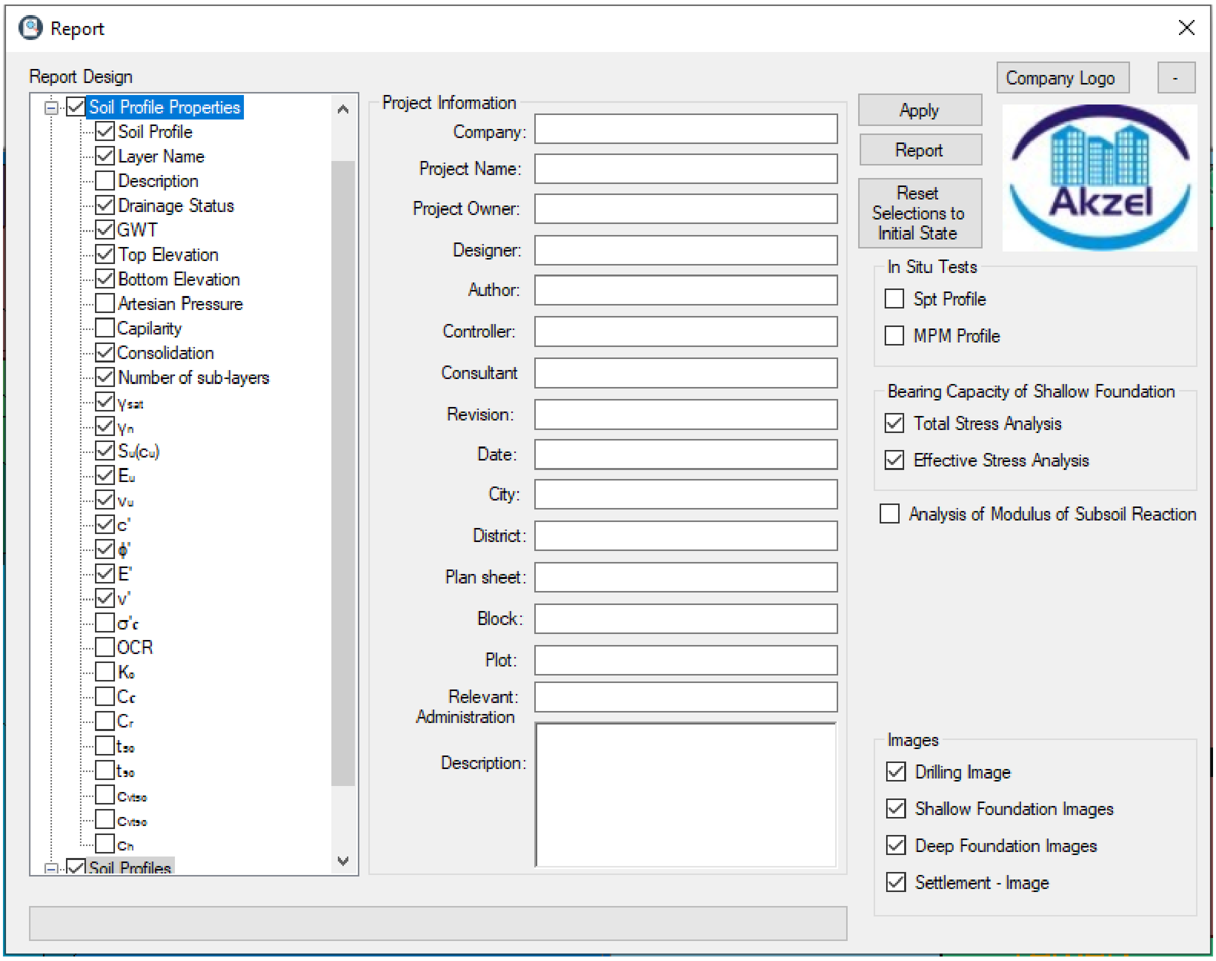


Figure 72. General Report Creation Window for Foundation Analyses

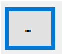
When generating a general report:

- The soil properties in the material data can be customized as desired.
- In the bearing capacity and settlement calculations, the user can select which foundations and settlement points in the model to include.

Report Content and Sections

- In the **Report Design** section, the foundations, soil profiles, and geotechnical material properties to be included in the report are defined.

- In the **Project Information** section, details such as project name, author, title deed, sheet, block, parcel, and address are entered. These details are automatically included in the report.
- In the **Field Tests** section, if SPT and MPM profiles are to be included in the report, the corresponding data fields are selected.
- In the **Shallow Foundation Bearing Capacity** section, the total stress, effective stress, and subgrade modulus calculation tables to be added to the report are selected.
- In the **Images** section, the visuals to be displayed in the report are defined.
- With the **Company Logo** button, the logo to be added to the report is selected in **.png** format.


: **Removes** the logo from the report when the corresponding removal button is clicked.

Reporting Operations

- **Apply** button: Saves all the information entered in the window.
- **Report** button: Generates the foundation general report.
- **Reset Selections** button: Restores the selections in the **Report Design** section to the program’s default settings.

9.2. DSM Mixture Calculations Local Report

In projects where soil improvement is performed using the **DSM (Deep Soil Mixing)** method, the amounts of water and cement in the injection mixture can be prepared as a **local report**.

To access the report, follow these steps:

1. Open the **Foundation Analysis** window.
2. Select the **Deep Foundation–Soil Improvement Bearing Capacity** tab.
3. Click the **Bearing Capacity Analysis** button to complete the analysis.

4. In the same tab, click the **DSM Material Report** button to generate the local report for DSM mixture calculations.

9.3. Excavation Support Structure General Report

It is possible to generate a report on excavation support systems for each **construction stage**. The user has the option of selecting which stages will be included in the report (Figure 73).

Reporting Settings

- In the **Images** section, the visuals to be added to the excavation support general report are selected. The corresponding data fields are checked to include the desired images.

 : **Updates** the report with newly added images or construction stage calculations.

- In the **Construction Stages** section, the stages to be included in the report are selected by checking the relevant options.

The prepared general report can be exported in **Excel, Word, or PDF** format and saved to the computer.

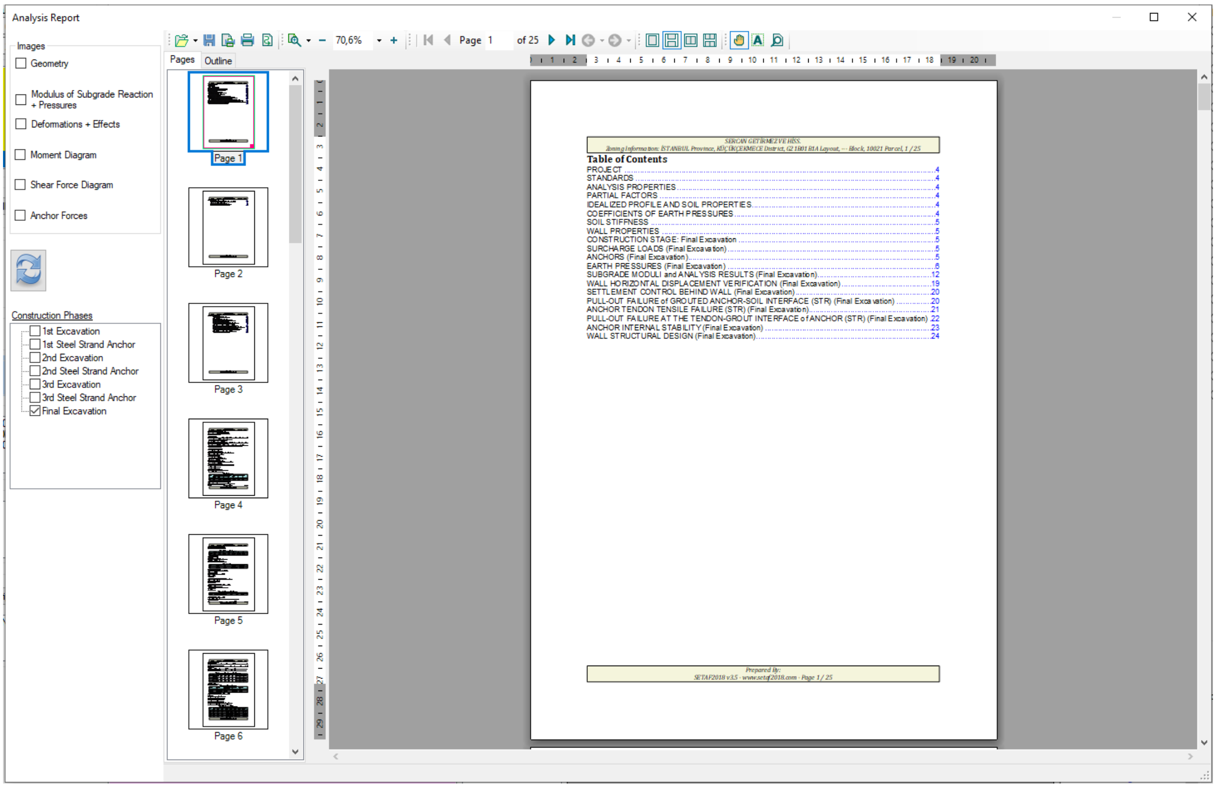


Figure 73. Excavation Support Structure General Report

9.4. Liquefaction Analysis Local Report

In the **Liquefaction** tab of the **Foundation Analysis** window, liquefaction analyses are performed using the **SPT profile** of the selected borehole according to the **TBDY Annex 16B** method (Figure 67).

Calculation Conditions

- If the foundation type is designed as a deep foundation,
- Or if a soil improvement method is defined beneath a shallow foundation,

→ In these cases, the **Rigid Column Group Effect** is taken into account in the liquefaction analysis.

The liquefaction calculations obtained from the analysis are prepared as a **local report**.

 : Generates the liquefaction report.

9.5. Slope Analysis Local Report

Through the **Slope Model Window** shown in Figure 49:

- The analysis method,
- The slip surface,
- And the analysis type

are selected, and the slope analysis is completed.

By clicking the **Analysis Report** button, a **local report** for the slope analysis based on the created analysis model is generated.

 : The local report for the slope analysis can be added to the geotechnical report.

9.6. Anchor and Support Design Local Report

In the **Section Analysis** window, click the **Local Design Reports** button located in the **Analysis Report** section to open the **Local Design Reports** window (Figure 74).

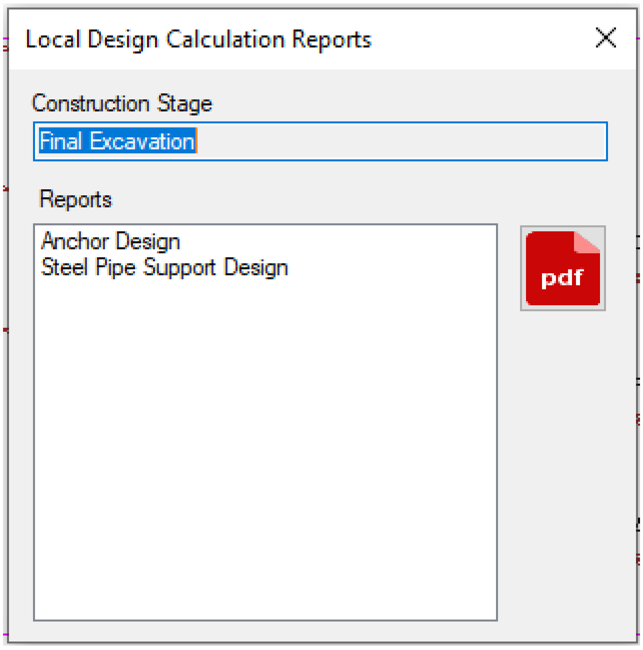


Figure 74. Local Design Calculation Reports

Reporting Steps

- In the **Construction Stage** section, select the stage for which the local report will be generated.
- In the **Reports** section, select the type of element (anchor or steel pipe support) to be reported for the chosen stage.
- Click the **PDF** button on the right side to generate the local report for the selected anchor or support element.

9.7. Geotechnical Reports

By clicking the **Geotechnical Reports** button in the **Report** menu, the geotechnical report window opens (Figure 75).

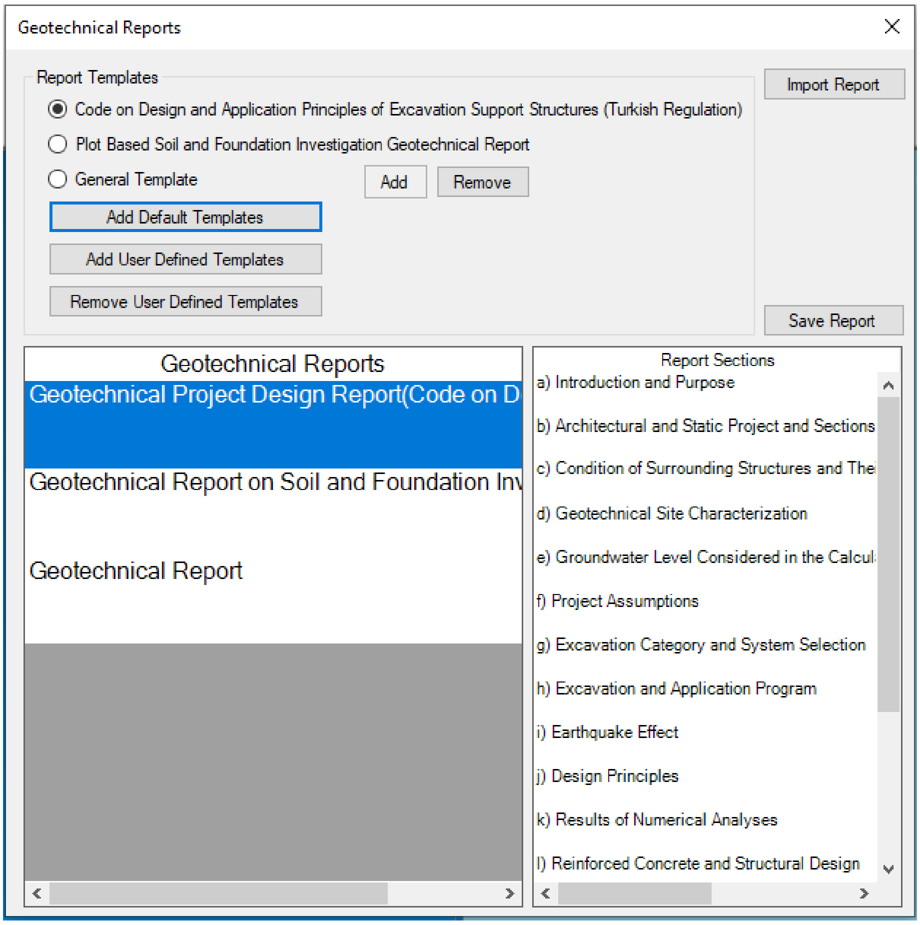


Figure 75. Geotechnical Reports

When right-clicking any report in the **Geotechnical Reports** section, the report name can be changed, or the report can be saved as a template.

Report Templates and Adding Operations

- In the **Report Templates** section, three predefined report types are available.
- To add a report, click the **Add** button for the selected template, and the corresponding report will be added to the **Geotechnical Reports** section.
- To remove an existing report, select it and click the **Remove** button.
- The **Add Default Templates** button adds sample reports for **DSM**, **retaining systems**, **jet grouting**, **shallow foundations**, and **slope analyses**.
- The **Add User-Defined Templates** button allows adding previously created custom templates.
- The **Remove User-Defined Templates** button deletes these templates from the program memory.
- The **Save Report** button saves the prepared report as a file with the **.srp** extension.

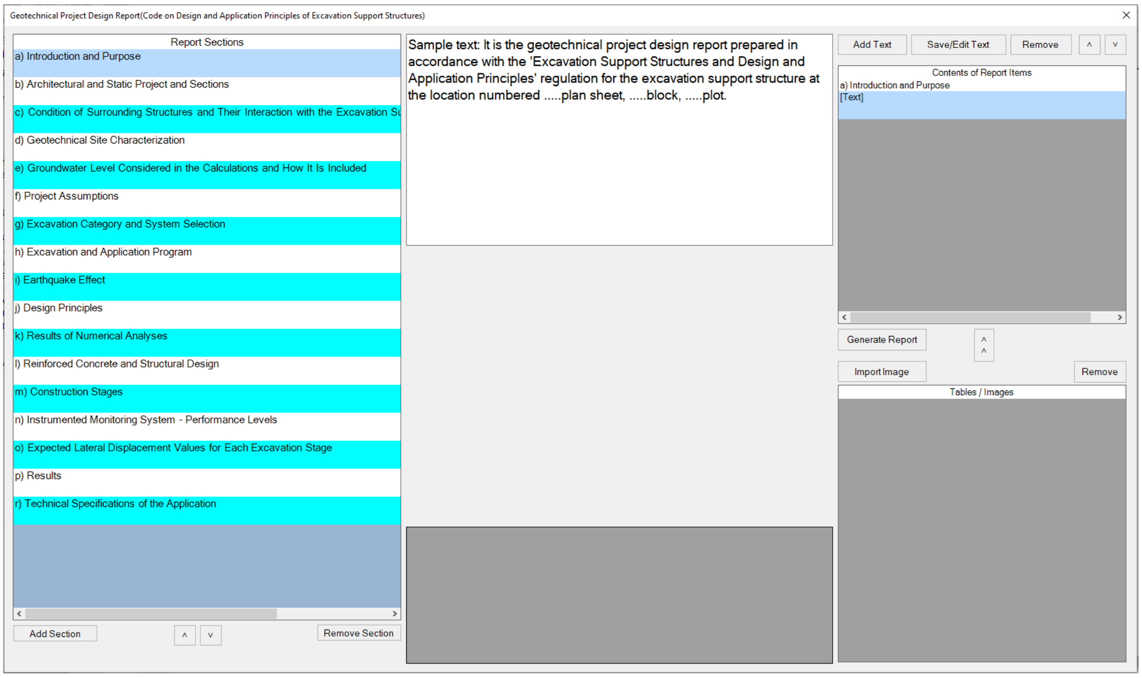


Figure 76. Geotechnical Report Editing Window

Report Editing Operations (Figure 76)

- By double-clicking a report listed in the **Geotechnical Reports** section, the editing window opens.
- In the **Report Sections** area, the main headings of the report are listed.
    - **Add Section**: Adds a new section.
    - **Remove Section**: Deletes the selected section.
    - The order of the sections can be rearranged.
- By double-clicking any section, the **Section Properties** window opens (Figure 77).

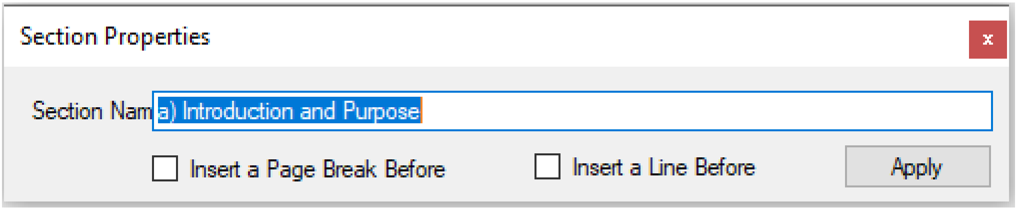


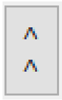
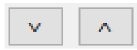
Figure 77. Section Properties

Section Properties

- The section name is entered.
- If **Insert a Page Break Before** is selected, the section starts on a new page.
- If **Insert a Blank Line Before** is selected, a blank line is added before the section.
- Changes are saved by clicking the **Apply** button.

Editing Report Content and Elements

- **Middle Section**: Displays the report’s text, tables, and images::
    - Texts at the top
    - Images in the middle
    - Tables at the bottom
- **Right Section**: Used to edit content elements.

- **Add Text**: Adds a text element to the selected section.
- **Edit Text**: Saves modifications made to the existing text.
- The images and tables in the **Tables/Images** section are transferred to the **Report Element Contents** area. 
- The order of the elements in this area can be rearranged. 

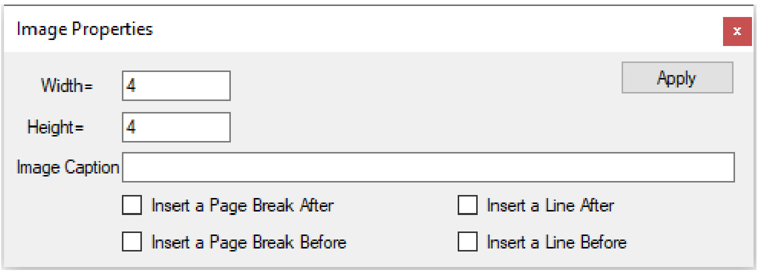


Figure 78. Image Properties

Image and Table Properties

- Double-clicking an image element opens the **Image Properties** window (Figure 78):
  - The width, height, and caption are defined.
  - Blank line or page break settings before/after the image can be adjusted.
- Double-clicking a table element opens the **Table Properties** window (Figure 79):
  - The table title is defined.
  - Blank line or page break settings before/after the table can be configured.
  - The row/column numbers to be hidden can be entered and added to the list.
  - Row and column numbering starts from zero.
  - Changes are saved by clicking the **Apply** button.

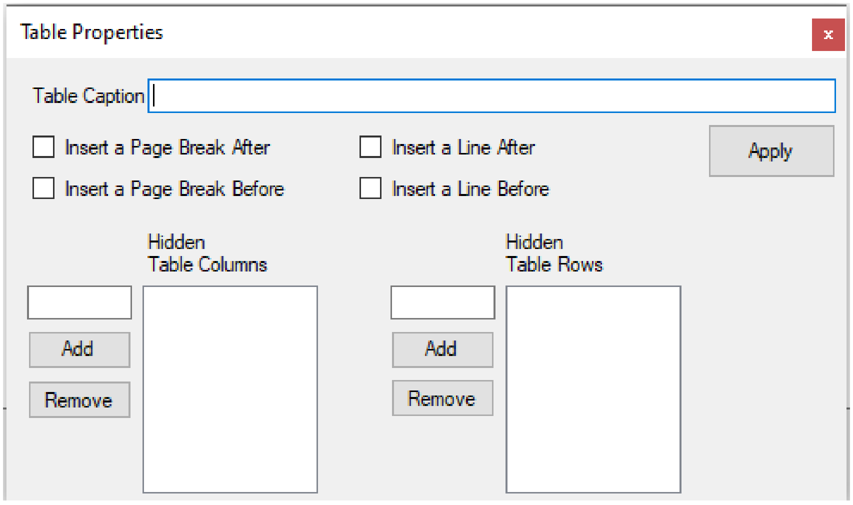


Figure 79. Table Properties

Additional Operations

- The **Import Image** button adds an image from the computer.
- The **Remove** button deletes the selected table or image.
- The **Generate Report** button finalizes and creates the edited geotechnical report.

10. DRAWINGS AND QUANTITIES

The program generates drawing and quantity files for foundations and excavation support structures.

- **Drawings** are prepared in **.dwg** format.
- **Quantity reports** are prepared in **.xlsx** format and saved to the computer.

10.1. Drawings and Quantities of Foundations

In the **Foundation Analysis** window, under the **Piled Foundation Reinforcement** tab, drawing and quantity operations related to foundations are performed (Figure 66).

Quantity Operations

- In the **Quantity Type** field, select the element type for which the quantity will be calculated.
- Click the **Quantity** button to generate the corresponding quantity table.

- The generated tables can be saved to the computer in **.xlsx** format using the **Export to Excel** button.

Reinforced Concrete Detailing

- By clicking the **Reinforced Concrete** button, the rigid columns in the system are reinforced according to the **TS500 – Requirements for Design and Construction of Reinforced Concrete Structures**, ensuring compliance with the minimum reinforcement conditions.
- The generated reinforcement table can be saved in **.xlsx** format using the **Export to Excel** button.

In the reinforcement table, columns such as **pile diameter**, **pile length**, **longitudinal rebar**, **spiral rebar**, **circular rebar**, and **Z-shaped rebar** can be edited by double-clicking the relevant cells. After making any changes, the drawings and quantity reports must be regenerated.

Generating Drawings

- Click the **Piled Foundation Drawing** button to create the drawing of the rigid-column foundation system.
- By right-clicking on the generated drawing, the following operations can be performed via the context menu:
    - Save in **.dwg** format
    - Print the drawing
    - Edit drawing settings

10.2. Drawings and Quantities of Excavation Support Structures

Creating Drawings

- From the **Define** menu, click the **Retaining Wall Project Drawings** button to open the **Project Drawings** window (Figure 80).

- In the opened window, select the desired **perspective**, **plan**, and **section** drawings to be included in the project from the relevant data fields.
- Click the **Generate Project** button to create the drawing for the excavation support structure.
- By right-clicking on the generated drawing, the following options are available from the context menu:
  - Save in **.dwg** format
  - Edit drawing settings

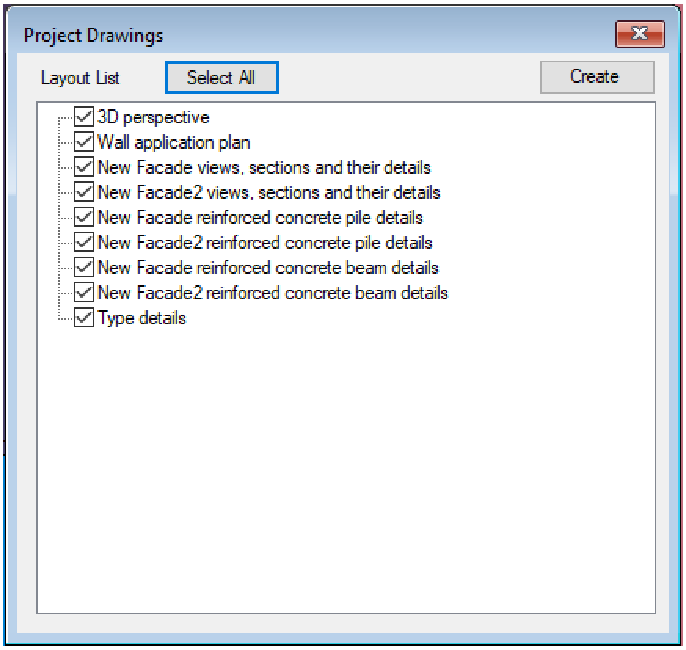


Figure 80. Project Drawings

Drawing Settings

- In the **Retaining Wall Model and Drawing Settings** window, all technical parameters related to the generated wall drawings can be configured (Figure 81).

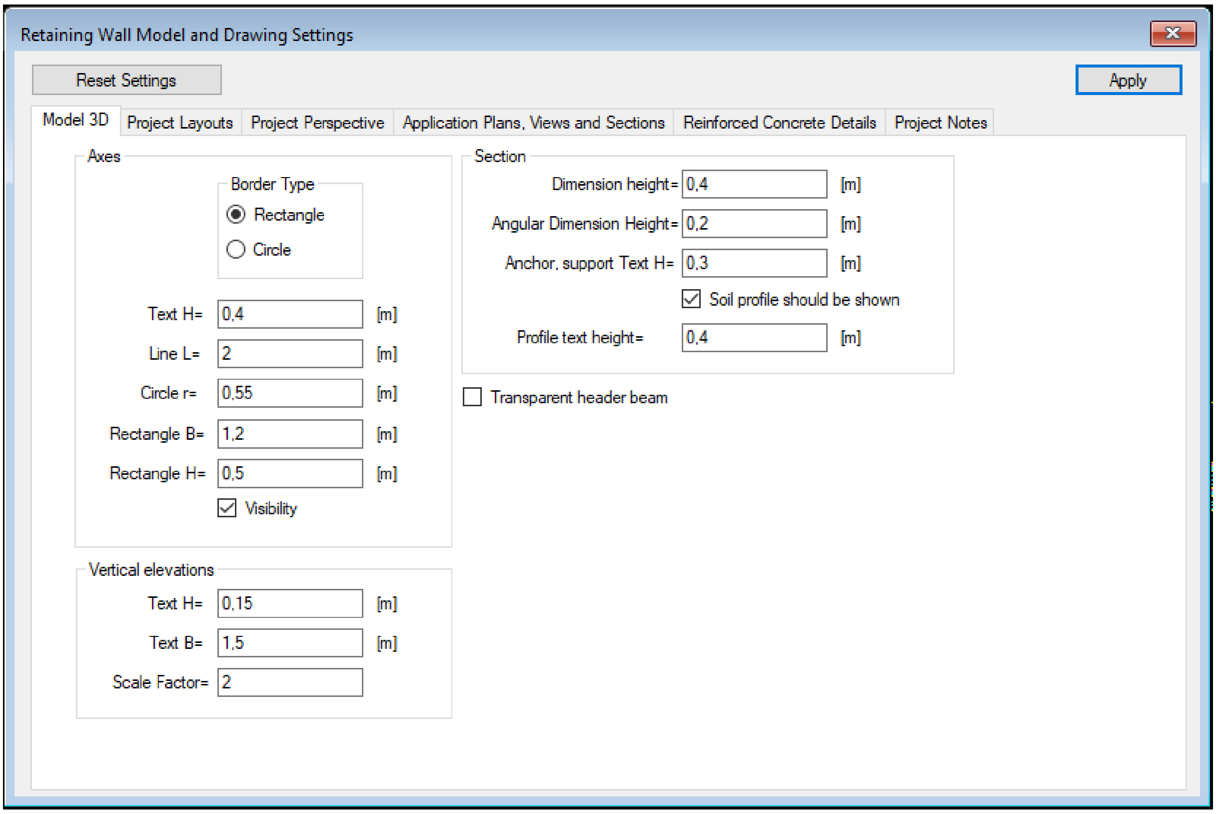


Figure 81. Retaining Wall Model and Drawing Settings

Project Notes

- In the **Project Notes** tab, the texts to be included in the project sheets can be edited.
- The edited notes are automatically added to the end of each sheet by clicking the **Apply** button (Figure 82).

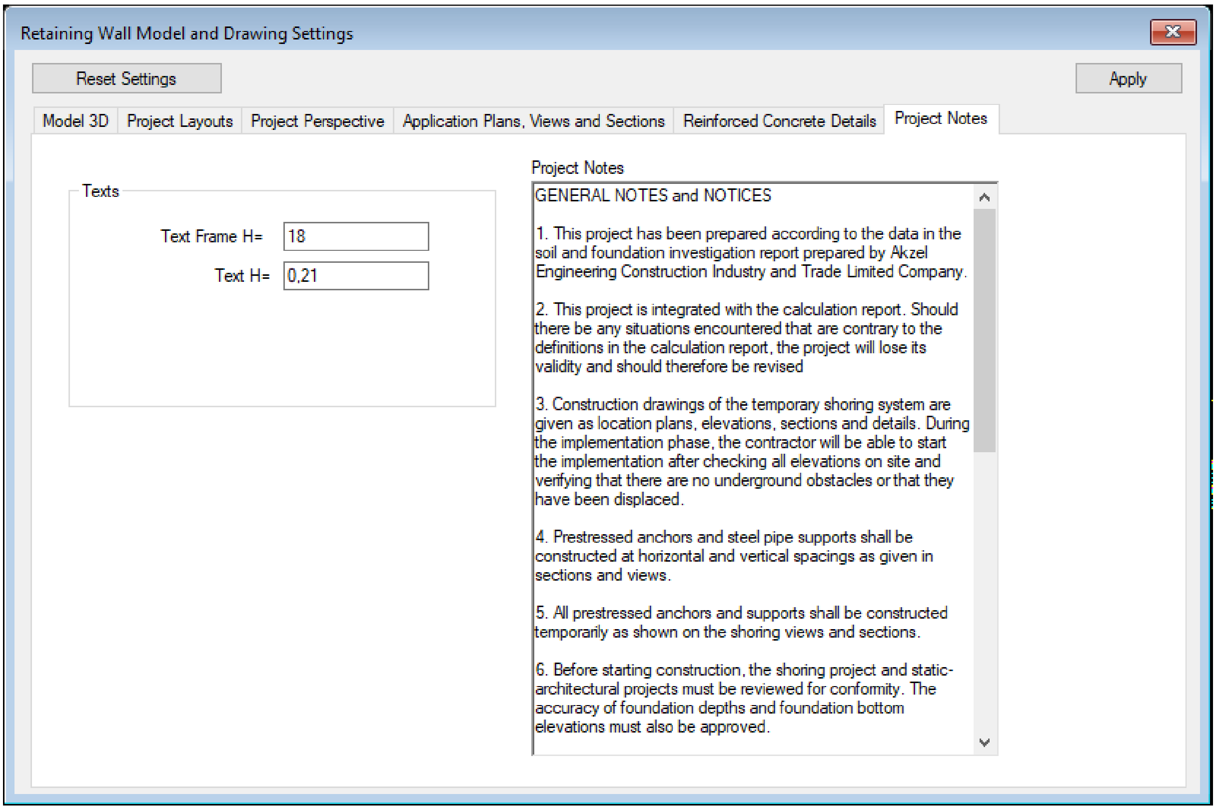


Figure 82. Project Notes

Quantity Calculations

- From the **Define** menu, click the **Retaining Wall Quantities** button to open the **Wall Quantity** window.
- In the **Wall Groups** section, select the wall group for which the quantity will be calculated.
- In the **Quantity Type** field, select the desired quantity category.
- Click the **Calculate Quantity** button to generate the quantity table.
- The generated quantity tables can be exported using the **Export to Excel** button and opened in Microsoft Excel.

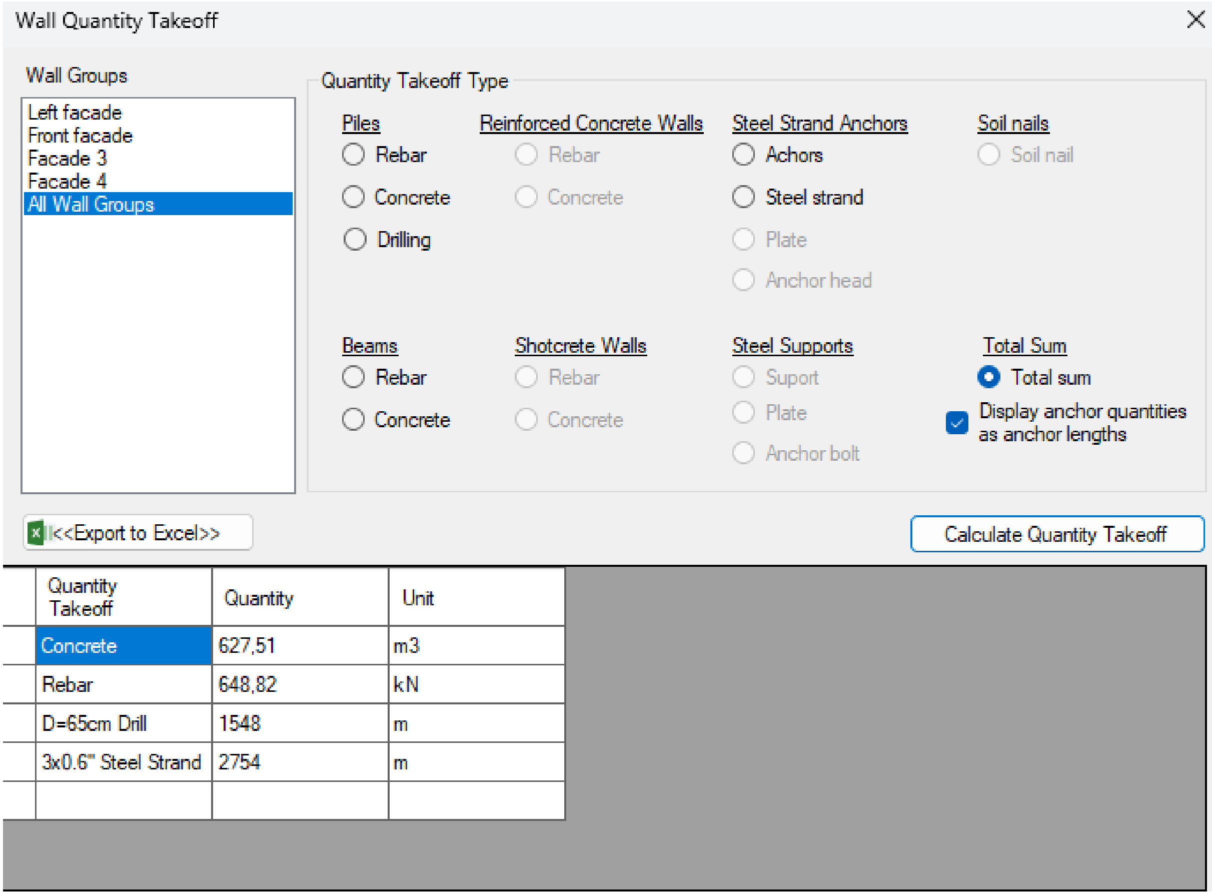


Figure 83. Wall Quantity

11. ANALYSIS SETTINGS

In **SETAF2018**, analysis parameters can be modified parametrically (Figure 84).

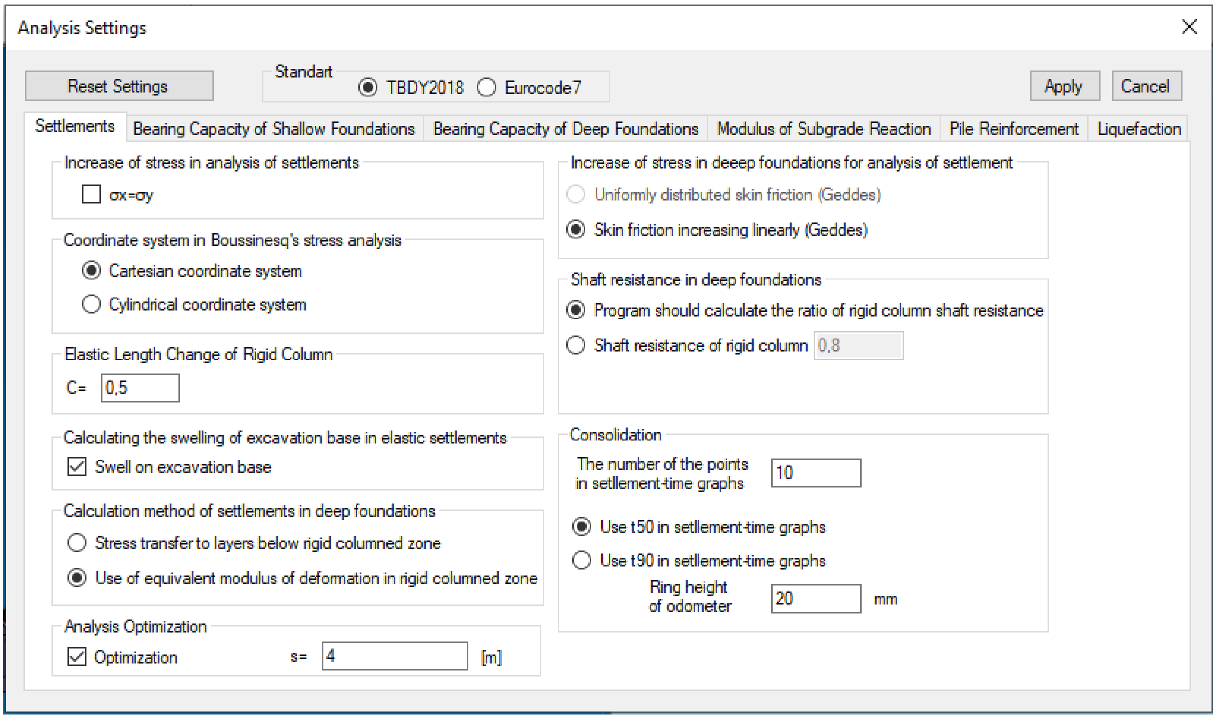
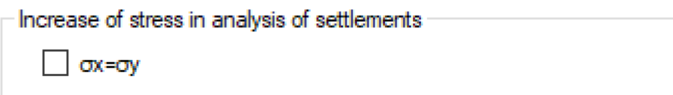
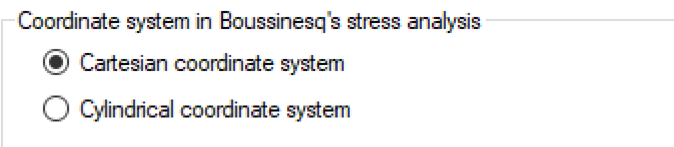


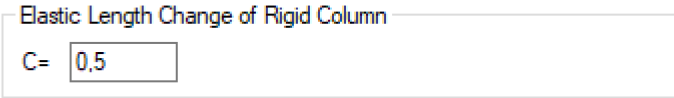
Figure 84. Analysis Settings

- 

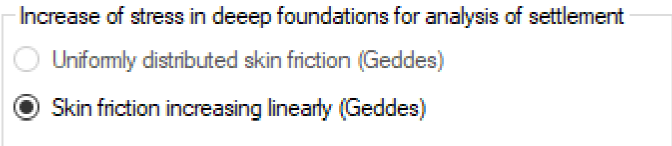
When checked, the stress increments in the **Y-axis** direction during settlement analysis are assumed to be the same as those in the **X-axis** direction.

- 

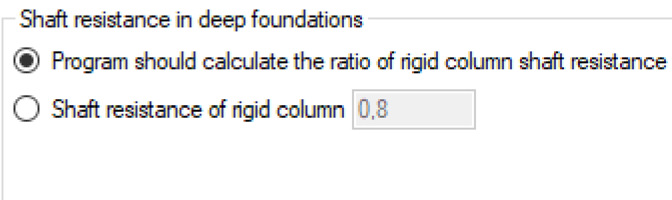
The coordinate system in which stress increments will be calculated during settlement analysis is determined based on the selection. If the model contains a deep foundation, the **cylindrical coordinate system** is automatically selected by the program during analysis. For models without deep foundations, the **Cartesian coordinate system** can be used.

- 

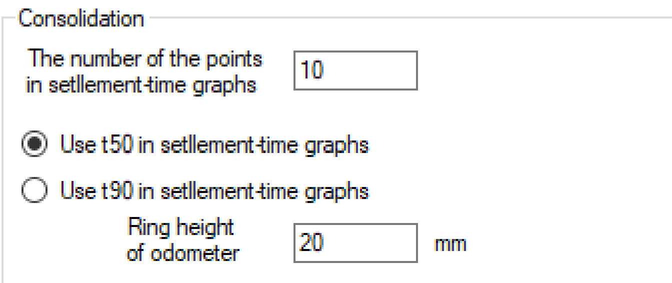
A **reduction factor** is applied to account for the decrease in vertical deformation along the pile. Generally, 0.5 can be used for soils. For clay soils, it can be 0.7. (Budhu, 2008). Values outside the range of 0.5 - 0.7 cannot be entered.

- 

When calculating stress increments in deep foundations using the **Geddes method**, the user can select whether the **shaft resistance** is assumed to be uniformly distributed or linearly increasing. The corresponding Geddes equations are applied based on this selection.

- 

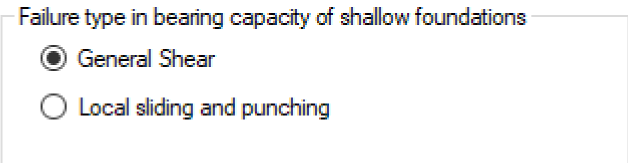
According to this ratio, the portion of the pile load transferred by **shaft resistance** is calculated. The remaining load is transferred through **tip bearing**. If "Let the program determine" is selected, the ratio is automatically calculated as the **shaft resistance divided by the total resistance**. If the user enters a value manually, that ratio is used instead. A value of **0** represents a **tip-bearing pile**, and **1** represents a **friction pile**. Values outside the range **0–1** are not accepted.

- 

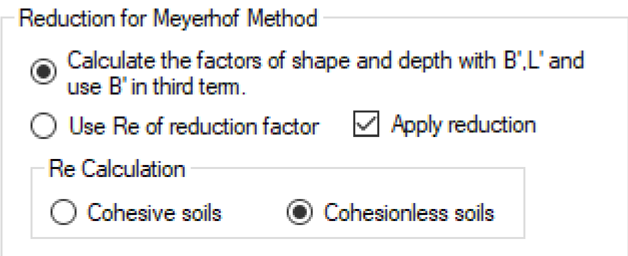
In the **Settlement–Time** graph, the number of points on the curve is determined according to this setting.

Depending on the selection, either the $c_{v,t_{50}}$ or $c_{v,t_{90}}$ consolidation coefficient is used in time-dependent consolidation calculations.

If the user obtains the consolidation coefficients using the **Calculate** button when defining material properties, the specimen height used in the calculation is taken from this parameter.

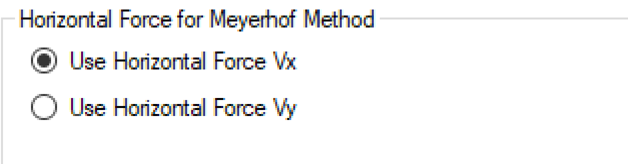
- 

In **shallow foundation** bearing capacity analyses, when **Local Shear or Punching** is selected, the shear strength parameters (**c′** and **ϕ′**) are reduced by multiplying them by **2/3**.

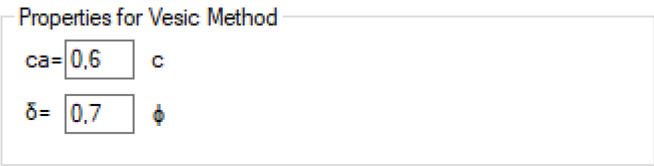
- 

When the **Meyerhof method** is used for bearing capacity analysis of shallow foundations, the reduction factor **R_e** is applied based on the selection.

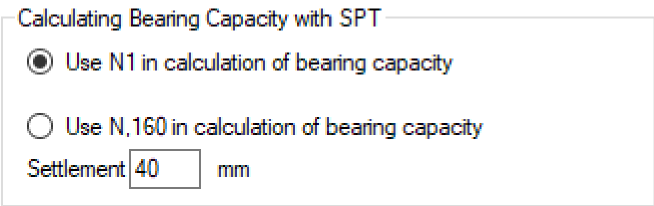
If **R_e** is not selected, shape and depth factors are calculated using the effective dimensions (**B′**, **L′**), and **B′** is used in the third term of the bearing capacity equation.

- 

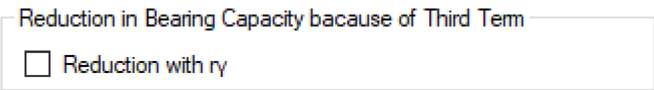
When performing bearing capacity calculations using the **Meyerhof method**, the horizontal load used in the calculation of load inclination factors is determined by the user's choice. By default, the **horizontal load in the X-axis direction** is used.

- 

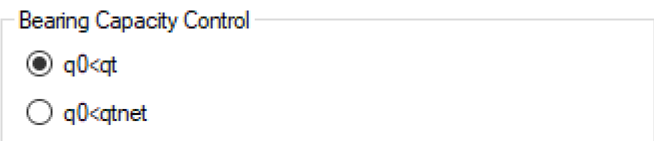
In the **Vesić method** for shallow foundation bearing capacity, the necessary value for **base adhesion** in load inclination factors is taken from this parameter.

- 

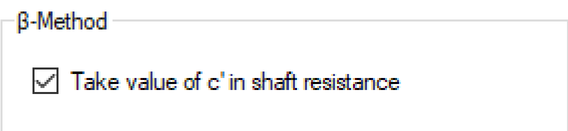
In the **SPT-based** bearing capacity calculation of shallow foundations, the corrected **N_1** or **N_{160}** values are used based on user selection. The bearing capacity is then calculated for the **target settlement value**, which is defined here.

- 

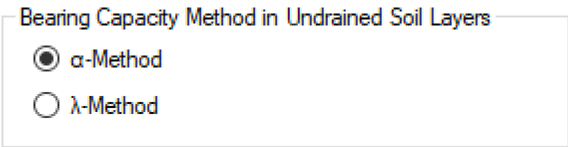
In shallow foundation bearing capacity calculations, a reduction can be applied in the third term of the equation using the **r_γ coefficient** based on user selection.

- 

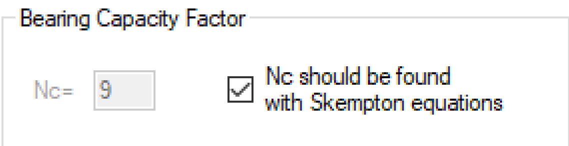
In shallow foundation bearing capacity calculations, the **design bearing capacity** is compared with the **base pressure** to perform a sufficiency check. Depending on the setting, the comparison can be made using either the **design bearing capacity** or the **net design bearing capacity**.

- 

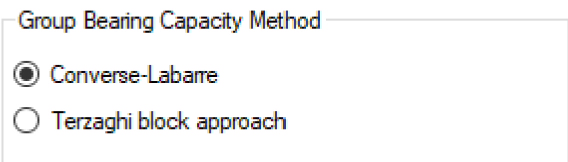
For deep foundation bearing capacity calculations in **drained soil layers**, the **β-method** is used to compute shaft resistance. The **effective cohesion (c′)** can optionally be included in the β-method calculation.

- 

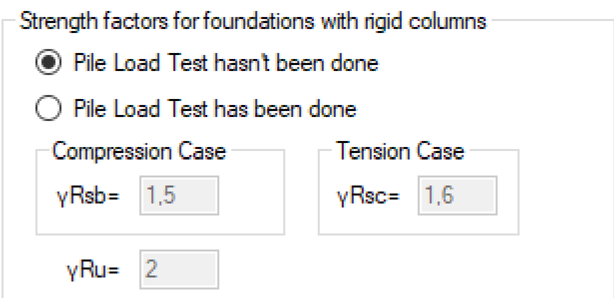
For deep foundation bearing capacity in **undrained layers**, shaft resistance is calculated using either the **α-method** or the **λ-method**, depending on the user's choice.

- 

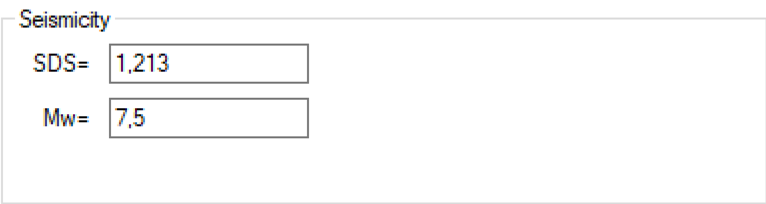
If the pile group tip is located in an **undrained layer**, the tip resistance is calculated using the **α-method**. The **bearing capacity factor (N_c)** can either be calculated from the relevant **Skempton equations** or manually entered by the user.

- 

In deep foundation bearing capacity calculations, the **group efficiency (E_g)** of the pile group can be determined using either the **Converse–Labarre geometric method** or the **Terzaghi block approach**.

- 

The **strength reduction factors** specified in the **TBDY (Turkish Building Earthquake Code)** are selected here for deep foundation bearing capacity calculations. These values are also compatible with Eurocode 7.

- 

In liquefaction analyses, soil liquefaction evaluation is performed according to **Annex 16B of TBDY**. The **local SDS value** can be entered both here and in the analysis window. The default **earthquake moment magnitude (M_w)** is **7.5**, as specified in the code.

12. DATA TABLES

The entered **material**, **soil profile**, and **foundation** data can be displayed in tabular form and exported to **Excel** (Figure 85).

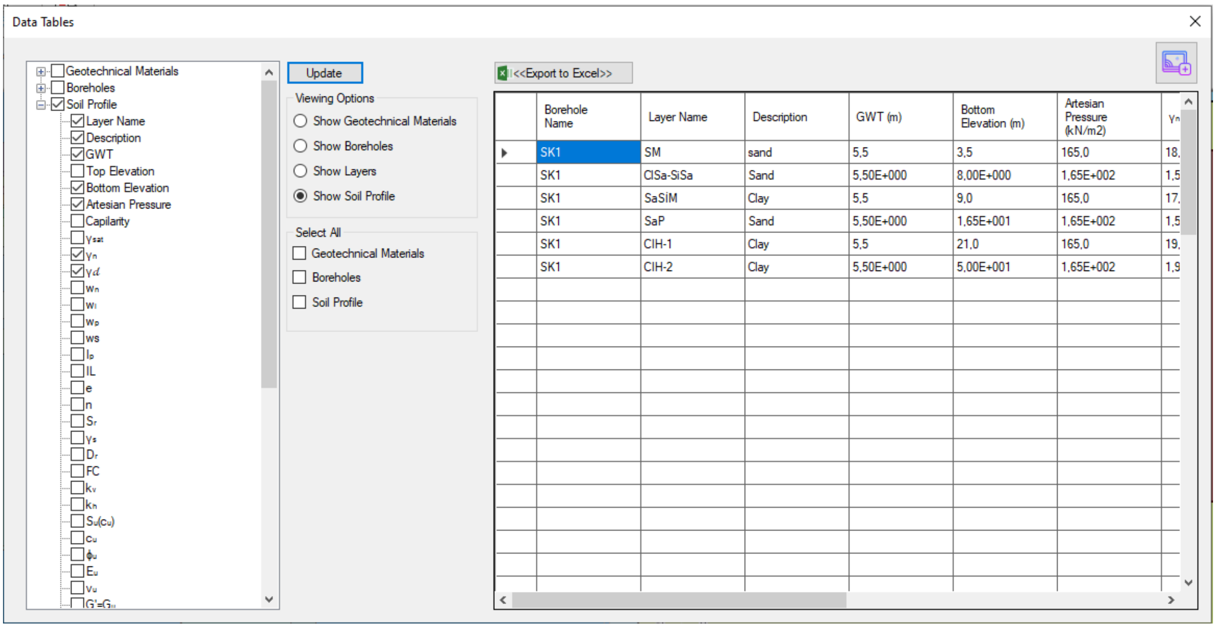


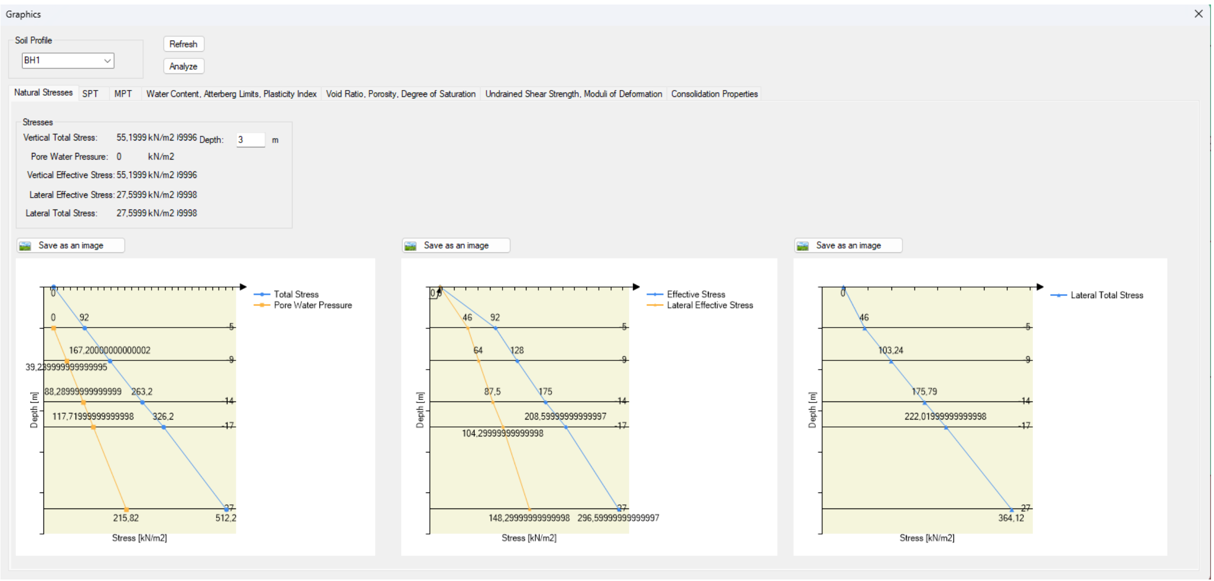
Figure 85. Data Tables

Summary tables can also be generated optionally, depending on the user's selection.

13. GRAPHS

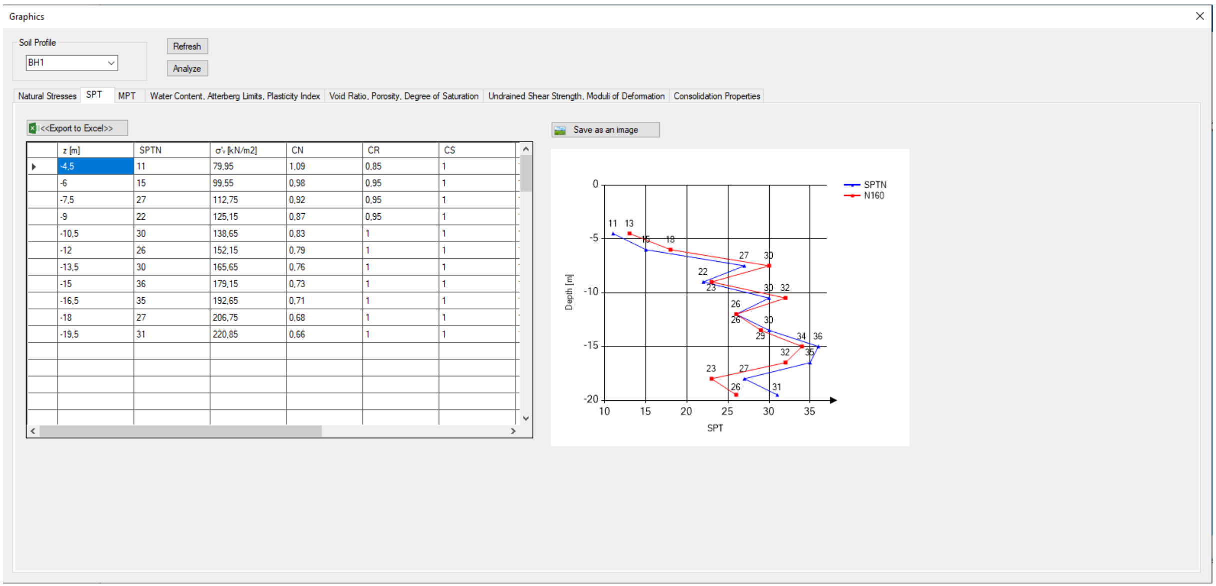
By clicking the **Graphs** button in the **Analysis** menu, charts of the entered material properties are plotted according to the selected soil profile.

13.1. Total, Effective, Pore Water Pressure, Horizontal Effective, and Horizontal Total Stress Diagrams



13.2. SPT

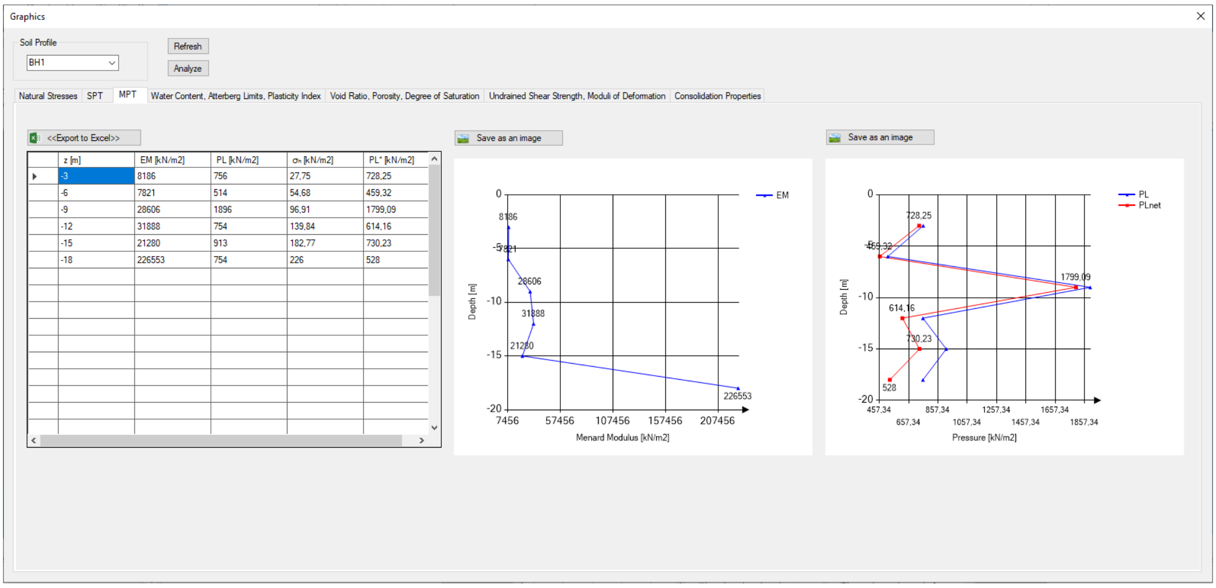
For the boreholes defined in the project, the **SPTN** profile is evaluated, and the **C_N**, **N_1**, and **N_{160}** values are calculated and presented both in tabular and graphical form.



13.3. MPM

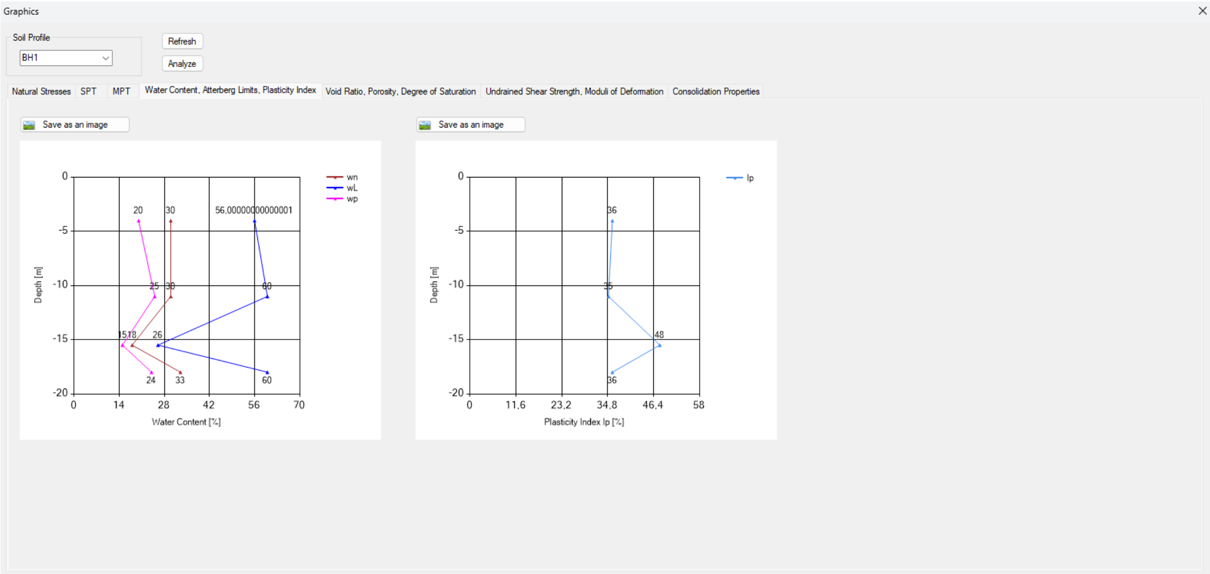
For the boreholes with defined **MPM** profiles, **horizontal total stresses** and **net limit pressures** are calculated and presented in tabular form.

Additionally, graphs of **Menard modulus**, **limit pressure**, and **net limit pressure** are plotted.

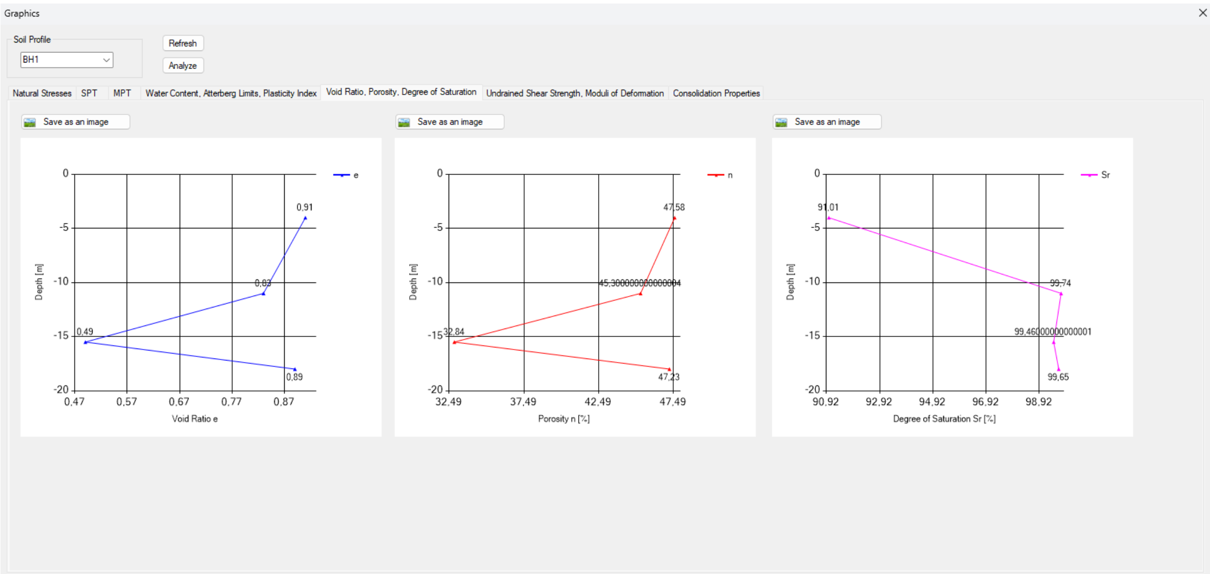


13.4. Water Content, Consistency Limits, and Plasticity Index

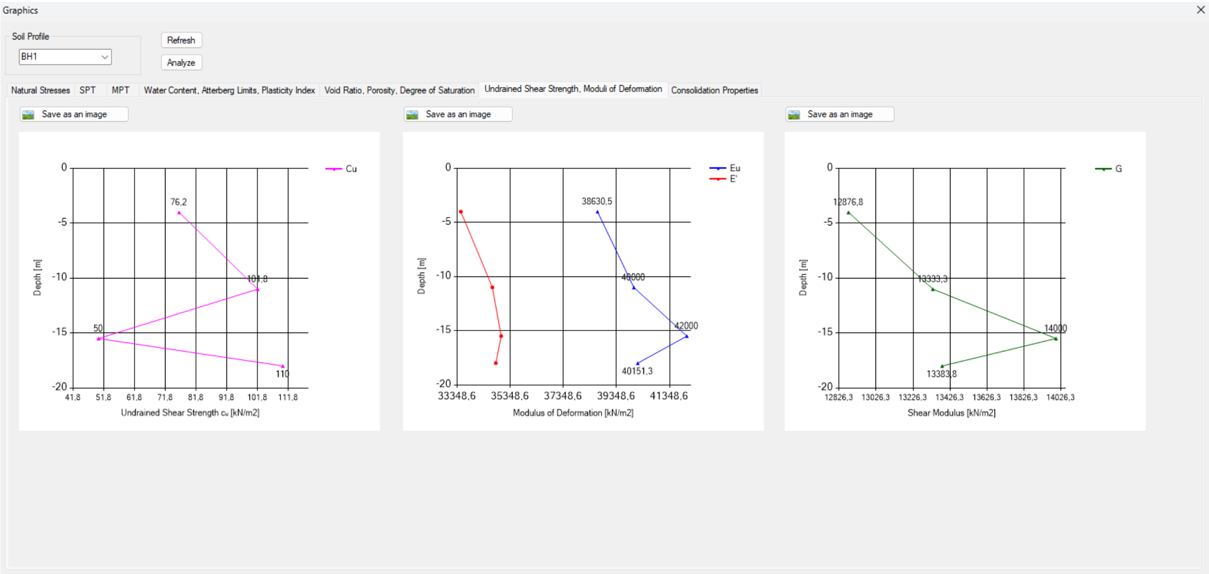
The **natural water content (w_n)**, **liquid limit (w_L)**, and **plastic limit (w_P)** are plotted on the same graph. A separate graph of the **plasticity index** is also generated.



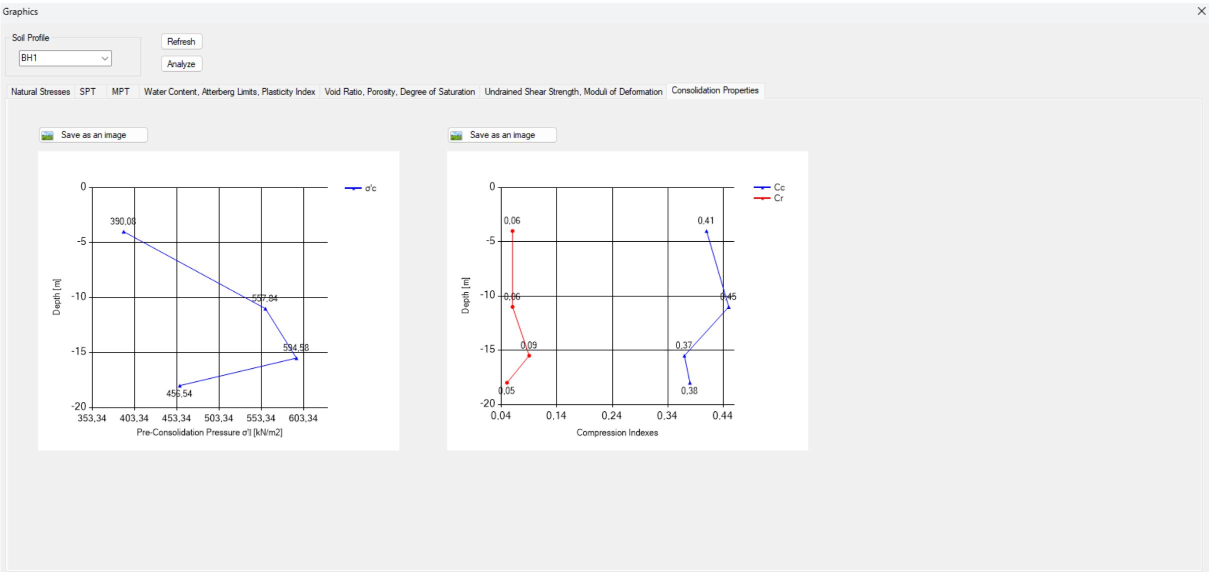
13.5. Void Ratio, Porosity, and Degree of Saturation



13.6. Undrained Shear Strength and Deformation Moduli



13.7. Preconsolidation Pressure and Compression Indices

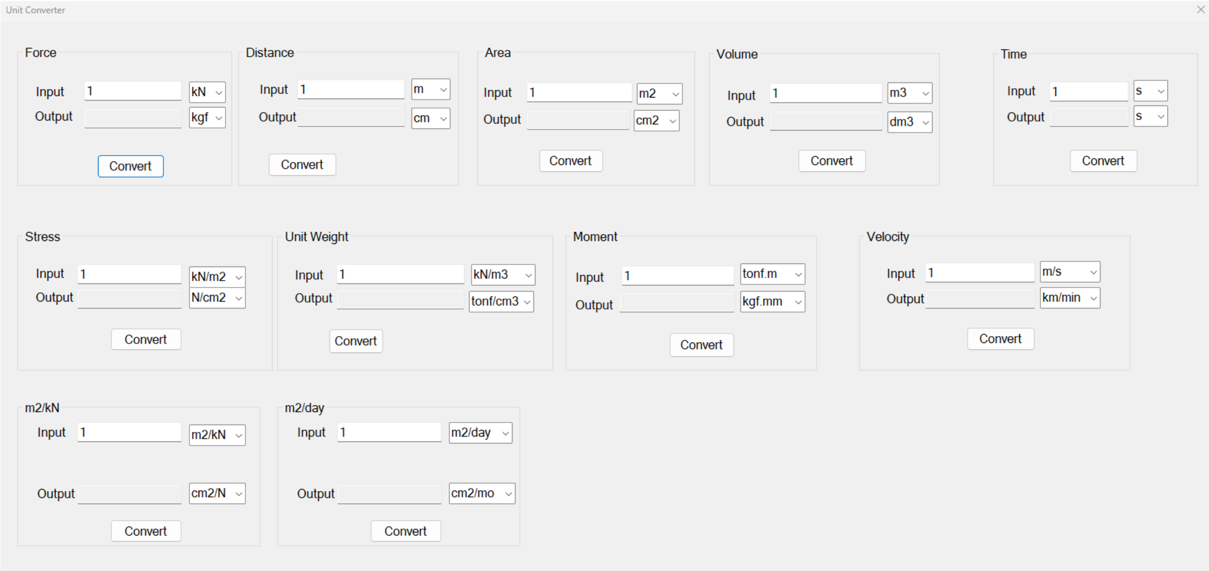


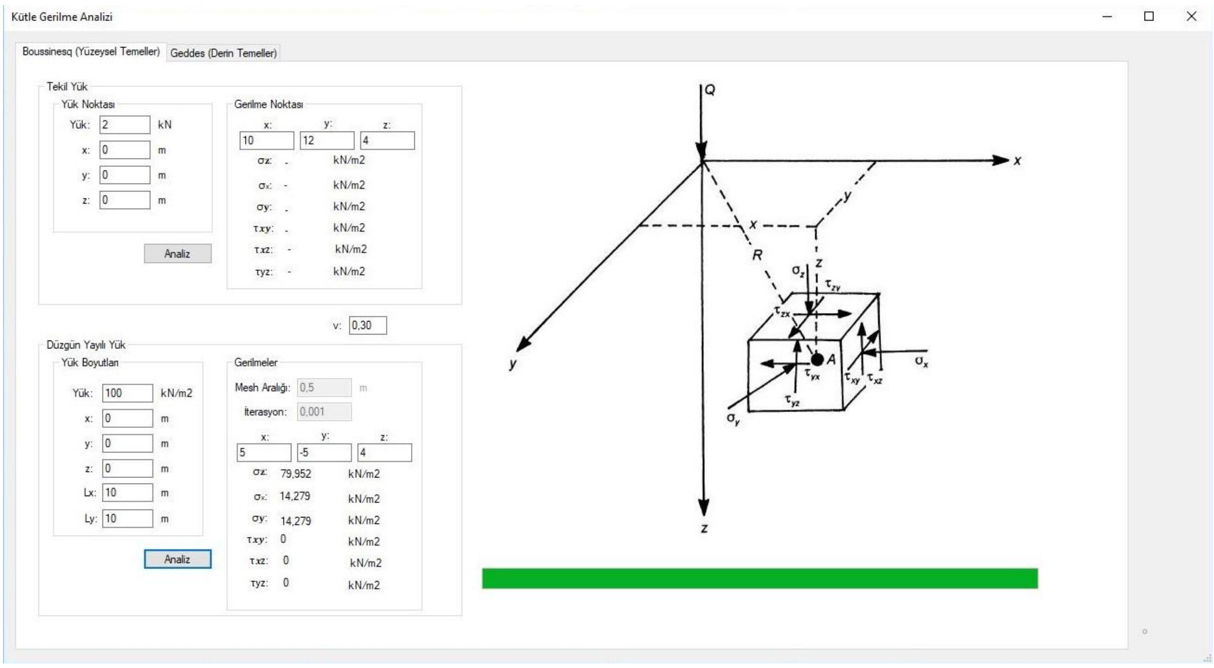
14. TOOLS

In **SETAF2018**, the **Tools** menu provides users with utilities commonly used in the field of geotechnical engineering.

- The **Unit Converter** tool allows users to convert frequently used geotechnical units between different measurement systems.

- The **Mass Stress Analysis** tool, accessible via the **Mass Stress Analysis** button in the **Analysis** menu, calculates stress increments within the soil mass beneath a foundation using **Boussinesq equations**. This tool can also be used for determining **borehole depths** in accordance with the **Seismic Code** requirements.





15. STANDARDS AND ANALYSIS METHODS

All geotechnical analyses and designs in **SETAF2018** are performed in accordance with the following standards and regulations:

- **EN 1997 (Eurocode 7)** – European Standard
- **Kazı Destek Yapıları ve Uygulama Esasları** – Turkish Regulation
- **TBDY** – Turkish Building Earthquake Code

15.1. Analysis Based on Factor of Safety

The **factor of safety (FS)** approach is historically the oldest and most widely used method for verifying structural safety.
Main advantages of this approach:

- Simplicity of calculation
- Clarity and ease of interpretation

In the program, designs are not directly verified using this approach; however, the **calculated safety factors** are displayed to the user.

$$GS^{*} = \frac{R}{E} > GS$$

Definitions:

- **R** = Resistance against failure (strength, capacity, resisting force)
- **E** = Effect causing failure (driving force, acting load, stress)
- **FS*** = Safety factor calculated by the program
- **FS** = Target safety factor defined by the user

In this approach, **loads and soil parameters are not reduced** by any design coefficients. Analyses are performed using **characteristic values**.

15.2. Analysis Based on Limit States

The **limit state theory** verifies structural safety by comparing opposing quantities such as:

- **Resistance** (bearing capacity, strength)
- **Load effect** (shear force, driving stress)

All geotechnical designs in the program are verified according to this approach.

15.3. Compliance with EN 1997

According to **EN 1997-1**, the design of geotechnical structures must be based on the **limit state approach**. In this context:

- Characteristic values of loads, materials, and resistances are defined.
- These values are adjusted using **partial safety factors** according to the selected **design approach** and included in the analysis.

The analyses and designs in SETAF2018 that comply with EN 1997 include:

- Bearing capacity and settlement analyses of **shallow foundations**
- Bearing capacity and settlement analyses of **rigid column foundations** (piles, DSM, jet grouting)
- Analysis and design of **excavation support structures**
- **Slope and embankment stability** analyses

15.3.1. Design Situations

Design situations covered by **EN 1997-1**, supported in SETAF2018:

- Short-term and long-term design situations
- Effects, combinations, and load cases
- Earthquake effects
- Structural sensitivity to deformations
- Effects of new structures on existing buildings, infrastructure, and environment

15.3.2. Geotechnical Design Methods

According to **EN 1997-1**, there are three general calculation approaches::

- **Analytical model**
- **Semi-empirical model**
- **Numerical model**

In the program:

- **Shallow and foundations with rigid inclusions** → Analytical model
- **Stress increments and excavation support analyses** → Numerical model
- **Slope stability analyses** → Limit equilibrium methods

15.3.3. Stress–Strain Method

According to **Annex F of EN 1997-1**, the total settlement is calculated as follows:

1. Determine the stress distribution in the soil mass due to foundation loading (generally based on elasticity theory, assuming homogeneous and isotropic soil).
2. Calculate strain from the stress–strain relationships obtained from laboratory or field tests.
3. Integrate vertical strain values to obtain the total settlement.
4. To ensure accurate results, select sufficient points below the foundation where stresses and strains will be computed.

This method is one of those used in SETAF for total settlement analysis.

15.3.4. Undrained Settlement

According to **Annex F of EN 1997-1**:

- **Short-term (undrained) settlement** is calculated using either the stress–strain method or the modified elasticity method.

- **The program** applies this approach by using the parameters $\mathbf{E_u}$ (undrained modulus of elasticity) and $\mathbf{\nu_u}$ (undrained Poisson's ratio).

15.3.5. Consolidation Settlement

According to **Annex F of EN 1997-1**, settlements caused by consolidation are evaluated based on the following assumptions:

- The soil undergoes one-dimensional deformation, constrained laterally.
- Calculations are performed using the consolidation test curve.

Combining **undrained settlement** and **consolidation settlement** directly often overestimates the total settlement; therefore, empirical corrections may be necessary.

In SETAF:

- The **Terzaghi one-dimensional consolidation theory** is applied.
- Calculations are based on user-provided consolidation test data.
- When the consolidation option is enabled:
  Total Settlement = Undrained settlement (from stress–strain method) + Consolidation settlement

15.3.6. Settlement–Time Behavior

According to **Annex F of EN 1997-1**, the amount of consolidation of cohesive soils can be estimated before full consolidation is achieved. This estimation uses consolidation test parameters and permeability coefficients obtained from field tests.

SETAF calculates amount of consolidation using these parameters and plots **settlement–time curves** accordingly.

15.3.7. Design Approach 2

When the **"Eurocode"** option is selected, SETAF applies **Design Approach 2** as defined in **EN 1997-1**.

15.3.8. Shallow Foundations

SETAF ensures compliance with EN 1997-1 in the design of shallow foundations by verifying:

- Loss of general stability
- Shear failure
- Combined soil–structure damage
- Structural damage due to foundation movement
- Excessive settlements

15.3.9. Foundations on Piles/Rigid Columns

Single Rigid columns are evaluated by their surface adhesion/skin friction and tip resistance. The effect of those calculated resistances are analysed for group action. Design is performed in accoradance with EN 1997-1 using the partial safety coefficients.

Supported rigid column types in SETAF include: **jet grouting, deep mixing, concrete pile, reinforced concrete pile, and micropile.**

15.3.10. Excavation Support Structures

Excavation support designs use the **Design Approach 2 (DA2)** method of EN 1997-1. Designs are performed according to limit states, considering **partial safety factors**.

15.3.11. Slope Stability

Slope stability calculations use methods based on the **partial safety factors** defined in EN 1997-1. These factors are adapted for various limit states to ensure the safety of geotechnical structures.

15.4. Compliance with TBDY

This program computes the bearing capacity of superficial and deep foundation systems in accordance with **TBDY2018** (Turkish Building Earthquake Code). In addition, it is possible to perform liquefaction analysis using standard penetration test results conforming to TBDY Addendum EK16.

15.5. Compliance with FHWA

The ratio of admixture to soil is determined according to the **FHWA standards.** Cement, lime or other binding materials are used as additives depending on the type and properties of the soil to be improved.

15.6. Compliance with the Turkish Regulation on Excavation Support Structures – Design and Application Principles

SETAF performs the design and analysis of excavation support structures in accordance with the current Turkish Regulation on Excavation Support Structures – Design and Application Principles. Anchorages, soil nails, and props are designed according to this regulation, and the deflection of the wall is also checked in compliance with it.

16. THEORY

This chapter summarises the fundamental theoretical principles and geotechnical analysis methods underlying the calculation procedures.

Theoretical Essentials

Soil behaviour, bearing capacity, and settlement analyses in SETAF2018 are analysed by different theoretical approaches and standards.

The main methods and theoretical approaches used are summarised below:

- **Immediate and consolidation settlements** for rectangular and polygonal footings are computed using the **theory of elasticity** and **Terzaghi's one-dimensional consolidation theory.**
- **Stress increases** caused by foundation loads are calculated using the **Boussinesq** and **Mindlin–Geddes equations.**
- **Natural stresses** in the soil include:
  - Total stresses
  - Hydrostatic porewater pressures
  - Vertical and lateral effective stresses

- Horizontal total stresses

These quantities are calculated using the **Coefficient of Earth Pressure at Rest (K_0)** and the **unit weight of soil.**

- The **composite mechanical properties** of soils improved with **rigid columns** are modelled and incorporated into the analyses.
- Analyses of **excavation support structures** are performed using **numerical analysis models** defined within the program.
- **Slope stability analyses** are carried out using the **Limit Equilibrium Method.**
- **Resistance to liquefaction** is evaluated based on **Standard Penetration Test (SPT)** data.
- The **main standards** used in the software are:
  - **Eurocode 7 (EN 1997)**
  - **Turkish Regulation on Excavation Support Structures – Design and Application Principles**
  - **TBDY 2018 (Turkish Building Earthquake Code)**
  - FHWA (Federal Highway Administration, U.S.) Reports

16.1. Natural Stresses

Diagrams of the following stress components are automatically generated for each soil profile defined by the user:

- Total stress
- Hydrostatic porewater pressure
- Effective stress
- Lateral effective stress
- Total horizontal stress

These stress values for the soil profile can be individually calculated and visualised at any selected depth.

When constructing the stress diagrams, the specific conditions assigned to each soil layer by the user are also taken into account. These include:

- Capillarity
- Artesian pressure

Such information is entered into the program during the definition of borehole soil properties.

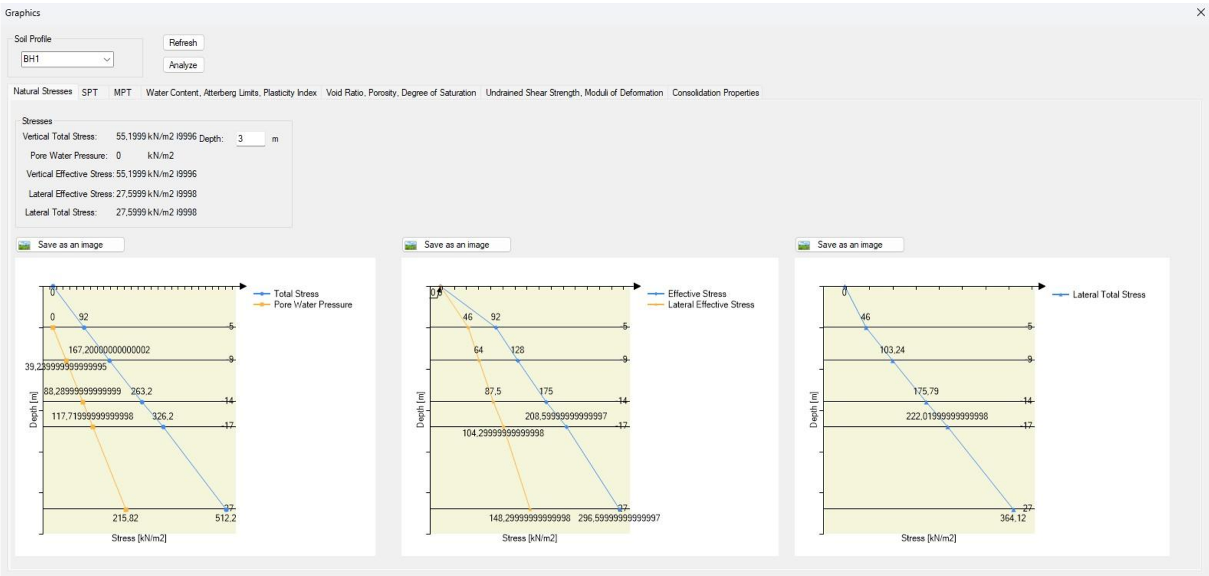


Figure 86. Natural Stresses

16.2. Mass Stresses

SETAF2018 calculates the increases in mass stresses induced by foundation loads using the following methods:

- **Boussinesq solution** for **shallow foundations**
- **Mindlin–Geddes equations** for **deep foundations** (rigid columns and piles)

The stress distributions within the soil mass under applied loads are computed using these methods. The calculated mass stresses are used as primary input values in settlement analyses.

16.2.1. Stress Increase Calculation by Boussinesq

The Boussinesq solution is a theoretical model developed to determine the stress increases in a semi-infinite elastic medium due to a point load. The program applies this solution to uniformly distributed loads acting over rectangular, polygonal, or circular areas by using **numerical integration** techniques.

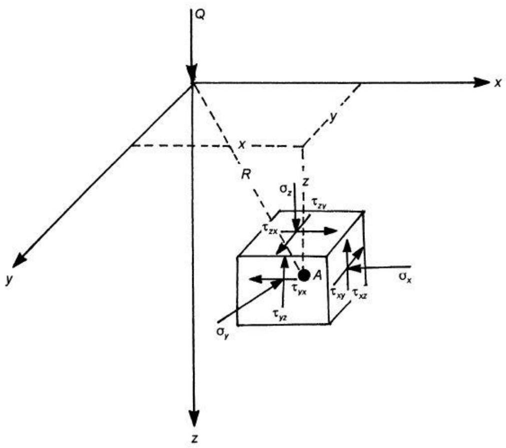


Figure 87. Stress Increase by Boussinesq in the Cartesian Coordinate System

$$
\begin{aligned}
\sigma_z &= \frac{3Qz^3}{2\pi R^5} \\
\sigma_x &= \frac{3Q}{2\pi}\left\{\frac{x^2 z}{R^5} + \frac{1-2v}{3}\left[\frac{1}{R(R+z)} - \frac{(2R+z)x^2}{R^3(R+z)^2} - \frac{z}{R^3}\right]\right\} \\
\sigma_y &= \frac{3Q}{2\pi}\left\{\frac{y^2 z}{R^5} + \frac{1-2v}{3}\left[\frac{1}{R(R+z)} - \frac{(2R+z)y^2}{R^3(R+z)^2} - \frac{z}{R^3}\right]\right\} \\
\tau_{xy} &= \frac{3Q}{2\pi}\left[\frac{xyz}{R^5} - \frac{1-2v}{3}\frac{2(R+z)xy}{R^3(R+z)^2}\right] \\
\tau_{xy} &= \frac{3Q}{2\pi}\frac{xz^2}{R^5}
\end{aligned}
\qquad (1)
$$

SETAF2018 performs the following computational steps using either the **footing contact pressure** or the **applied net stress**:

- Divides the loaded area into sub-sections using its **mesh algorithm.**

- Converts the distributed load into **equivalent point loads** acting at the centers of these sub-sections.
- Calculates the stress increase at a target point due to each point load.
- Sums all incremental values to obtain the **total stress increase** at the desired point.

By progressively decreasing the **mesh spacing**, iterative calculations are performed in both the **Cartesian** and **Cylindrical coordinate systems** to compute the stress increases under the load (Bowles, 1996).

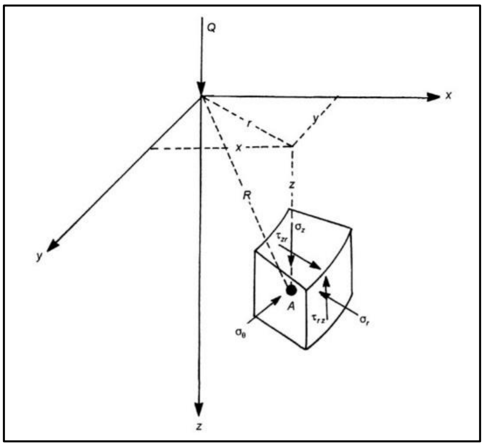


Figure 88. Stress Increase by Boussinesq in the Cylindrical Coordinate System

$$
\begin{aligned}
\sigma_z &= \frac{3Qz^3}{2\pi R^5} \\
\sigma_r &= \frac{Q}{2\pi}\left[\frac{3zr^2}{R^5} - \frac{1-2v}{R(R+z)}\right] \\
\sigma_\theta &= \frac{Q}{2\pi}(1-2v)\left[\frac{1}{R(R+z)} - \frac{z}{R^3}\right] \\
\tau_{rz} &= \frac{3Qrz^2}{2\pi R^5}
\end{aligned}
\tag{2}
$$

Stress Components in the Cartesian Coordinate System (Figure 87)

- $\Delta\sigma_Z$: Normal stress increase in the Z-axis (+ compression)
- $\Delta\sigma_X$: Normal stress increase in the X-axis (+ compression)

- $\Delta\sigma_y$: Normal stress increase in the Y-axis (+ compression)
- $\Delta\tau_{xy}$: Shear stress increase in the Y-direction on a plane normal to the X-axis
- $\Delta\tau_{xz}$: Shear stress increase in the Z-direction on a plane normal to the X-axis
- $\Delta\tau_{yz}$: Shear stress increase in the Z-direction on a plane normal to the Y-axis

Stress Components in the Cylindrical Coordinate System (Figure 88)

- $\Delta\sigma_z$: Normal stress increase in the Z-axis (+ compression)
- $\Delta\sigma_r$: Radial stress increase (+ compression)
- $\Delta\sigma_\theta$: Circumferential (tangential) stress increase (+ compression)
- $\Delta\tau_{rz}$: Shear stress increase in the Z-direction on a plane normal to the r axis

16.2.2. Stress Increase Calculation with Mindlin–Geddes Theory

In the analysis of rigid columns, the Mindlin–Geddes theory is used to calculate the stress increases in the soil under point loading.
Three different loading conditions are defined for a single column, as illustrated in Figure 89:

1. Point loading along the vertical axis
2. Uniformly distributed loading
3. Linearly increasing loading with depth

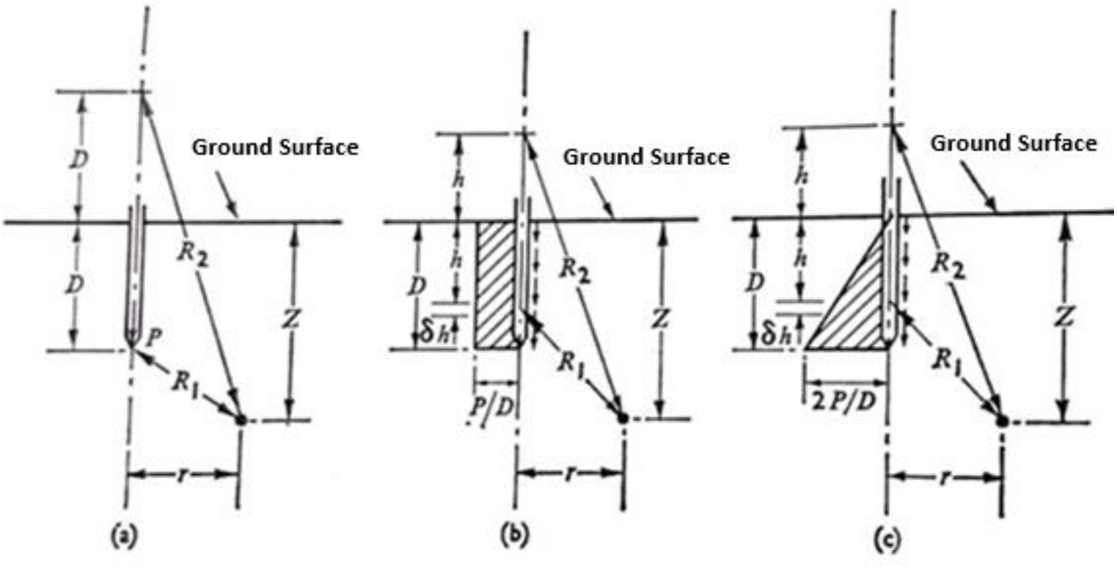


Figure 89. Load Transfer Conditions according to Geddes

$$n = r / D \tag{3}$$

$$m = z / D \tag{4}$$

$$zz = \frac{P.K_{zz}}{D^2} \tag{5}$$

$$rr = \frac{P.K_{rr}}{D^2} \tag{6}$$

$$\theta\theta = \frac{P.K_{\theta\theta}}{D^2} \tag{7}$$

Stress Components

- zz: Vertical normal stress
- rr: Radial stress
- θθ: Circumferential stress

These stresses are calculated using the corresponding **stress coefficients**:

- K_{zz}: Stress coefficient for vertical normal stress
- K_{rr}: Stress coefficient for radial stress
- $K_{\theta\theta}$: Stress coefficient for circumferential stress

Load Transfer Ratio (LTR)

The load transfer ratio is a critical parameter in stress increase calculations.

- **LTR = 0:** Load is fully transferred through tip resistance.
- **LTR = 1:** Load is fully transferred through shaft (skin) resistance.
- **0 < LTR < 1:** Load is transferred proportionally through both tip and shaft resistances.

These conditions are illustrated in Figure 89 as:

- (a): Tip resistance mode
- (b) and (c): Shaft (skin friction) resistance modes

16.2.2.1. Point Loading

Mindlin solved the problem of an elastic medium subjected to a point load P applied at a depth D, deriving the following equations for the stress components along the vertical axis:

$$K_{zz} = zz\frac{D^2}{P} = \frac{1}{8\pi(1-\nu)}\left[-\frac{(1-2\nu)(m-1)}{A^3} + \frac{(1-2\nu)(m-1)}{B^3} - \frac{3(m-1)^3}{A^5} - \left\{\frac{3(3-4\nu)m(m+1)^2 - 3(m+1)(5m-1)}{B^5}\right\} - \frac{30m(m+1)^3}{B^7}\right] \tag{8}$$

$$K_{rr} = rr\frac{D^2}{P} = \frac{1}{8\pi(1-\nu)}\left[\frac{(1-2\nu)(m-1)}{A^3} - \frac{(1-2\nu)(m+7)}{B^3} + \frac{4(1-\nu)(1-2\nu)}{B(B+m+1)} - \frac{3n^2(m-1)}{A^5} + \frac{6(1-2\nu)(m+1)^2 - 6(m+1) - 3(3-4\nu)n^2(m-1)}{B^5} - \frac{30n^2m(m+1)}{B^7}\right] \tag{9}$$

$$K_{\theta\theta} = \theta\theta\frac{D^2}{P} = \frac{1}{8\pi(1-\nu)}\left[\frac{(1-2\nu)(m-1)}{A^3} + \frac{(1-2\nu)(3-4\nu)(m+1) - 6(1-2\nu)}{B^3} - \frac{4(1-2\nu)(1-\nu)}{B(B+m+1)} + \frac{6(1-2\nu)(m+1)^2 - 6(m+1)}{B^5}\right] \tag{10}$$

The coefficients *K* are used to determine the stress increases within the soil medium.

For loading applied along the axis ($n = 0$):

$$K_{zz} = \frac{1}{8\pi(1-\nu)}\left[-\frac{2(2-\nu)}{(m-1)^2} - \left\{\frac{(1-2\nu)(5m+1) + 3m}{(m+1)^3}\right\} - \frac{3(5m+1)}{(m+1)^4}\right] \tag{11}$$

$$K_{rr} = K_{\theta\theta} = \frac{1}{8\pi(1-\nu)}\left[\frac{1-2\nu}{(m-1)^2} + \frac{(1-2\nu)^2}{(m+1)^2} - \frac{6}{(m+1)^4}\right] \tag{12}$$

16.2.2.2. Uniformly Distributed Loading along Vertical Axis

This loading type represents the condition in which a uniformly distributed shaft resistance acts along the vertical axis of a rigid column.

Calculation using Geddes Equations

For a total load *P* developed by the uniformly distributed resistance force along the vertical axis at any depth *D*, the following expressions are used:

Vertical Normal Stress Coefficient:

$$
\begin{aligned}
K_{zz} = zz\frac{D^2}{P} = \frac{1}{8\pi(1-\nu)}\Bigg[& -\frac{2(2-\nu)}{A} + \frac{2(2-\nu)+2(1-2\nu)\frac{m}{n}\left(\frac{m}{n}+\frac{1}{n}\right)}{B} \\
& -\frac{(1-2\nu)2\left(\frac{m}{n}\right)^2}{F} + \frac{n^2}{A^3} + \frac{4m^2-4(1+\nu)\left(\frac{m}{n}\right)^2 m^2}{F^3} \\
& +\frac{4m(1+\nu)(m+1)\left(\frac{m}{n}+\frac{1}{n}\right)^2-\left(4m^2+n^2\right)}{B^3} + \frac{6m^2\left(\frac{m^4-n^4}{n^2}\right)}{F^5} \\
& +\frac{6m\left(mn^2-\frac{1}{n^2}[m+1]^5\right)}{B^5}\Bigg]
\end{aligned} \tag{13}
$$

Radial Stress Coefficient:

$$K_{rr}=rr\frac{D^2}{P}=\frac{1}{8\pi(1-\nu)}\left[\frac{(1-2\nu)}{A}-\frac{(7-2\nu)-12(1-\nu)\frac{m}{n}\left(\frac{m}{n}+\frac{1}{n}\right)}{B}\right.$$
$$-\left\{\frac{4(2-\nu)+12(1-\nu)\left(\frac{m}{n}\right)}{F}\right\}-\frac{n^2}{A^3}$$
$$+\frac{4n^2-2m^2+2(1+2\nu)\left(\frac{m}{n}\right)^2 m^2}{F^3} \qquad (14)$$
$$-\left\{\frac{3n^2-2m^2+2(1+2\nu)\frac{m}{n}(m+1)^2\left(\frac{m}{n}+\frac{1}{n}\right)}{B^3}\right\}$$
$$+\frac{6\left[n^2m^2-m^4\left(\frac{m}{n}\right)^2\right]}{F^5}+\frac{6\left[\frac{m}{n}(m+1)^4\left(\frac{m}{n}+\frac{1}{n}\right)-m^2n^2\right]}{B^5}$$
$$\left.+4(1-\nu)(1-2\nu)\left\{\frac{1}{F+m}-\frac{1}{B+m+1}\right\}\right]$$

where

$F^2=n^2+m^2$

Circumferential Stress Coefficient:

$$
\begin{aligned}
K_{\theta\theta} = \theta\theta\frac{D^2}{P} = \frac{1}{8\pi(1-\nu)}\Bigg[&\frac{(1-2\nu)}{A} + \frac{6-(1-2\nu)(3-4\nu)+6(1-2\nu)\frac{m}{n}\left(\frac{m}{n}+\frac{1}{n}\right)}{B} \\
&+\frac{2(1-2\nu)^2-6(1-2\nu)\left(\frac{m}{n}\right)^2-6}{F} \\
&+\frac{2m^2-4\nu n^2+2(1+2\nu)\frac{m}{n}(m+1)^2\left(\frac{m}{n}+\frac{1}{n}\right)}{B^3} \\
&+\frac{4\nu n^2-2m^2-2(1+2\nu)m^2\left(\frac{m}{n}\right)^2}{F^3} \\
&-4(1-\nu)(1-2\nu)\left\{\frac{1}{F+m}-\frac{1}{B+m+1}\right\}\Bigg]
\end{aligned}
\tag{15}
$$

Special Case on the Axis

If the load is applied on the axis ***(n = 0)*** and the footing radius ratio is ***m > 1***, the equations reduce to the following form:

$$
K_{zz} = \frac{1}{8\pi(1-\nu)}\left[-\frac{4(1-\nu)}{m}-\frac{2(2-\nu)}{(m-1)}+\frac{2(2-\nu)}{(m+1)}+\frac{4m(2-\nu)}{(m+1)^2}-\frac{4m^2}{(m+1)^3}\right] \tag{16}
$$

$$
K_{rr} = K_{\theta\theta} = \frac{1}{8\pi(1-\nu)}\left[-\frac{2+2\nu(1-2\nu)}{m}+\frac{(1-2\nu)}{(m-1)}+\frac{6-(1-2\nu)^2}{(m+1)}-\frac{6m}{(m+1)^2}+\frac{2m^2}{(m+1)^3}\right] \tag{17}
$$

16.2.2.3. Linearly Increasing Loading along the Vertical Axis

This loading mode represents the condition where the shaft resistance acting on the rigid column increases linearly with depth within the soil.

Calculation using Geddes Equations

The coefficients for the total load **P** transmitted by the linearly increasing shaft resistance along the vertical axis at a depth *D* are given as follows:

- Vertical Normal Stress Coefficient:

$$
\begin{aligned}
K_{zz} = zz\frac{D^2}{P} = \frac{1}{4\pi(1-\nu)}\Bigg[& \frac{-2(2-\nu)}{A} + \frac{2(2-\nu)(4m+1)-2(1-2\nu)\left(\frac{m}{n}\right)^2(m+1)}{B} \\
& + \frac{2(1-2\nu)\frac{m^3}{n^2}-8(2-\nu)m}{F} + \frac{mn^2+(m-1)^3}{A^3} \\
& + \frac{4\nu n^2 m + 4m^3 - 15n^2 m - 2(5+2\nu)\left(\frac{m}{n}\right)^2(m+1)^3 + (m+1)^3}{B} \\
& + \frac{2(7-2\nu)mn^2 - 6m^3 + 2(5+2\nu)\left(\frac{m}{n}\right)^2 m^3}{F^3} \\
& + \frac{6mn^2(n^2-m^2) + 12\left(\frac{m}{n}\right)^2(m+1)^5}{B^5} - \left\{\frac{12\left(\frac{m}{n}\right)^2 m^5 + 6mn^2(n^2-m^2)}{F^5}\right\} \\
& - 2(2-\nu)\log_e\left(\frac{A+m-1}{F+m} x \frac{B+m+1}{F+m}\right)\Bigg]
\end{aligned}
\tag{18}
$$

- Radial Stress Coefficient:

$$
\begin{aligned}
K_{rr} = rr\frac{D^2}{P} = \frac{1}{4\pi(1-\nu)}\Bigg[& \frac{(1-2\nu)}{A} + \frac{(7-2\nu)-12m+12(1-\nu)\left(\frac{m}{n}\right)^2(m+1)}{B} \\
& + \frac{12m-12(1-\nu)\frac{m^3}{n^2}}{F} - \left\{\frac{(m-1)^3+mn^2}{A^3}\right\} \\
& + \frac{3(m+1)^3-2m^3+(21-4\nu)mn^2+2(5+2\nu)\left(\frac{m}{n}\right)^2(m+1)^3}{B^3} \\
& - \left\{\frac{2(5+2\nu)\frac{m^5}{n^2}+4(5-\nu)mn^2}{F^3}\right\} \\
& + \frac{6mn^2(m^2-n^2)-12\left(\frac{m}{n}\right)^2(m+1)^5}{B^5} - \left\{\frac{6mn^2(m^2-n^2)-12\frac{m^7}{n^2}}{F^5}\right\} \\
& + (1-2\nu)\log_e\left(\frac{A+m-1}{F+m}\right) + \left\{(1-2\nu)^2-6\right\}\log_e\left(\frac{B+m+1}{F+m}\right) \\
& + 2(1-\nu)(1-2\nu)\left\{\frac{m-1}{B+m+1}-\frac{m}{F+m}\right\}\Bigg]
\end{aligned}
\tag{19}
$$

Circumferential Stress Coefficient:

$$K_{\theta\theta} = \theta\theta\frac{D^2}{P} = \frac{1}{4\pi(1-\nu)}\left[\frac{(1-2\nu)}{A} - \left\{\frac{(1-2\nu)(3-4\nu)+6(1-2\nu)\left(\frac{m}{n}\right)^2(m+1)+6(2m-1)}{B}\right\}\right.$$
$$+\frac{6(1-2\nu)\frac{m^3}{n^2}+12m}{F} - (1-2\nu)\left\{\frac{2(m+1)^3+4mn^2-2\left(\frac{m}{n}\right)^2(m+1)^3}{B^3}\right\}$$
$$+\frac{2(m+1)^3+6mn^2-2m^3-6\left(\frac{m}{n}\right)^2(m+1)^3}{B^3} \tag{20}$$
$$+\frac{\left(2m^3+4mn^2-2\frac{m^5}{n^2}\right)(1-2\nu)}{F^3} - \left\{\frac{6mn^2-6\frac{m^5}{n^2}}{F^3}\right\}$$
$$+(1-2\nu)\log_e\left(\frac{A+m-1}{F+m}\right)+\left\{(1-2\nu)^2-6\right\}\log_e\left(\frac{B+m+1}{F+m}\right)$$
$$\left.-2(1-\nu)(1-2\nu)\left\{\frac{m-1}{B+m+1}-\frac{m}{F+m}\right\}\right]$$

Axial Loading Case

For axial loading and ***m* > *1***, the following special forms of the equations are used:

$$K_{zz} = \frac{1}{4\pi(1-\nu)}\left[2-\frac{2(2-\nu)m}{(m-1)}+\frac{6(2-\nu)m}{(m+1)}-\frac{2(7-2\nu)m^2}{(m+1)^2}\right.$$
$$\left.+\frac{4m^3}{(m+1)^3}-2(2-\nu)\log_e\left\{\frac{m^2-1}{m^2}\right\}\right] \tag{21}$$

$$K_{rr} = K_{\theta\theta} = \frac{1}{4\pi(1-\nu)}\Bigg[11 - 2(1-2\nu)(1-\nu) + (1-2\nu)\log_e\left\{\frac{m-1}{m}\right\} + (1-2\nu)^2\log_e\left\{\frac{m+1}{m}\right\} - 6\log_e\left\{\frac{m+1}{m}\right\} + (1-2\nu)\frac{m}{(m-1)} + \left[(1-2\nu)^2 - 18\right]\frac{m}{(m+1)} + \frac{9m^2}{(m+1)^2} - \frac{2m^3}{(m+1)^3}\Bigg] \tag{22}$$

16.2.3. Stress Increases in a Group of Rigid Columns

The program distributes the total axial load P and bending moments M_x and M_y acting on the pile cap to the rigid column group. The following assumptions are made:

- The pile cap is considered rigid.
- The connections between the columns and the pile cap are assumed to be pinned.

Under these assumptions, the load acting on each column is calculated proportionally to its position relative to the centroid of the column group.

Calculation:

The loads acting on the pile cap are distributed to each column using the following equations:

$$P_n = \frac{P}{n} + \frac{e_x.P}{I_{y-y}}X_n \pm \frac{e_y.P}{I_{x-x}}Y_n \tag{23}$$

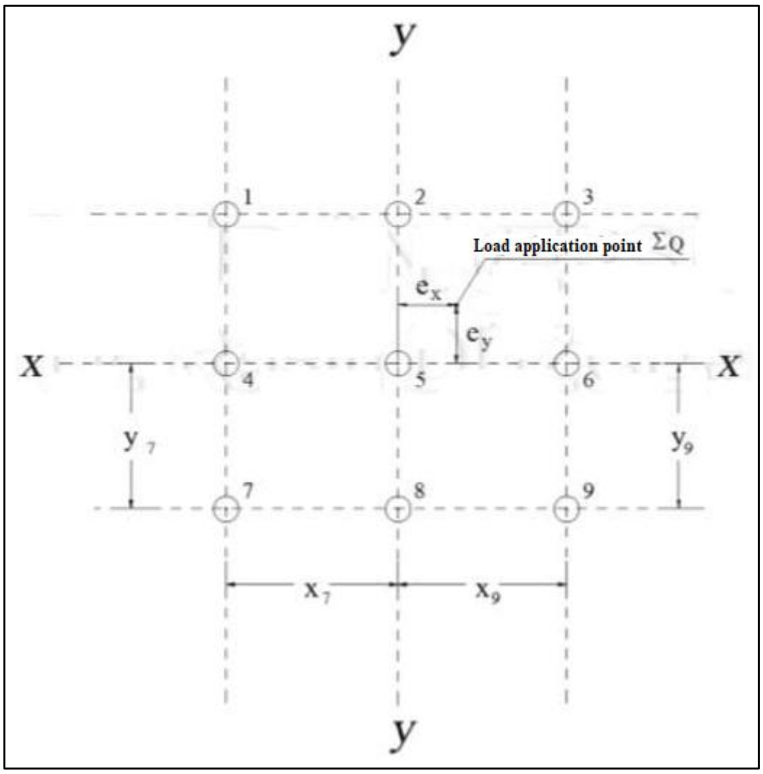


Figure 90. Load Transfer to Rigid Columns

Parameters:

- P: Total axial load acting on the pile cap
- n: Number of columns in the group
- X_n: Distance of each column from the y–y axis
- Y_n: Distance of each column from the x–x axis
- $I_{x\text{-}x}$: Moment of inertia of the column group with respect to the x–x axis
- $I_{y\text{-}y}$: Moment of inertia of the column group with respect to the y–y axis
- e_x: Eccentricity of the applied load from the centroid of the column group (with respect to the y-axis)
- e_y: Eccentricity of the applied load from the centroid of the column group (with respect to the x-axis)
-

$$I_{x-x} = y_1^2 + y_2^2 + y_3^2 + \text{..........} y_n^2 \text{ (column-m2)} \tag{24}$$

$$I_{y-y} = x_1^2 + x_2^2 + x_3^2 + \text{..........} x_n^2 \text{ (column-m2)} \quad (25)$$

After determining the axial load acting on each column, the corresponding stress increases are calculated using the **Mindlin–Geddes** equations. The total stress increase at the target point is then obtained by summing the contributions of all columns.

16.3. Settlements

In SETAF2018, **elastic**, **consolidation**, and **total settlements** are calculated at all user-defined points for both shallow and deep foundations

SETAF2018 calculates the following types of settlement at all user-defined points for shallow and deep foundations:

- **Elastic settlement S_e**
- **Consolidation settlement S_c**
- **Total settlement ΣS**

The total settlement is expressed as:

$$\sum S = S_e + S_c \quad (26)$$

Users can define any number of points on the model to examine settlements in detail at those specific locations. This allows evaluation of both total and differential settlements across the foundation. For each point:

- Surface coordinates are defined.
- Stress increases induced by all foundations are considered.
- Elastic and consolidation settlements are calculated based on these stresses.

The user may specify as many analysis points as required.

16.3.1. Elastic (Immediate/Undrained) Settlements

The immediate settlement at a defined surface point is calculated using the elastic method. This calculation is performed based on the soil profile corresponding to the specified point coordinates.

Each soil layer defined in the soil profile is automatically divided by the program into several sub-layers to obtain more accurate results. The elastic settlement is then calculated as:

$$S_e = \int_0^H \varepsilon dH = \sum_{i=1}^{i=n} \varepsilon_i H_i \tag{27}$$

For each sub-layer, the vertical strain ε_i is calculated. This value is multiplied by the height H_i of the sub-layer to obtain its elastic compression. The strain is defined by the following equation:

$$\varepsilon_i = \frac{\Delta\sigma_i}{E_i} \tag{28}$$

Stress difference:

$$\Delta\sigma_i = \Delta\sigma_v - 2\nu.\Delta\sigma_{hx} \tag{29}$$

Here, $\Delta\sigma_v - \Delta\sigma_h$ represents the difference between the vertical and horizontal stress increases at the midpoint of the sub-layer. Under two-dimensional conditions ($\sigma_{hx} = \sigma_{hy}$) and in undrained conditions ($\nu_u = 0.50$), this difference corresponds to the deviatoric stress increase. E_i represents the modulus of elasticity of the parent **soil layer** to which the sub-layer belongs. This modulus is assigned as E' or E_u depending on the drainage condition.

If the option "$\sigma_x = \sigma_y$" in the analysis settings is deactivated, the following equation is used:

$$\Delta\sigma_i = \Delta\sigma_v - \nu.(\Delta\sigma_{hx} + \Delta\sigma_{hy}) \tag{30}$$

In the cylindrical coordinate system, it is expressed as:

$$\Delta\sigma_i = \Delta\sigma_v - \nu.(\Delta\sigma r + \Delta\sigma_\theta) \tag{31}$$

SETAF2018 computes the vertical and horizontal stress increases at the center of each sub-layer using the user-defined load intensity, coordinates, and geometry. The calculations are performed using the Boussinesq or Mindlin–Geddes equations. If the layer is **undrained**, E_u and $\nu_u = 0.50$ are used; if **drained**, E' and ν' are applied.

16.3.2. Consolidation Settlements

The consolidation settlement of a surface point defined by coordinates (x, y) is calculated in the program based on the corresponding soil profile.

For underconsolidated soils:

$$(\sigma_0^{'} > \sigma_c^{'}) \qquad S_c = H_0 \frac{C_c}{1+e_0} \log \frac{\sigma_0^{'} + \Delta\sigma}{\sigma_c^{'}} \tag{32}$$

For normally consolidated soils:

$$(\sigma_0^{'} = \sigma_c^{'}) \qquad S_c = H_0 \frac{C_c}{1+e_0} \log \frac{\sigma_0^{'} + \Delta\sigma}{\sigma_c^{'}} \tag{33}$$

For overconsolidated soils:

If the sum of the in-situ effective stress and the stress increase exceeds the preconsolidation pressure ($\sigma_0^{'} + \Delta\sigma > \sigma_c^{'}$):

$$S_c = H_0 \frac{C_r}{1+e_0} \log \frac{\sigma_c^{'}}{\sigma_0^{'}} + H_0 \frac{C_c}{1+e_0} \log \frac{\sigma_0^{'} + \Delta\sigma}{\sigma_c^{'}} \tag{34}$$

If it is smaller ($\sigma_0^{'} + \Delta\sigma < \sigma_c^{'}$):

$$S_c = H_0 \frac{C_r}{1+e_0} \log \frac{\sigma_0^{'} + \Delta\sigma}{\sigma_0^{'}} \tag{35}$$

For each sub-layer, the calculated $\sigma_0' + \Delta\sigma$ value is compared with its preconsolidation pressure σ'_c to determine which equation (34) or (35) will be applied. The preconsolidation pressure for a single sub-layer may be taken as equal to the value assigned to its parent soil layer. If there is more than one sub-layer, different σ'_c values are determined depending on the material properties and depth, as these values vary with depth.

To obtain σ'_c values for sub-layers within the same soil layer:

- Each sub-layer is defined by the depth and σ'_c value of the soil layer to which it belongs. If the layer contains more than one material, the depth is taken as the average value.
- When the **User-Defined** option is selected, the depth assigned to the material corresponds to the mid-depth of the layer. At this depth, the effective vertical stress σ'_v is calculated, and the difference (σ'_c - σ'_v) is determined.
- This difference is added to the average effective stress of each sub-layer, giving the preconsolidation pressure at the sub-layer midpoint as: $\sigma'_{ci} = \sigma'_{0vi}$ + Difference

Using these values, the Overconsolidation Ratio (OCR) is calculated as: $OCR_i = s¢_{ci} / s_0¢_{vi}$

If OCR ≥ 1, the sub-layer is considered **overconsolidated (OC)**; if OCR < 1, it is considered **normally consolidated (NC)**.

The consolidation settlement of the specified point is obtained by summing the compressions of all sub-layers in the soil profile:

$$S_c = \sum_{i=1}^{i=n} S_{ci} \tag{36}$$

16.3.3. Settlement–Time Analysis and Curves

The program calculates the percentage of consolidation and the corresponding settlements for the user-defined time period in all layers according to **Terzaghi's consolidation theory**. However, it should be noted that if the analyzed soil layer is located **above the groundwater table (GWT)**, the actual settlements will be smaller than the calculated ones.

The variations of consolidation settlement and total settlement with time, up to the specified duration, are displayed graphically.

Consolidation Time and Time Factor

For each layer, the time factor (T_v) corresponding to the consolidation period *t* is obtained from the following equation:

$$T_v = \frac{c_v.t}{(H/n)^2} \tag{37}$$

- n: Number of drainage paths (2 for double drainage, 1 for single drainage)
- c_v: Coefficient of consolidation

Distribution of Excess Porewater Pressure

The distribution of porewater pressures within each layer is determined according to the boundary conditions of the soil layer, as illustrated in Figure 91. Depending on the permeability characteristics, different distributions of excess porewater pressure (*u*) and the related $\mathbf{T_v}$ values are used.

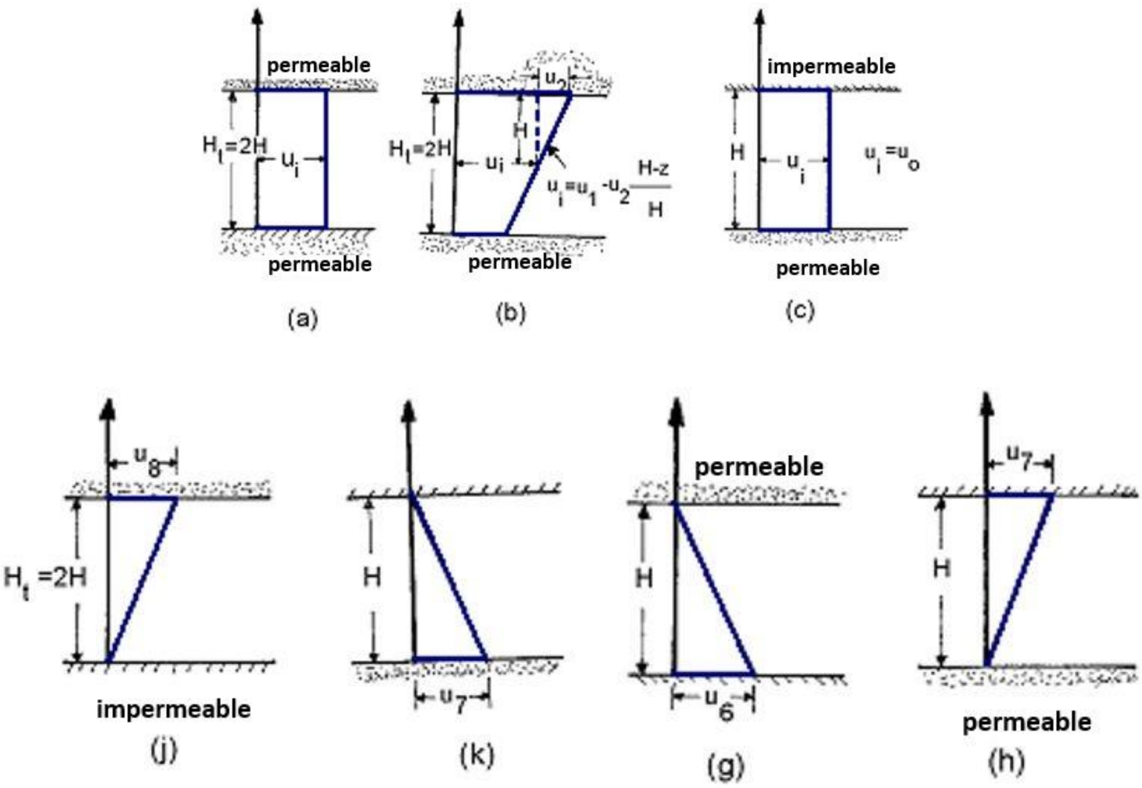


Figure 91. Pore Water Pressure Distributions

Percent Consolidation

Once the **Time Factor (T_v)** for each layer has been determined, the **percent consolidation (U)** is obtained from **Table 1**, depending on the boundary condition.

Table 1. U–T_v Relationship for Different Boundary Conditions

Uniform Linear Distribution (a, b, c) Case		Linear Distribution (j, k) Case		Linear Distribution (h, g) Case	
0,1	0,008	0,1	0,003	0,1	0,047
0,2	0,031	0,2	0,009	0,2	0,1
0,3	0,071	0,3	0,024	0,3	0,158
0,4	0,126	0,4	0,048	0,4	0,226
0,5	0,197	0,5	0,092	0,5	0,294
0,6	0,287	0,6	0,16	0,6	0,39
0,7	0,403	0,7	0,271	0,7	0,5
0,8	0,567	0,8	0,44	0,8	0,665
0,9	0,848	0,9	0,72	0,9	0,94

The *U* values for boundary conditions such as *(a, b, c)*, *(j, k)*, and *(h, g)* are selected from the table.

Consolidation compression is calculated by considering the consolidation percentages at each layer. The compressions calculated for each layer are added up to arrive at the total consolidation compression for the time interval selected. Initially calculated elastic settlement value is added to the consolidation settlement to obtain total settlement value. Settlement-time curves for the selected interval are plotted to show the development of settlements up to the time of completion.

Correction of Settlement–Time Curve for the Duration of Construction

The load applied by the structure in practice is never abrupt as assumed by theory. Loading starts with the excavation of foundation pit, followed by progress of building at different storeys of the building. This situation results in a lesser consolidation effect compared to the sudden loading in theory.

Figure 92 illustrates this condition and the correction applied:

- A certain amount of settlement has taken place during construction.
- Only half of calculated total settlement occurs at the point shown on the theoretical curve.
- As loading increases gradually by elapsed time, the settlement curve is shifted to the right to revise the theoretical curve.
- Net load **P** acting on the ground is the difference between the weight of the structure and the soil excavated. This load is applied in steps. $t_{c,month}$ is the total time taken for construction and represents the compression effect in the time-settlement analysis.

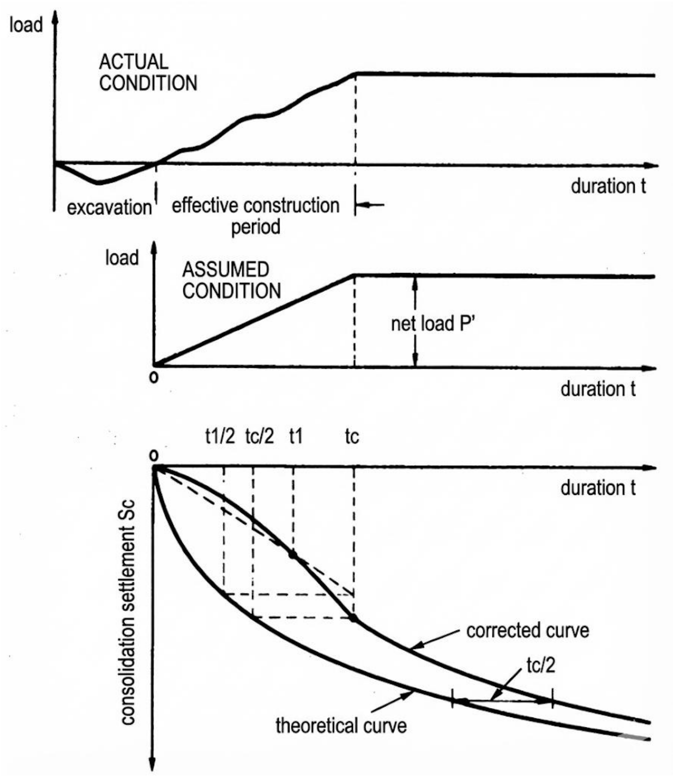


Figure 92. Correction of Settlement-Time Curve considering the Construction Period

The method described here is based on the assumption that the percent consolidation at time t_c days was caused by the application of full load was implemented at time $½t_c$. Therefore, actual settlement at any time t would be one half of the value had load application was sudden.

The calculated settlement must be reduced proportionally at this stage because the full load has not yet been applied. Calculated settlement at any time must be shifted to the right by half of the effective contruction time. The effect of construction time on settlement curve decreases as time progresses.

SETAF2018, draws the settlement-time curve using the **Terzaghi corrections**.

16.3.4. Settlement of Shadow Footings

SETAF2018 employs the following method to perform settlement analysis:

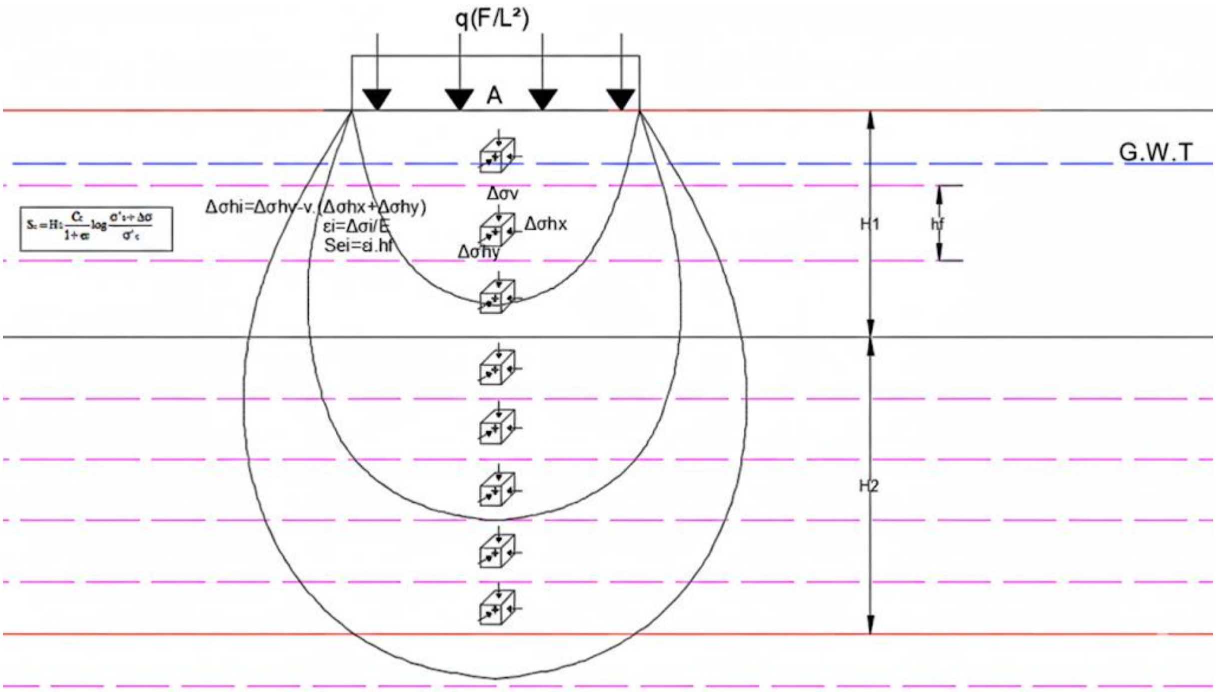


Figure 93. Calculation of Stress Distribution under a Footing using the Boussinesq Method

- Stress increase at the mid-level of each sub-layer is calculated using the Boussinesq method. If the **“Swell on excavation base”** option is selected, the net stress ($q = q_{net}$) is obtained by subtracting the weight of the excavated soil. For consolidation calculations, qnet is always considered.
- Her alt katman ortasında düşey efektif gerilme hesaplanır. Effective vertical stress at the mid-level of each sub-layer is calculated.
- The elastic strain of each sub-layer is calculated using the equations $\Delta\sigma_i = \Delta\sigma_v - \nu.(\Delta\sigma_{hx} + \Delta\sigma_{hy})$ and $\varepsilon_i = \dfrac{\Delta\sigma_i}{E_i}$. The elastic compression of the sub-layer is

calculated using the equation $S_{ei}=\varepsilon_i.h_f$. Its mechanical properties are taken as that of the layer. Total or effective parameters are used depending on the drainage conditions.

- Consolidation compression of the sub-layers within undrained layers is also calculated. Equations (32), (33), (34), and (35) are used to compute the compression of each sub-layer based on the acting effective stress and the stress increase.
- In comparing the equations the precompression value, σ'_c of the sub-layer is obtained using the depth of the material, the effective stress and the σ'_c of this layer.
- $S_e = \int_0^H \varepsilon dH = \sum_{i=1}^{i=n} \varepsilon_i H_i$ to calculate elastic compression at point A
- $S_c = \sum_{i=1}^{i=n} S_{ci}$ to calculate consolidation compression at point A
- Total compression is $\Sigma S=S_e+S_c$.

16.3.5. Settlement of Rigid Column Groups

SETAF2018 applies the following procedure for settlement analysis:

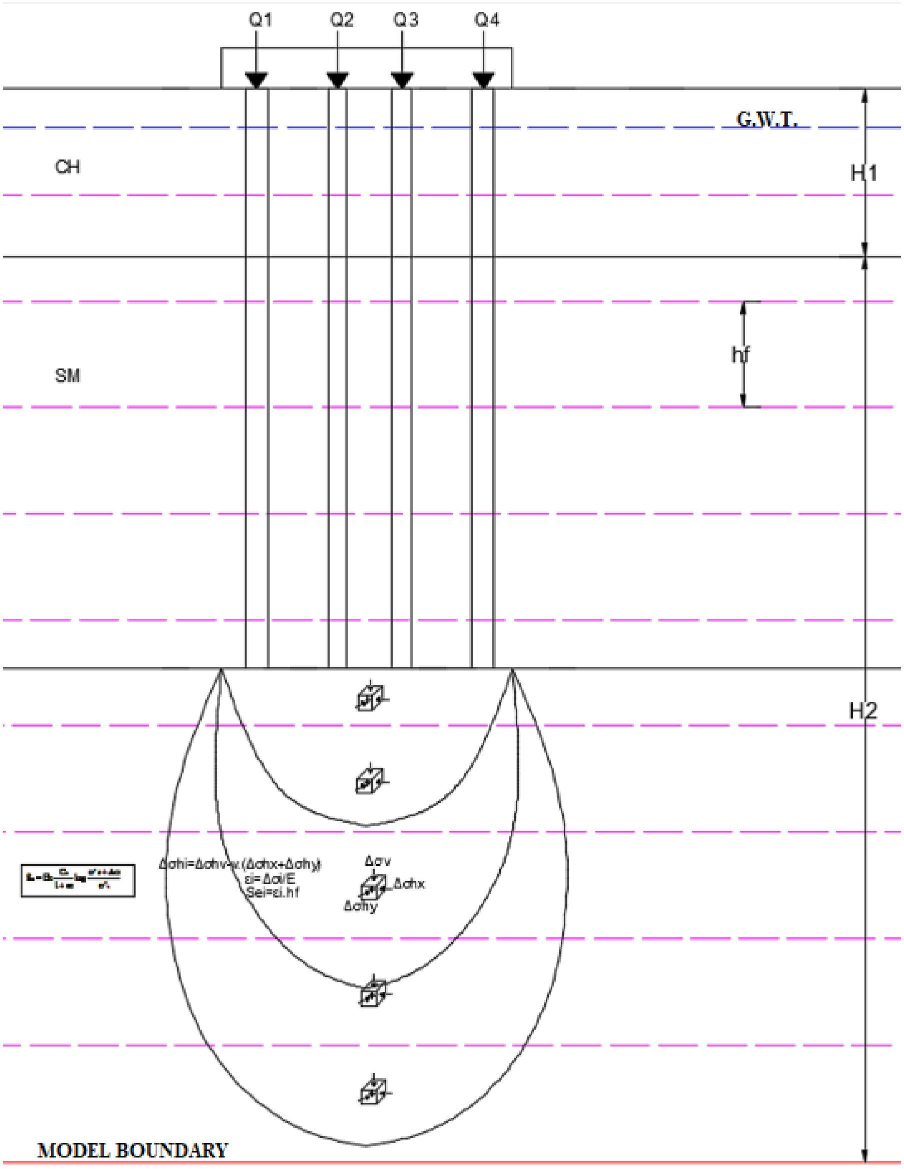


Figure 94. Transfer of Stresses Below the Pile Zone Using the Mindlin–Geddes Method

If the foundation consists of rigid column-type piles and the option **“Stress transfer to layers below rigid columns zone ”** is selected:

- Load **P** is distributed to the piles assuming pinned (hinged) connections. If the option **“Swell on excavation base”** is selected, the weight of the excavated soil is subtracted, and **P = $P_{group,net}$**. If not selected, **P = P_{group}**. Stress increases at the mid-depth of each sub-layer below the pile zone are calculated using the **Mindlin–Geddes method**, and the load carried by each pile is transferred to the mid-level of the corresponding sub-layer.

- **Effective stress** is calculated at the mid-level of each sub-layer.
- The **elastic vertical strain** of each sub-layer below the pile zone is obtained using equations $\Delta\sigma_i = \Delta\sigma_v - \nu.(\Delta\sigma_{hx} + \Delta\sigma_{hy})$ and $\varepsilon_i = \frac{\Delta\sigma_i}{E_i}$. The **elastic compression** of each sub-layer is calculated using the equation $\mathbf{S_{ei} = \varepsilon_i \cdot h_f}$. Mechanical properties of each sub-layer are taken from the corresponding soil layer. Depending on the drainage condition, either **total** or **effective stress parameters** are used.
- The **consolidation compression** of sub-layers within undrained layers below the pile zone is also calculated. Equations (32), (33), (34), and (35) are used with the acting **effective stress** and **stress increase** at the mid-level of each sub-layer.
- In comparing the equations, the precompression value, σ'_c of the sub-layer is obtained using the depth of the material, the effective stress and the σ'_c of this layer.
- $S_e = \int_0^H \varepsilon dH = \sum_{i=1}^{i=n} \varepsilon_i H_i$ to calculate elastic compression at point A
- $S_c = \sum_{i=1}^{i=n} S_{ci}$ to calculate consolidation compression at point A
- The **elastic compression of the pile**, S_p, is also calculated.
- The **total settlement** is given by $\Sigma S = S_e + S_c + S_p$'dir.

If the "Use of equivalent modulus of deformation in rigid column zone" option is selected or if soil improvement is carried out using rigid columns:

- The soil profile to which the calculation point belongs is copied, and the zones interacting with the columns are **re-layered**. The layers within this zone are defined with the **soil name** and the **rigid column type**. Composite soil parameters (**E_{com}, ν_{com}**) are assigned to these layers, and the analysis continues using the modified soil profile.

$$E_{com} = [(A - nA_c)E_s + nA_cE_c]\frac{1}{A} \tag{38}$$

Equation (38) is used to determine the **modulus of elasticity of the composite layer**. In this method, the **deformation of the soil and columns** within the composite medium is assumed to be equal (Garassino, 1997). Here, $\mathbf{E_c}$ is the modulus of elasticity of the pile material in pile foundations or the deformation modulus of the soil–additive mixture in soil-improvement columns.

- The **stress increase** at the mid-level of each sub-layer is calculated using the **Boussinesq method**. If the option **“Swell on excavation base”** is selected, the weight of the excavated soil is subtracted, and $\mathbf{q = q_{net}}$. In consolidation analyses, $\mathbf{q_{net}}$ is always considered.
- The **effective stress** is calculated at the mid-level of each sub-layer.
- The **elastic vertical strain** at the mid-level of each sub-layer is calculated using equations (39).

$$\Delta\sigma_i = \Delta\sigma_v - \nu.(\Delta\sigma_{hx} + \Delta\sigma_{hy}) \text{ and } \varepsilon_i = \frac{\Delta\sigma_i}{E_i} \tag{39}$$

The **elastic compression** of each sub-layer is determined by $\mathbf{S_{ei} = \varepsilon_i \cdot h_f}$. Mechanical properties are taken from the corresponding soil layer, and depending on the drainage condition, either **total** or **effective stress parameters** are used.

- The **consolidation compression** of sub-layers within undrained layers below the rigid column zone is also calculated. Equations (32), (33), (34), and (35) are used with the acting effective stress and stress increase values.
- In comparing the equations, the **preconsolidation pressure $\boldsymbol{\sigma'_c}$** of each sub-layer is obtained from its **depth, effective stress**, and the $\boldsymbol{\sigma'_c}$ value of the main layer it belongs to.

- $S_e = \int_0^H \varepsilon dH = \sum_{i=1}^{i=n} \varepsilon_i H_i$ to calculate elastic compression at point A

- $S_c = \sum_{i=1}^{i=n} S_{ci}$ to calculate consolidation compression at point A

- The **total settlement** $\Sigma S = S_e + S_c$’dir.

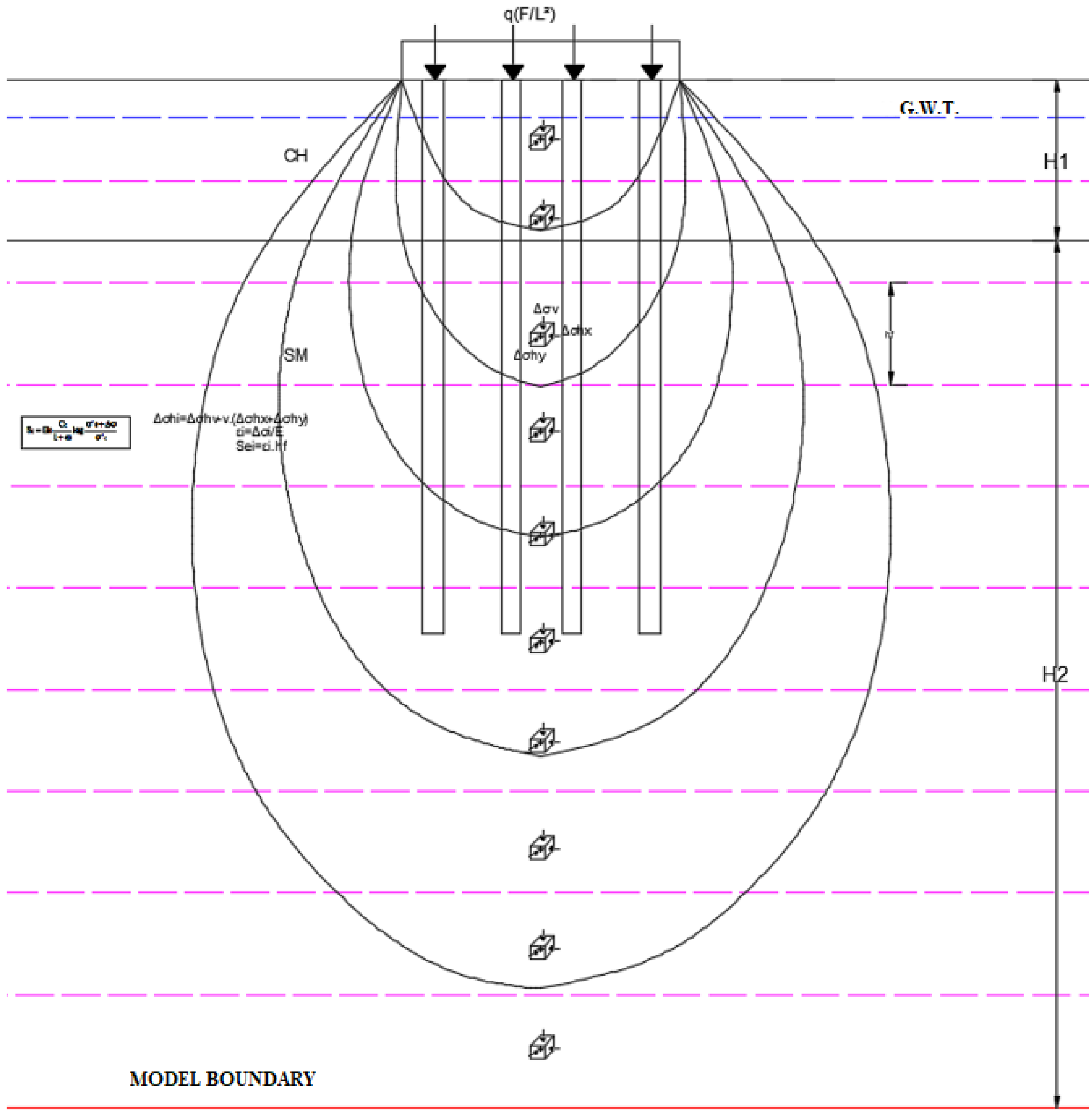


Figure 95. Stress Transfer from Rigid Column Group to Soil Using the Boussinesq Method

(Composite mechanical properties are used in the column–soil zone)

16.4. Bearing Capacity of Shallow Foundation

The bearing capacity of shallow and deep foundations is calculated in SETAF2018. The Terzaghi, Meyerhof, and Vesic methods are employed for the analysis of shallow footings. The weighted average of the soil parameters for the layers down to a depth equal to the footing width *B* is used in the calculation.

The capacity of deep foundations is calculated using the α, β, and λ methods. Group efficiency is calculated by the geometric Converse Labarre and Terzaghi block approaches.

16.4.1. Terzaghi Method

Bearing capacity in shallow footings is calculated by the formula:

$$q_{ult} = c.N_c.s_c + q.N_q + 0.5.\gamma.B.N_\gamma.s_\gamma \tag{40}$$

Effective B′ is used in the third term.

Definitions:

- Cohesion c : c′ for effective stress analysis, $S_u(c_u)$ for UU (undrained) conditions
- N_c: **Bearing capacity factor**. Calculated with ϕ′ of the soil layer directly beneath the footing for effective stress analysis, ϕ = 0 for total stress analysis under UU conditions.
- N_q: **Bearing capacity factor**. Calculated with ϕ′ of the soil layer above the footing through the embedment depth ($D_f \leq B$) for effective stress analysis, ϕ = 0 for UU (total stress) conditions.
- N_γ: **Bearing capacity factor**. Calculated with ϕ' of the soil layer directly under the footing for effective stress analysis, ϕ = 0 for UU (undrained) total stress conditions.
- s_c , s_γ: Shape factors for the footing
- D_f: Depth of embedment of the footing
- q: **Effective stress at the base of the footing** for effective stress analysis; **total stress at the base** for undrained analysis.
- γ: **Weighted average unit weight** of the soil below the footing, taken over a depth equal to the foundation width B

Terzaghi's method provides the bearing capacity of foundations under vertical loading, while allowing the consideration of eccentricity in two directions. As a condition, **D/B ≤ 1** is adopted — that is, the embedment depth D_f cannot exceed the footing width B. During the calculation of q, this limitation is also taken into account. Shape factors are calculated using the corrected B′ and L′ dimensions for rectangular footings. The unit weight γ in the third term is adjusted according to the position of the **Groundwater Table (GWL)**. When **ϕ = 0**, $\mathbf{N_\gamma = 0}$ and $\mathbf{N_q = 1}$. If the **cohesion c = 0**, the first term of the equation becomes zero.

In the undrained c–ϕ condition, the second term uses $\mathbf{q = q_{total}}$, which is the total stress at the base of the footing. The relation $\mathbf{D/B \leq 1}$ is considered when calculating q_{total}. In the third term, the unit weight γ is taken as $\boldsymbol{\gamma_{sat}}$ (saturated unit weight).

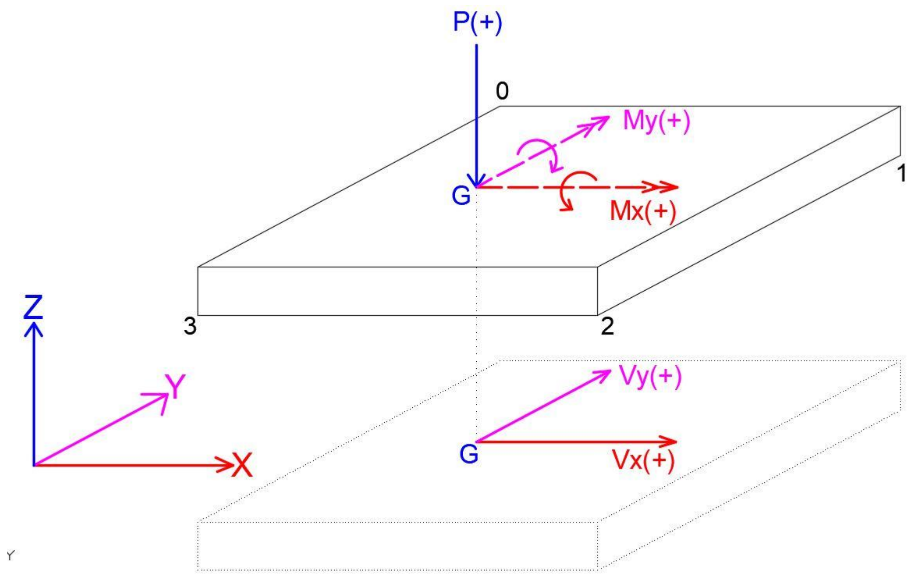


Figure 96. Loads Acting on Footing

The influence of the groundwater table is taken into account in the second term when calculating the effective stress at the base of the footing. In the third term, the γ value is corrected according to the groundwater level. Depending on the position of the GWL, the following cases are applied:

- Case 1: GWL = 0 (at the surface) → use γ′
- Case 2: 0 < GWL $\leq D_f$ → use γ′
- Case 3: $D_f \leq$ GWL ≤ B → use corrected unit weight calculated by Equation ((41)

$$\bar{\gamma} = \gamma' + \left(\frac{z - D_f}{B}\right)(\gamma - \gamma') \tag{41}$$

Eccentricity

L_x: Footing dimension in X direction

L_y: Footing dimension in Y direction

B: Short side of footing

L: Long side of footing

e_x : Eccentricity in X direction

$$e_x = \frac{M_y}{P} \tag{42}$$

e_y : Eccentricity in Y direction

$$e_y = \frac{M_x}{P} \tag{43}$$

$$L'_x = L_x - 2 \cdot e_x \tag{44}$$

$$L'_y = L_y - 2 \cdot e_y \tag{45}$$

$$B' = \mathrm{Min}(L'_x, L'_y) \tag{46}$$

$$L' = \mathrm{Maks}(L'_x, L'_y) \tag{47}$$

B′: Effective length of short side

L′: Effective length of long side

The effective pressure q_a is calculated by:

$$q_a = \frac{P}{(B' \times L')} \tag{48}$$

The ultimate bearing capacity q_{ult} is defined as q_t (characteristic capacity of footing) in accordance with the Turkish 2018 Earthquake Code.

$$q_t = \frac{q_k}{\gamma_{rv}} \tag{49}$$

Net bearing capacity:

$$q_{knet} = q_k - q \tag{50}$$

Safe (allowable) bearing capacity:

$$q_{tnet} = \frac{q_{knet}}{\gamma_{rv}} \tag{51}$$

If a fill is placed on the footing after construction:

$$q_a + q < q_t \tag{52}$$

If no backfill is made, as in basement excavations:

$$q_a < q_{tnet} \tag{53}$$

Both conditions must satisfy these inequalities. SETAF2018 computes the factor of safety for both cases. When calculating the safety factors, the average value q_{avg} defined by the user in the "Footing Loads" tab is used instead of q_a. These inequalities are valid for all other bearing capacity methods as well.

Terzaghi bearing capacity factors are calculated by the following expressions:

$$N_q = \left[\frac{e^{2.\left(\frac{3\pi}{4}-\frac{\phi}{2}\right).\tan(\phi)}}{2.\cos^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)} \right] \tag{54}$$

$$N_c = \left(N_q - 1\right).\cot\phi \tag{55}$$

$$N_\gamma = \frac{\tan\phi}{2}\left[\frac{K_{p\gamma}}{\cos^2\phi} - 1\right] \tag{56}$$

$$K_{p\gamma} = 3\tan^2\left(45 + \frac{\phi+33}{2}\right) \tag{57}$$

For rectangular footings, the shape factors are:

$$s_c = 1 + 0.2\left(\frac{B'}{L'}\right) \tag{58}$$

$$s_\gamma = 1 - 0.2\left(\frac{B'}{L'}\right) \tag{59}$$

For strip footings:

$$s_c = 1\ ,\ \ s_\gamma = 1 \tag{60}$$

For circular footings:

$$s_c = 1.3\ ,\ s_\gamma = 1.6 \tag{61}$$

16.4.2. Meyerhof Method

Meyerhof equation for bearing capacity has the form:

$$q_{ult} = c.N_c.s_c.d_c.i_c + q.N_q.s_q.d_q.i_q + 0.5.\gamma.B.N_\gamma.s_\gamma.d_\gamma.i_\gamma \tag{62}$$

Horizontal load as well as vertical load can be included. Eccentricity in two directions can be calculated. Condition $\mathbf{D_f/B \leq 1}$ should be observed when calculating q in the second term.

s_c, s_q, s_γ : Shape factors

d_c, d_q, d_γ : Depth factors

i_c, i_q, i_γ : Load inclination factors

P: Vertical load

The values B and L are used for effective stress q_a.

$$q_a = \frac{P}{(BxL)} \tag{63}$$

Bearing Capacity Factors

$$N_q = e^{\pi \tan\phi} \tan^2\left(45 + \frac{\phi}{2}\right) \tag{64}$$

$$N_c = \left(N_q - 1\right)\cot\phi \tag{65}$$

$$N_\gamma = \left(N_q - 1\right)\tan\left(1.4\phi\right) \tag{66}$$

Shape Factors

$$s_c = 1 + 0.2 K_p \frac{B}{L} \quad \text{(for any value of } \phi\text{)} \tag{67}$$

$$s_q = s_\gamma = 1 + 0.1 K_p \frac{B}{L} \qquad \phi>0 \tag{68}$$

$$s_q = s_\gamma = 1 \qquad \phi=0 \tag{69}$$

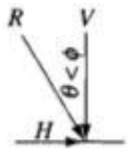
Depth Factors

$$d_c = 1 + 0.2\sqrt{K_p}\frac{D}{B} \qquad \text{any } \phi, \tag{70}$$

$$d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\frac{D}{B} \qquad \phi>0, \tag{71}$$

$$d_q = d_\gamma = 1 \qquad \phi=0, \tag{72}$$

Load Inclination Factors



$$i_c = i_q = \left(1 - \frac{\theta^\circ}{90^\circ}\right)^2 \quad \text{for any } \phi, \tag{73}$$

$$i_\gamma = \left(1 - \frac{\theta^\circ}{\phi^\circ}\right)^2 \qquad \phi>0 \tag{74}$$

$$i_\gamma = 0 \qquad \phi=0 \text{ in the case of } \theta>0 \tag{75}$$

$$K_p = \tan^2\left(45 + \phi/2\right) \tag{76}$$

θ: The angle between the load vector and the vertical. All i factors become 1.0 when θ = 0

The inclination of horizontal load is considered in one direction in the Meyerhof equation. In SETAF2018, the user may specify the direction of horizontal load and its inclination in the analysis settings. By default, the horizontal load **V_X** acting along the x-axis is used.

The use of the **Reduction Factor R_e** is recommended in the Meyerhof equation. This reduction accounts for the effect of eccentricity.

$$q_{ultazalt} = q_{ulttam} \times R_e \tag{77}$$

The reduction factor are taken as

$$R_e = 1 - \frac{2e}{B} \quad \text{(killer)} \tag{78}$$

$$R_e = 1 - \sqrt{\frac{e}{B}} \quad \text{(kumlar)} \tag{79}$$

for fine grained(cohesive), sands respectively at interval 0<e/b<0.3. Alternatively, the shape and depth factors may be calculated using **B′** and **L′**, and **B′** is used in the third term. In this case, the acting stress is **q_a = P / (B′ × L′)**. The user can adjust these settings in the **Analysis Settings** window of SETAF2018.

16.4.3. Vesic Method

The Vesic bearing capacity equation is expressed as the resultant of two horizontal forces (Bowles, 1996). Condition **D/B ≤ 1** applies when calculating q in the second term.

$$q_{ult} = c.N_c.s_c.d_c.i_c.g_c.b_c + q.N_q.s_q.d_q.i_q.g_q.b_q + 0.5.\gamma.B.N_\gamma.s_\gamma.d_\gamma.i_\gamma.g_\gamma.b_\gamma \tag{80}$$

$$\text{When } \phi=0 \rightarrow q_{ult} = 5.14s_u\left(1 + s_c^1 + d_c^1 - i_c^1 - b_c^1 - g_c^1\right) + \bar{q} \tag{81}$$

s_c, s_q, s_γ : Shape factors

d_c, d_q, d_γ : Depth factors

i_c, i_q, i_γ : Load inclination factors

g_c, g_q, g_γ : Ground slope factors

b_c, b_q, b_γ : Base slope factors

Bearing Capacity Factors

N_c: Identical with Meyerhof's

N_q: Identical with Meyerhof's

$$N_\gamma = 2\left(N_q + 1\right)\tan\phi \tag{82}$$

Shape Factors

$$s_{c(v)} = 1 + \frac{N_q}{N_c} \cdot \frac{B}{L} \tag{83}$$

$$s_{q(v)} = 1 + \frac{B}{L}\tan\phi \qquad \text{for all values of } \phi \tag{84}$$

$$s_{\gamma(v)} = 1 - 0,4\frac{B}{L} \qquad \geq 0,6 \tag{85}$$

Depth Factors

$$d_c^{'} = 0,4k \qquad (\phi=0) \tag{86}$$

$$d_c = 1 + 0,4k \tag{87}$$

$$k = D/B \quad \text{for } D/B \leq 1 \tag{88}$$

$$k = \tan^{-1}(D/B) \quad \text{for } D/B > 1 \tag{89}$$

k in radians.

$$d_q = 1 + 2\tan\phi(1 - \sin\phi)^2 k \tag{90}$$

$$d_\gamma = 1 \qquad \text{for all values of } \phi \tag{91}$$

Load Inclination Factors

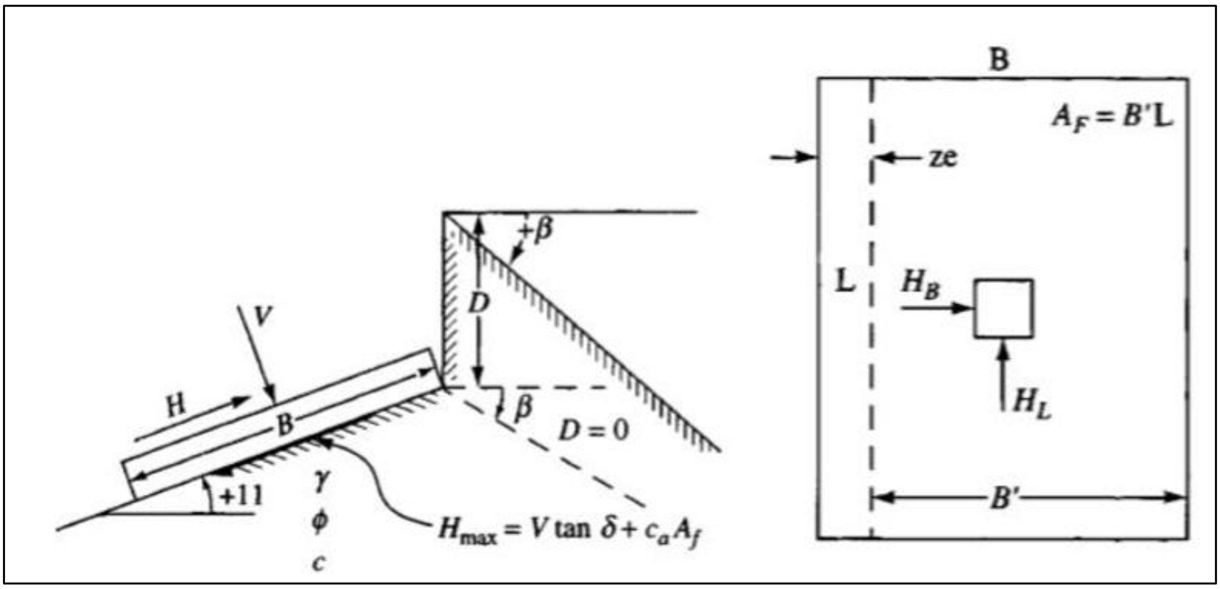


Figure 97. Loads and Geometry in Vesic Method

$$A_f = B'L' \tag{92}$$

$$i_c' = 1 - \frac{mH_i}{A_f\ c_a\ N_c} \qquad (\phi = 0) \tag{93}$$

$$i_c = i_q - \frac{1 - i_q}{N_q - 1} \qquad (\phi > 0) \tag{94}$$

$$i_q = \left[1 - \frac{H_i}{V + A_f\ c_a \cot\phi}\right]^m \tag{95}$$

$$i_\gamma = \left[1 - \frac{H_i}{V + A_f\ c_a \cot\phi}\right]^{m+1} \tag{96}$$

$$m = m_B = \frac{2 + B/L}{1 + B/L} \tag{97}$$

$$m = m_L = \frac{2 + L/B}{1 + L/B} \tag{98}$$

Ground slope factors

$$g_c' = \frac{\beta}{5{,}14} \quad \text{when } \phi = 0, \quad (\text{units of } \beta \text{ in radians}) \tag{99}$$

$$g_c = i_q - \frac{1 - i_q}{5{,}14 \tan\phi} \qquad (\phi > 0) \tag{100}$$

$$g_q = g_\gamma = (1 - \tan\beta)^2 \tag{101}$$

Base slope factors

$$b_c' = g_c' \qquad (\phi = 0) \tag{102}$$

$$b_c = 1 - \frac{2\beta}{5{,}14 \tan\phi} \tag{103}$$

$$b_q = b_\gamma = (1 - \eta \tan\phi)^2 \tag{104}$$

B and L are used to obtain shape and depth factors.

16.4.4. Bearing Capacity in Sands with SPT

Shallow bearing capacity in sand is calculated using the SPTN values recorded in the soil profile. The allowable bearing capacity $\boldsymbol{\sigma_{em}}$ (in kPa) is given by:

$$\sigma_{em} = q_t = \frac{N_1}{0.05} \qquad (B \le 1{,}2\text{m}) \tag{105}$$

$$\sigma_{em} = q_t = \frac{N_1}{0.08}\left(\frac{B + 0{,}3}{B}\right)^2 \left[1 + 0.33\frac{D_f}{B}\right] \qquad (B > 1{,}2\text{m}) \tag{106}$$

The average N1 value obtained over a depth of **6B** in the soil profile is used for the calculation.

16.5. Bearing Capacity of Deep Foundations

The procedure explained in this section is not limited to conventional pile foundations. It also applies to other deep foundation systems such as **micropiles**, **concrete piles**, **jet grouting columns**, and **deep mixing columns**. These systems similarly develop bearing capacity through **tip** and **skin resistance**. **SETAF2018** evaluates bearing capacity based on the **geometrical** and **mechanical** properties of these elements, following similar analytical principles.

The capacity of deep foundations is calculated using the **α**, **λ**, and **β** methods. The general expression for pile capacity is given by:

$$Q_u = Q_{çev} + Q_{uç} - W_p \tag{107}$$

Q_u: Total capacity of floating rigid column

Q_s: Skin resistance

Q_{tip}: Tip resistance

W_p: Net weight of the rigid column (after deducting buoyancy under groundwater table)

Skin resistance components are calculated separately for each soil layer and summed using the α, λ, and β methods. The symbols used in the result tables and reports are defined in accordance with the Eurocode 7 standard or the 2018 Turkish Earthquake Code, depending on the user's selection in the analysis settings.

A rigid column embedded in the soil must displace by a certain amount to mobilize both **skin** and **tip** resistance. Field tests have shown that full mobilization of skin resistance requires a penetration of **2.5 to 10 mm**, whereas full mobilization of tip resistance requires a displacement of approximately **8 – 19% of the column diameter**. Complete skin friction is generally mobilized at about **one-tenth** of the displacement required for full tip resistance.

16.5.1. Alpha Method

The "α" method is used to calculate the **short-term bearing capacity** of rigid columns. The contribution of the selected **undrained soil layers** to the total capacity is determined using the α method.

Skin Resistance

$$f_s = \alpha \times S_u \tag{108}$$

$$\alpha = 0,5\ \psi^{-0,50} \qquad (\psi \le 1) \tag{109}$$

$$\alpha = 0,5\ \psi^{-0,25} \qquad (\psi > 1) \tag{110}$$

$$A_s = \pi \times D \times L \tag{111}$$

$$Q_{çev} = f_s \times A_s \tag{112}$$

f_s: Unit skin resistance

s_u: Undrained shear strength

α: Adhesion factor

ψ: S_u/σ'_0 ratio at the depth of calculation

A_s: Lateral surface area of the column

$Q_{çev}$: Skin resistance

Tip Resistance

$$f_b = S_u \times N_c \tag{113}$$

$$Q_{uç} = \left(S_u \times N_c\right) A_b \tag{114}$$

$$f_b = S_u \times N_c \tag{115}$$

A_b: Column cross sectional area

f_b: Ultimate tip resistance

N_c: Bearing capacity factor

$Q_{uç}$: Tip resistance

16.5.2. Lambda Yöntemi

The contribution of the **undrained soil layers** to the total capacity is determined using the λ method. By default, the short-term analysis in **SETAF2018** uses the **"α" method**. To perform the calculation using the **λ method**, the user must select "λ Method" in the **Analysis Settings** window.

Skin Resistance

$$f_s = \lambda\left(\bar{\sigma}'_0 + 2\,\bar{c}_u\right) \tag{116}$$

$$\lambda = 0{,}178 - 0{,}016\left(\ln \pi_3\right) \quad \text{Normally consolidated clay} \tag{117}$$

$$\lambda = 0{,}232 - 0{,}0321\left(\ln \pi_3\right) \quad \text{Overconsolidated clay} \tag{118}$$

$$\pi_3 = \frac{\pi D f_{max} L_e^2}{A E_p \mu} \tag{119}$$

$$Q_{çev} = f_s \times A_s \tag{120}$$

f_{max}: Maximum friction resistance ≅ S_u

E_p: Elastic modulus of pile/column

D: Diameter of column

L_e: Effective length of pile in the soil

A: Cross-sectional area of the pile/column

μ: Required displacement to activate skin resistance (usually ≅ 3mm)

Tip Resistance

$$f_b = S_u \times N_c \tag{121}$$

$$Q_{uç} = \left(S_u \times N_c\right) A_b \tag{122}$$

16.5.3. Beta Method

"β" Method is employed to calculate the long-term (drained) capacity of a pile/rigid column. The contribution of drained layers to capacity is calculated with this method.

Skin Resistance

$$f_s = c' + \beta \sigma_0^1 \tag{123}$$

In fine-grained soil:

$$\beta = \left(1 - \sin\phi'\right).OCR^{0,5}.\tan\phi' \tag{124}$$

In coarse-grained soil, taking OCR=1:

$$Q_{çev} = f_s \times A_s \tag{125}$$

Tip Resistance

$$f_b = N_t.\sigma'_{0Df} \tag{126}$$

$$N_t = \left(\tan\phi' + \sqrt{1+\tan^2\phi'}\right)^2 e^{2\psi_p.\tan\phi'} \tag{127}$$

$$Q_{uç} = f_b.A_b \tag{128}$$

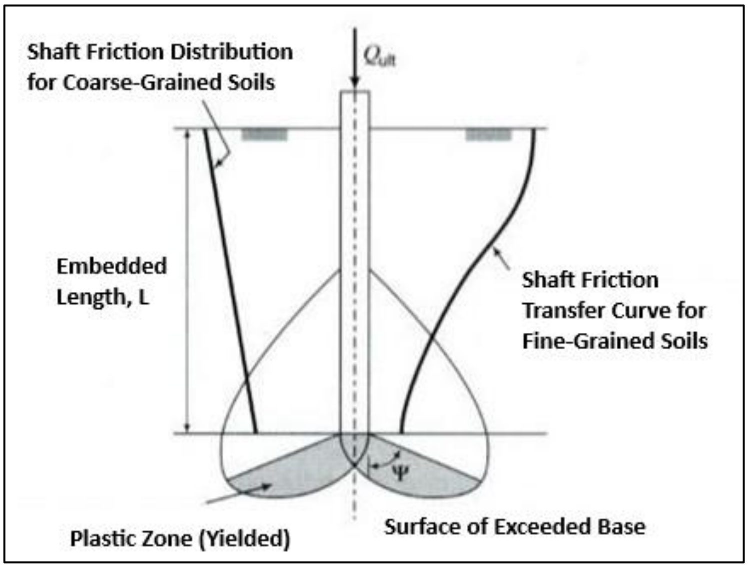


Figure 98. Transfer of Loads from Column to Soil

ψ_p: Plastification angle (Budhu, 2008)

In soft fine-grained soils: $\psi_p \leq \pi/3$

From soft fine-grained soils to dense coarse-grained soils and overconsolidated fine-grained soils: $\pi/3 \leq \psi_p \leq 0{,}58\pi$

In dense coarse-grained soils, the value of ψ should not exceed $\pi/2$.

16.5.4. Capacity of Pile Groups

In **SETAF2018**, the bearing capacity of pile groups is calculated using either the **Geometric approach** or the **Terzaghi Block Behaviour** method. Group capacity is expressed as:

$$Q_{U,group} = Q_u.n_p.E_g \tag{129}$$

$Q_{u,group}$: Ultimate group capacity

n_p: Number of columns

E_g: Group efficiency

Group efficiency $\mathbf{E_g}$ according to the **Converse-Labarre formula**:

$$E_g = 1 - \theta \frac{(n-1)m + (m-1)n}{90mn} \tag{130}$$

$$\theta = \arctan\left(\frac{D}{s}\right) \tag{131}$$

In the Terzaghi block method, the piles and soil filling the spaces between piles are considered as a block of soil. Capacity is then calculated by using the resistance of the surface area of the block and the bearing capacity of the block base. The condition below must be satisfied:

$$Q_{u,group} \leq Q_u \times n_p$$

16.6. Determination of Subgrade Reaction Moduli

The **moduli of subgrade reaction** for **shallow** and **deep foundations** are obtained by different methods. For shallow foundations, the modulus is determined from the **average settlement** of the foundation, by using the **Vesic equation**, or from the **results of plate bearing tests**, applying appropriate **corrections for foundation dimensions**.

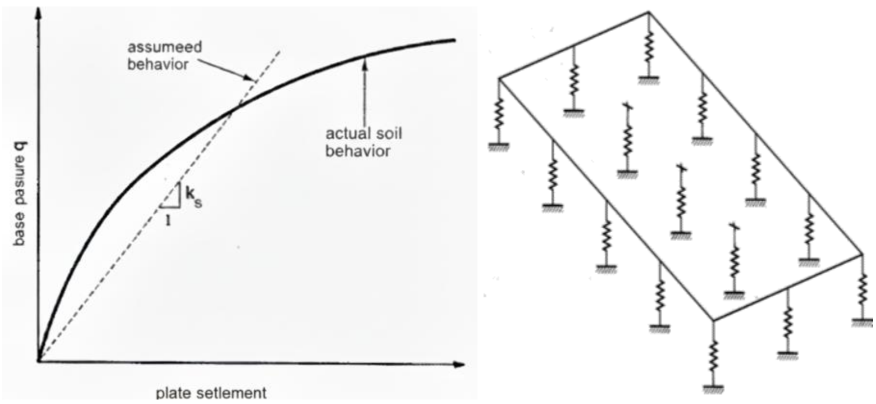


Figure 99. Model Foundation on Elastic Springs and Contact Pressure–Settlement Curve

For **pile foundations**, the **vertical modulus of subgrade reaction** is obtained from the **elastic settlement of a single pile**. The **horizontal modulus** is calculated from the existing **elastic coefficients**, which can be derived from theoretical relationships.

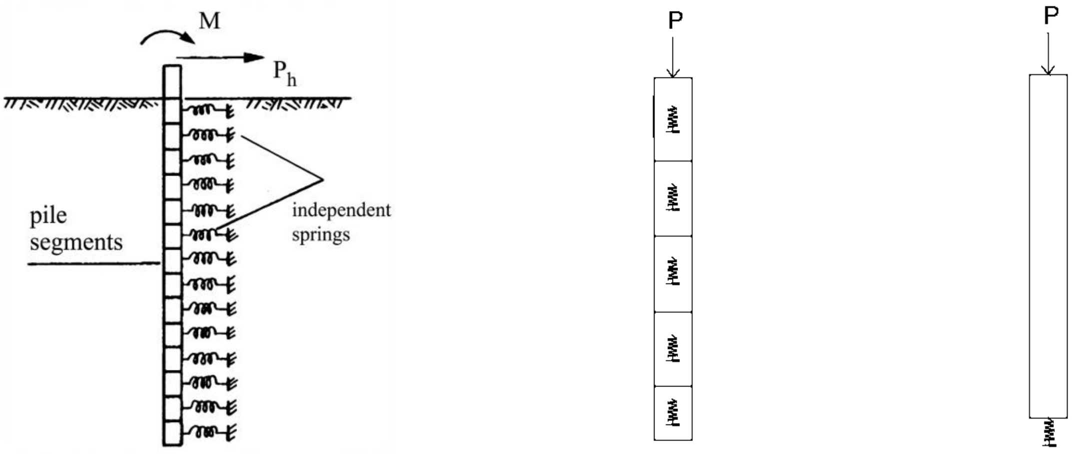


Figure 100. Springs Representing Lateral, Shaft, and Tip Resistance

16.6.1. From Settlements of Shallow Foundations

The **center of gravity** of the footing is determined according to its geometry — whether **rectangular**, **circular**, or **polygonal**. From this center of gravity, lines are drawn along the **x** and **y** axes within the foundation area, and **settlements** are calculated at selected points along these lines (Figure 101). For the points located on the **x-axis**, the **vertical modulus of subgrade reaction** is calculated using the following relation:

$$k_v = \frac{q}{\Delta H} \tag{132}$$

Where **q** is the **contact pressure** at the base of the foundation, and **ΔH** is the **elastic settlement**. The **average modulus of subgrade reaction in the x direction** is obtained by taking the mean of the moduli computed at the points along the x-axis. Similarly, the **average modulus in the y direction** is determined from the settlements along the y-axis. Finally, either the **average** or the **minimum** of these two moduli (in x and y directions) is adopted as the **average vertical modulus of subgrade reaction** for the foundation.

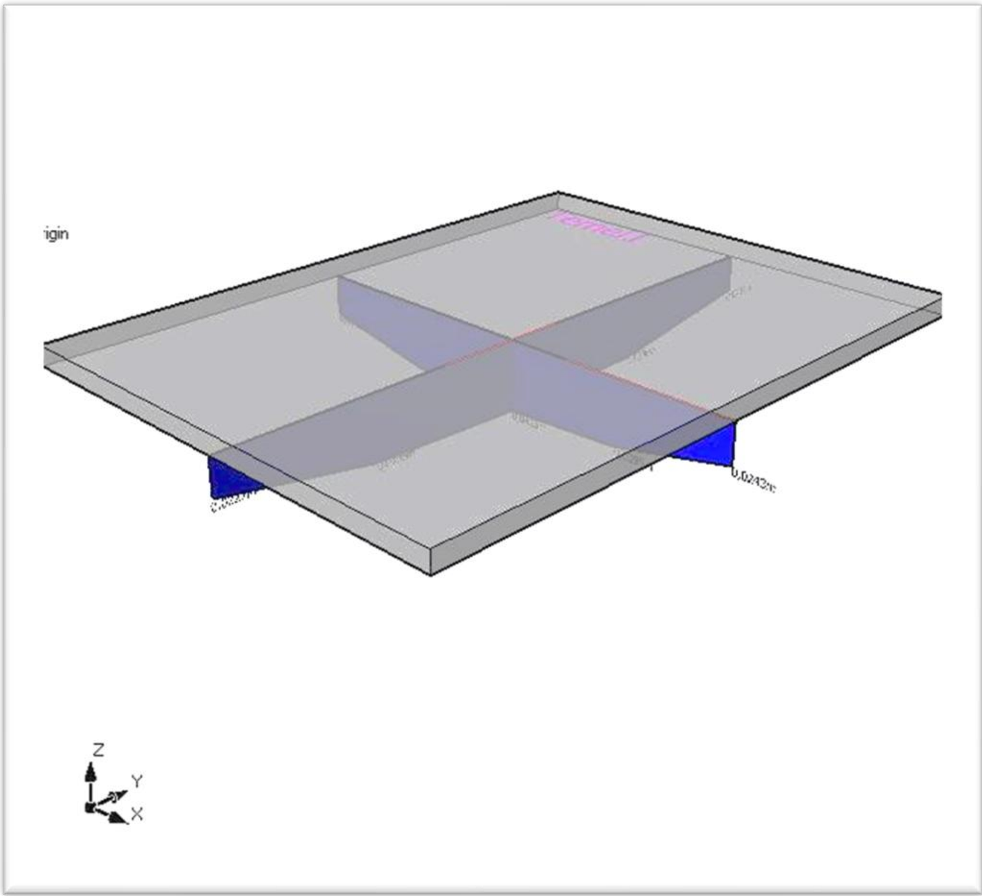


Figure 101. Settlement Profiles along Axes Passing through the Center of Gravity of the Footing

16.6.2. From Vesic Equation

Derivation using the theory of elasticity:

$$k_s = \frac{0.65}{B} \sqrt[12]{\frac{E_s B^4}{E_b I_b}} \frac{E_s}{1-\nu^2} \tag{133}$$

The **vertical modulus of subgrade reaction** for a shallow foundation can be calculated using **Vesic's equation**. Here, **B** is the **width of the footing**, $\mathbf{E_s}$ is the **elasticity modulus of soil**, $\mathbf{E_b}$ is the **elasticity modulus of the foundation (concrete)**, **ν** is **Poisson's ratio of the soil**, and $\mathbf{I_b}$ is the **second moment of area of the foundation plate**.

16.6.3. From Plate Loading Test

The **user** can enter the **diameter of the plate**, **bearing capacity**, and **modulus of subgrade reaction** in the **"Properties of Footing"** window. The **vertical modulus of subgrade reaction (k_v)** for the footing is derived from the **$k_{platediameter}$** value by applying a **correction for the actual footing size**. For example, in **sands**, the subgrade modulus **$k_{0.3}$**, measured in a **305 mm diameter plate loading test**, is converted to the actual footing size by the following expression:

$$k_s = k_{0.3}\left(\frac{B+0.3}{2B}\right)^2 \tag{134}$$

and, similarly, in **clays**, the correction is made by:

$$k_s = k_{0.3}\left(\frac{0.3}{B}\right) \tag{135}$$

16.6.4. Vertical Subgrade Modulus for Skin Resistance in Rigid Columns

For a **single rigid column**, the portion of the total load carried by **skin (shaft) resistance** is obtained from the ratio **Q_s / Q_{total}**. The **settlement** of this single column is calculated assuming that the entire load is transferred through **skin resistance**, following **Geddes' approach**. The **vertical modulus of subgrade reaction** for the **elastic spring** representing the load transfer by skin resistance is given by:

$$P_s = P.(Q_{çev} / Q_u) \tag{136}$$

$$\tau_s = \frac{P_s}{\pi.D.L} \tag{137}$$

$$k_{vs} = \frac{\tau_s}{\Delta H_s} \tag{138}$$

Here:

Q_s: Skin resistance offered by the surface of the rigid column

Q_u: Ultimate capacity of the pile / rigid column

P: Axial service load of the rigid column within a group

P_s: Portion of the service load transferred through surface resistance

τ_s: Surface shear stress

ΔH_s: Elastic settlement of a single rigid column under load P_s

k_{vs}: Vertical modulus of subgrade reaction for skin resistance

16.6.5. Vertical Subgrade Modulus for Tip Resistance in Rigid Columns

The **load transferred by tip (end bearing) resistance** of a single rigid column is obtained from the ratio **Q_p / Q_{total}**. This load is assumed to be fully carried through the **tip**, and the **settlement** of the single column is calculated accordingly. The **vertical modulus of subgrade reaction** for the **elastic spring** representing load transfer through tip resistance is given by:

$$P_p = P.\left(1 - \frac{Q_{çev}}{Q_u}\right) \tag{139}$$

$$\sigma_p = \frac{P_p}{\left(\pi.D^2/4\right)} \tag{140}$$

$$k_{vp} = \frac{\sigma_p}{\Delta H_p} \tag{141}$$

Q_s: Surface (skin) resistance of the rigid column

Q_u: Ultimate capacity of the rigid column

P: Maximum service axial load of the rigid column within the group

P_p: Portion of the service load transferred through the tip

σ_p: Tip stress of the rigid column

ΔH_p: Elastic settlement of a single rigid column under load Pp

k_{vp}: Vertical modulus of subgrade reaction for tip (end bearing) resistance

16.6.6. Horizontal Modulus of Subgrade Reaction for Rigid Columns

The **horizontal modulus of subgrade reaction** for a **rigid column** is calculated based on the **oedometric moduli** of the soil layers and the **bending stiffness** of the column.

The **oedometric moduli** of all soil layers are computed using the **effective Poisson's ratio** and the **shear modulus** of each layer, as follows:

$$E_{oed} = \frac{2.(1-\nu).G}{(1-2.\nu)} \tag{142}$$

E_{oed}: Oedometric modulus

ν': Effective Poisson's ratio

G: Shear modulus

The horizontal modulus of subgrade reaction is then expressed as:

$$k_h = 2,1\left(\frac{E_{oed}^{4/3}}{E_p.I_p^{1/3}}\right) \tag{143}$$

E_p: Modulus of elasticity of the rigid column

I_p: Moment of inertia of the rigid column

k_h: Horizontal modulus of subgrade reaction

16.7. Soil Improvement

Rigid columns may be used to improve the soil conditions beneath **shallow foundations**. Rigid columns formed by **Deep Mixing** or **Jet Grouting** techniques are considered in this method. The **settlement analysis** of deep mixing and jet grouting columns is carried out according to the procedures described in **Section 16.3**. The **deformation modulus (E_{com})** of the improved zone is obtained by comparing the **total cross-sectional area of the rigid columns** with the **total area to be improved**. The **composite modulus of elasticity** of the treated soil is given by:

$$E_{com} = [(A-nA_c)E_s + nA_cE_c]\frac{1}{A} \tag{144}$$

where:

A: Cross-sectional area of the soil block to be improved

n: Number of rigid columns

A_c: Cross-sectional area of a rigid column

E_s: Modulus of elasticity of the soil

E_c: Modulus of elasticity of the rigid column

E_{com}: Composite modulus of elasticity of the soil improved with rigid columns

A new **soil profile** is then established by inserting the rigid columns. The **improved layers** are assigned their corresponding **E_{com}** values, and the **settlement analysis** is performed in accordance with the principles outlined in **Section 16.3**.

16.7.1. Deep Mixing Method

The **deep mixing method** can be applied in two forms: **dry mixing** and **wet mixing**. **Dry mixing** is used in **saturated soils**, where **cement powder** is directly mixed with the in-situ soil. **Wet (or slurry) mixing** is applied in soils of **any degree of saturation**, where the natural soil is mixed with a **cement–water slurry**. In soils improved with these rigid columns, the **compressibility decreases,** and the **shear strength increases**.

The proportions of **cement** and **slurry** used in the mixture are determined according to the **Federal Highway Administration Design Manual: Deep Mixing for Embankment and Foundation Support (FHWA)**. The **phase diagrams** for the deep mixing process are illustrated in **Figure 102**. In these diagrams:

- V_a = Volume of air
- $V_{w,soil}$ = Volume of water in the soil before mixing
- $W_{w,soil}$ = Weight of water in the soil before mixing
- V_s = Volume of the soil solids
- W_s = Weight of the soil solids
- V_b = Volume of the binder
- W_b = Weight of the binder
- $V_{w,slurry}$ = Volume of water in the slurry for wet mixing

- $W_{w,slurry}$ = Weight of water in the slurry for wet mixing
- $V_{w,mix}$ = Volume of water in the mixture
- $W_{w,mix}$ = Weight of water in the mixture

Aggregates of these quantities include the following:

- V_v = Volume of voids in the soil before mixing ($V_a + V_{w,soil}$)
- V_{soil} = Volume of soil before mixing ($V_s + V_{w,soil} + V_a$)
- W_{soil} = Weight of soil before mixing ($W_s + W_{w,soil}$)
- V_{slurry} = Volume of slurry before mixing ($V_b + V_{w,slurry}$)
- W_{slurry} = Weight of slurry before mixing ($W_b + W_{w,slurry}$)
- V_{mix} = Volume of the mixture ($V_s + V_b + V_{w,mix}$)
- W_{mix} = Weight of the mixture ($W_s + W_b + W_{w,mix}$)

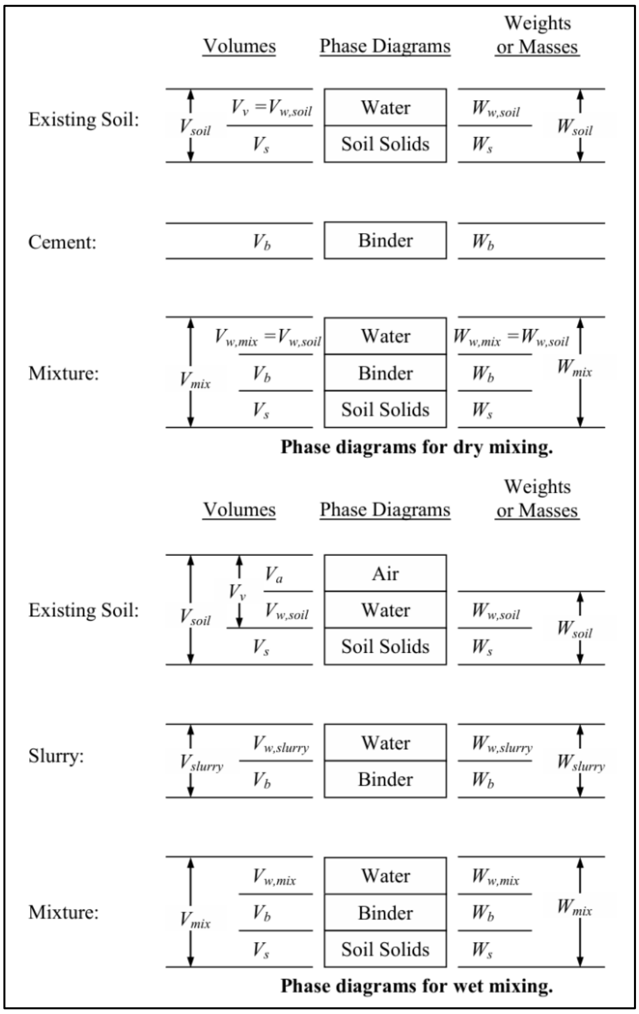


Figure 102. Phase Diagrams for Soil Mixing

The **specific gravities** are obtained from the following equations:

$$G_s = \frac{W_s}{V_s \gamma_w} \tag{145}$$

$$G_b = \frac{W_b}{V_b \gamma_w} \tag{146}$$

- G_s = Specific gravity of the soil solids
- G_b = Specific gravity of the binder
- γ_w = Unit weight of water (W_w / V_w).

The parameters useful for controlling **deep mixing operations** and for **reporting test results** on laboratory or field mixed samples are listed in **Table 2**.

Table 2. Definitions of Deep Mixing Parameters

Item	Applicability	
	Dry Method	Wet Method
Binder factor: $\alpha = \frac{W_b}{V_{soil}}$ (lb/ft^3 (kg/m^3))	Yes	Yes
Binder factor in-place: $\alpha_{in-place} = \frac{W_b}{V_{mix}}$ (lb/ft^3 (kg/m^3))	Yes	Yes
Binder content (percent): $a_w = \frac{W_b}{W_s}$	Yes	Yes
Total water-to-binder ratio (dimensionless): $w_T{:}b = \frac{W_{w,mix}}{W_b}$	Yes	Yes
Water-to-binder ratio of the slurry(dimensionless): $w{:}b = \frac{W_{w,slurry}}{W_b}$	No	Yes
Volume ratio (dimensionless): $VR = \frac{V_{slurry}}{V_{soil}}$	No	Yes

For dry mixing, the contractor controls the rate of delivery of dry binder during mixing, which means that the contractor is directly controlling the binder factor (α). For a saturated soil, as shown in Figure 102, α is related to the binder factor in-place (α in-place), binder content (a_W), and total water-to-binder ratio (w_T:b).

$$\alpha_{\text{in-place}}=\frac{\alpha\gamma_b}{\alpha+\gamma_b} \tag{147}$$

$$a_w=\frac{\alpha}{\gamma_{d,soil}} \tag{148}$$

$$w_T{:}b=\frac{w\gamma_{d,soil}}{\alpha} \tag{149}$$

γ_b = Unit weight of the binder solids (W_b/V_b).

$\gamma_{d,\ soil}$ = Dry unit weight of the soil (W_s/V_{soil}).

w = Water content of the soil ($W_{w,soil}/W_s$).

For **dry mixing**, the **total water-to-binder ratio (wT:b)** is selected from Figure 103 based on the **required column strength** of the project. Using the parameters **binder specific gravity (G_b)**, **w_T:b**, and **dry unit weight of the soil ($\gamma_{d,soil}$)**, the factors **α**, **$\alpha_{in\text{-}place}$**, and **a_w** are computed from **Equations (147)–(149)**.

The **weight of binder (Wb)** to be added in DSM production is determined by: $W_b = V_{soil} x\alpha$

For example, for a **DSM column** with a **diameter of 0.60 m** and a **length of 2 m** in a **clay layer**, the soil volume is: $V_{soil}=(\pi x0.6^2/4)x2=0.282m^3$

The **unit weight of the mixture (column)** obtained from DSM production is calculated as:

$$\gamma_{mix}=\frac{\gamma_b\left(\gamma_{soil}+\alpha\right)}{\gamma_b+\alpha} \tag{150}$$

Equation (150) represents the **total unit weight of the saturated mixture** obtained using **dry mixing**.

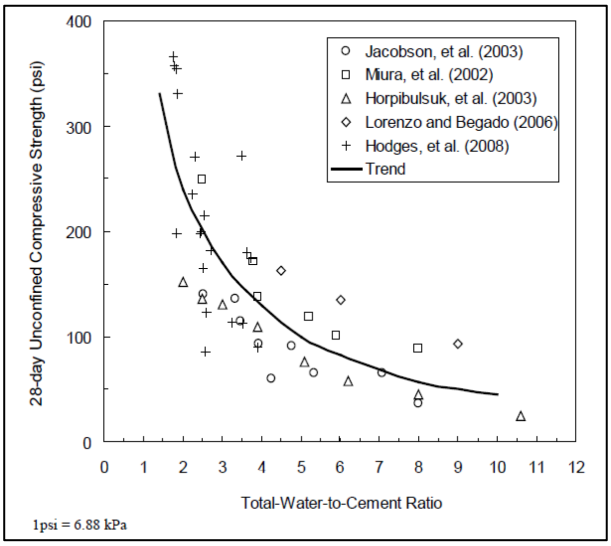


Figure 103. Relationship between Total Water-to-Binder Ratio and Unconfined Compressive Strength

For **wet mixing**, the **contractor** controls the **water-to-binder ratio (w:b)** of the slurry and the **volume ratio (VR)** of the slurry injected into the soil. By controlling these parameters, all mixture proportions can be expressed in terms of $\boldsymbol{\alpha}$, $\boldsymbol{\alpha_{in\text{-}place}}$, $\mathbf{a_w}$, and $\mathbf{w_T}$**:b**, which define the composition of the treated soil in different ways. For a given **w:b** ratio, the contractor adjusts **VR** to achieve the desired values of **α**, **α_in-place**, **aw**, or **wT:b**. The corresponding **VR** values are obtained from **Equations (151)-(155)**:

$$VR = \frac{\alpha}{\gamma_{d,slurry}} \tag{151}$$

Equation (151) expresses the **volume ratio** in terms of the **binder factor**.

$$VR = \frac{\alpha_{in\text{-}place}}{\gamma_{d,slurry} - \alpha_{in\text{-}place}} \tag{152}$$

Equation (152) expresses the **volume ratio** in terms of the **binder factor in-place** for **S = 1** (fully saturated soil).

$$VR = \frac{S(1+wG_s)}{S+wG_s} \times \frac{\alpha_{in\text{-}place}}{\gamma_{d,slurry} - \alpha_{in\text{-}place}} \tag{153}$$

Equation (153) expresses the **volume ratio** in terms of the **binder factor in-place** for **any degree of saturation (S)**.

$$VR = \frac{\gamma_{d,soil}}{\gamma_{d,slurry}} a_w \tag{154}$$

Equation (154) expresses the **volume ratio** in terms of the **binder content** for **any S**.

$$VR = \frac{w\gamma_{d,soil}}{\left(w_T : b - w : b\right)\gamma_{d,slurry}} \tag{155}$$

Equation (155) expresses the **volume ratio** in terms of the **total water-to-binder ratio** for **any S**.

$\gamma_{d,\ slurry}$ = Dry unit weight of the slurry (W_b/V_{slurry}).

S = Degree of saturation of the soil ($V_{w,soil}/V_v$).

$$\alpha = \frac{\gamma_{d,slurry}\alpha_{in-place}}{\gamma_{d,slurry} - \alpha_{in-place}} \tag{156}$$

Equation (156) expresses the **binder factor** in terms of the **binder factor in-place** for **S = 1**.

$$\alpha = \frac{S\left(1 + wG_s\right)}{S + wG_s} \cdot \frac{\gamma_{d,slurry}\alpha_{in-place}}{\gamma_{d,slurry} - \alpha_{in-place}} \tag{157}$$

Equation (157) expresses the **binder factor** in terms of the **binder factor in-place** for any **S**.

$$\alpha = a_w \gamma_{d,soil} \tag{158}$$

Equation (158) expresses the **binder factor** in terms of the **binder content** for any **S**.

$$\alpha = \frac{w\gamma_{d,soil}}{w_T : b - w : b} \tag{159}$$

Equation (159) expresses the **binder factor** in terms of the total water-to-binder ratio for any **S**.

For **wet mixing**, the **total water-to-binder ratio (w_T:b)** is selected from **Figure 103** according to the **required column strength** of the project. The **slurry water-to-binder ratio (w:b)** is then determined.

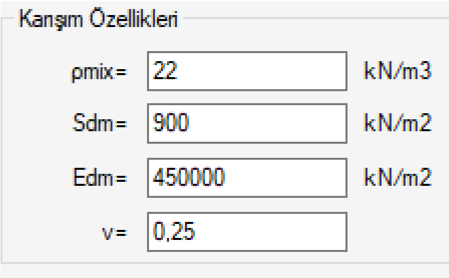
Using **G_b**, **$w_{T:b}$**, **w:b**, and **$\gamma_{d,soil}$**, the parameters **α**, **$\alpha_{in-place}$**, **a_w**, **VR**, and **$\gamma_{d,slurry}$** are calculated.

The **weight of binder** to be added in DSM production is again determined by: $W_b = V_{soil} x \alpha$.

The **unit weight of the mixture (column)** obtained from DSM production is given by:

$$\gamma_{mix} = \frac{\gamma_{soil} + VR\gamma_{slurry}}{1 + VR} \tag{160}$$

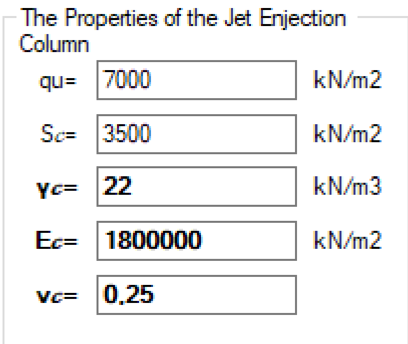
The DSM column properties, including ρ_{mix}, can be defined or modified by the user. These parameters may also be obtained using the **FHWA correlation tool** when desired.



These values are used in both **settlement** and **bearing capacity analyses**. Using the **elastic parameters E_{dm}** and **ν**, the **mechanical properties** of the **improved composite soil layers**, namely **E_{com}** and **ν_{com}**, are determined. The **mixture unit weight (ρ_{mix})** can be obtained using the **"Calculate"** button. The **allowable stress (S_{dm})** is computed as **$S_{dm} = q_u / 2$**. The **E_{dm} value** can be estimated using the **FHWA correlation**, also accessible via the **"Calculate"** button (**see Section 16.12**)

16.7.2. Jet Grouting Method (JG)

The **properties of the Jet Grouting (JG) columns** are specified by the user to be used in the analysis of the **improved composite soil layers** for determining their **mechanical properties (E_{com} and ν_{com})**.



The **unconfined compressive strength (q_u)** and the **modulus of elasticity (E_c)** can be obtained from the **correlations** (Section 16.12). The **shear resistance of the column (S_c)** is calculated as **$S_c = q_u / 2$**.

16.8. Negative Skin Friction

If the soil surrounding the pile, deep mixing, or jet grouting columns is highly compressible, it will, in the long term, tend to drag the column downward due to surface adhesion or friction. The stresses developed between the soil and the column surface are referred to as negative skin friction. This phenomenon typically occurs in normally consolidated or underconsolidated (soft) clays and in compressible fills surrounding the column. As a result, settlements of the column increase due to the additional load generated by the negative friction. In overconsolidated clays, where compressibility is low, negative skin friction is not expected to be mobilized, and therefore, no calculation is made for this condition.

Negative skin friction develops gradually as the surrounding soil consolidates under **drained conditions**. This process depends not only on the **relative settlements** but also on the **rate of consolidation**, **drainage conditions**, and **compressibility** of the different soil layers in the profile. After initial loading, the pile undergoes a **settlement (Δ)**, producing **drained deformation** in the surrounding soil. However, while the pile completes its movement, the surrounding soil does **not necessarily complete its own deformation** at the same time.

Part of the soil mass in contact with the pile may have already completed its consolidation corresponding to Δ, thus applying **positive skin friction** on the pile surface.

On the other side, the weak soil zones around the pile which have not yet reached full consolidation continue to compress well after the pile has completed its movement. This results in relative movement of the surrounding clay with respect to the stationary pile. **Negative skin friction** is activated due to this differential movement.

Consequently, along the pile length:

- **Positive skin friction** occurs in the lower sections, and
- **Negative skin friction** develops in the upper sections.

Proper modeling of this complex behavior requires consideration of the **geometry**, **variable drainage conditions**, **compressibility**, and **consolidation characteristics** of the soil layers.

Negative skin friction depends not only on the **type of soil** but also on the **differential settlements** that occur in various layers over time.

The designer should recognize that this effect is particularly significant when **soft clay or fill** overlies a **stiff overconsolidated clay** or a **rigid stratum**.

In **SETAF2018**, the **neutral plane** is determined to identify the **zone of negative skin friction**. The **long-term settlement** of the soil within this zone is computed. If the calculated settlement exceeds the **limiting value required to mobilize friction** (default value: **10 mm**, user-defined), the **negative skin friction force (P_n)** and the corresponding **additional settlements** are calculated. If the **"Negative Skin Friction"** option is not selected, this effect is **not considered** in the analysis.

16.8.1. Neutral Plane

A **rigid column** must move differently from the surrounding soil in order to become active.

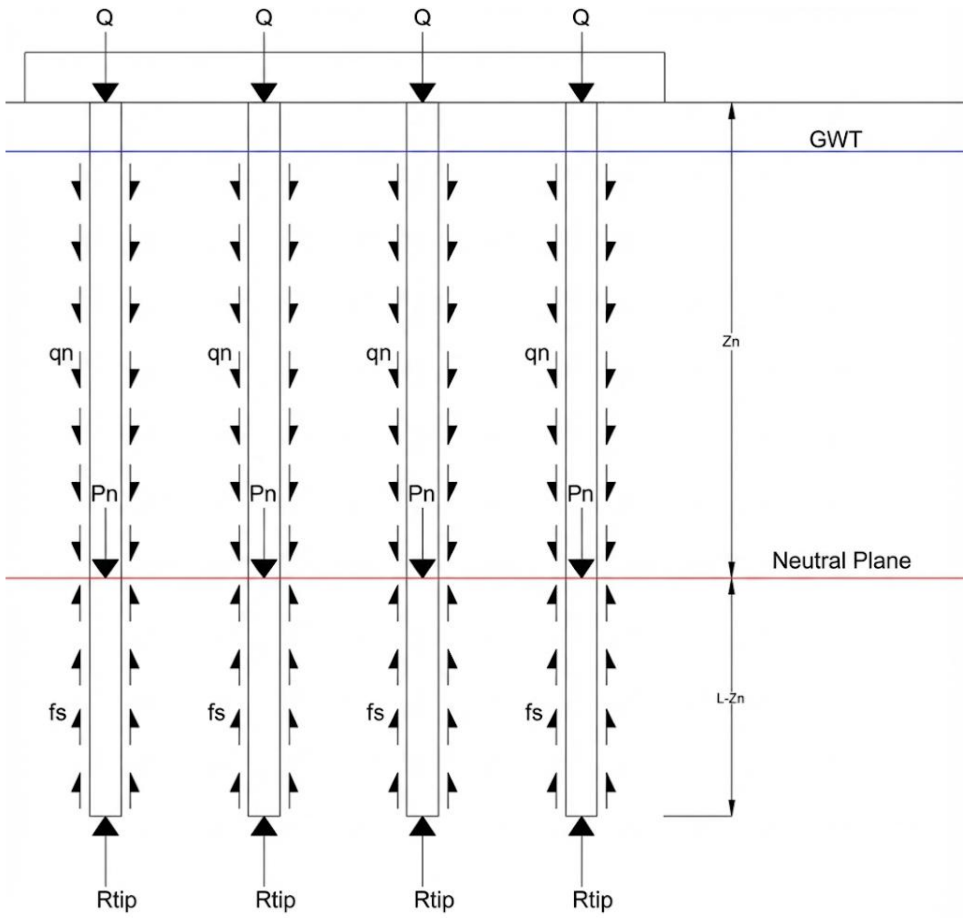


Figure 104. Negative Skin Friction in a Pile/Rigid Column Group

It is this **differential movement** that enables the **transfer of load** from the column to the soil (Figure 104). The **load on the column (Q)** increases with the **negative skin friction (q_n)** and reaches the **maximum load (Q_{max})** at the **neutral plane**. For each pile, **Q** is calculated from **P_{group}**.

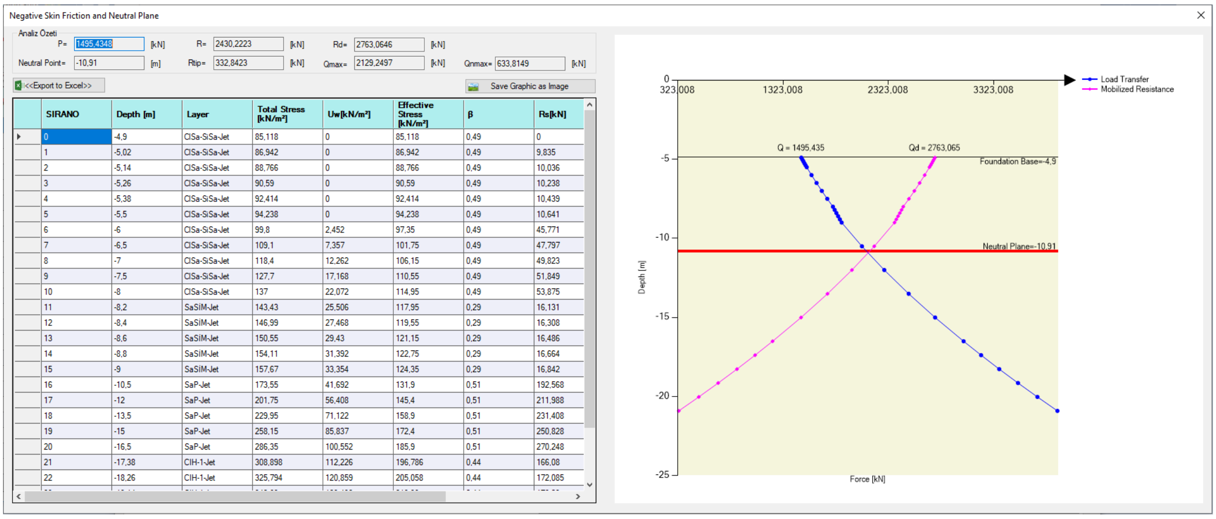


Figure 105. Determining the Level of Neutral Plane in a Rigid Column/Pile

The program constructs the **load transfer curves** acting on a rigid column or pile and determines the **location of the neutral plane**. The **Q_{max}** and **P_n** values at the level of the neutral plane are also calculated.

16.8.2. Settlements due to Negative Skin Friction

The **maximum downdrag force (Q_{max})** on the column appears at the level of the **neutral plane**. It represents the **sum of the load applied at the pile head** and the **downdrag force** induced by the **consolidating clay** or **compressing fill** surrounding the column. The steps of calculation are as follows:

- **Settlements** are calculated using one of the applied loads — **P_{grup}**, **$P_{groupnet}$**, **q**, or **q_{net}** — in accordance with the principles described in **Section 16.3.5**.
- The **neutral planes** are determined for all columns or piles.
- The **long-term compression** of the zone above the neutral plane is calculated. In evaluating the **column–soil interaction**, the **composite modulus of deformation** is used.

- If the calculated settlements are **smaller than the limiting value** required to fully mobilize negative skin friction, only the **mobilized portion** of the negative friction is considered. The **mobilization ratio** is obtained by dividing the compression of the negative friction zone by the limiting value. If the settlements are **equal to or greater than** the limiting value, the **P_n** values acting on the neutral plane are used. When the **applied column load (P)** exceeds the **bearing capacity (R_d)**, **P_n** is not calculated in the settlement analysis and the **neutral plane** is not determined — indicating that the column load exceeds its allowable capacity.
- The **long-term settlement (S_n)** of the column or pile group, with a length of **L–zn**, is calculated by converting the individual **P_n** values (or their sum) into an **equivalent distributed load**. The **stress transfer** corresponding to these settlements can be evaluated using either the **Mindlin–Geddes method** (Figure 106) or the **Boussinesq method** (Figure 107), taking into account the **conjugate modulus** within the **soil–column zone**. These long-term settlements are computed assuming **elastic behavior** under **drained conditions**. If the pile group or any individual column is found to be inadequate in terms of bearing capacity, the additional settlement due to negative skin friction is not calculated.

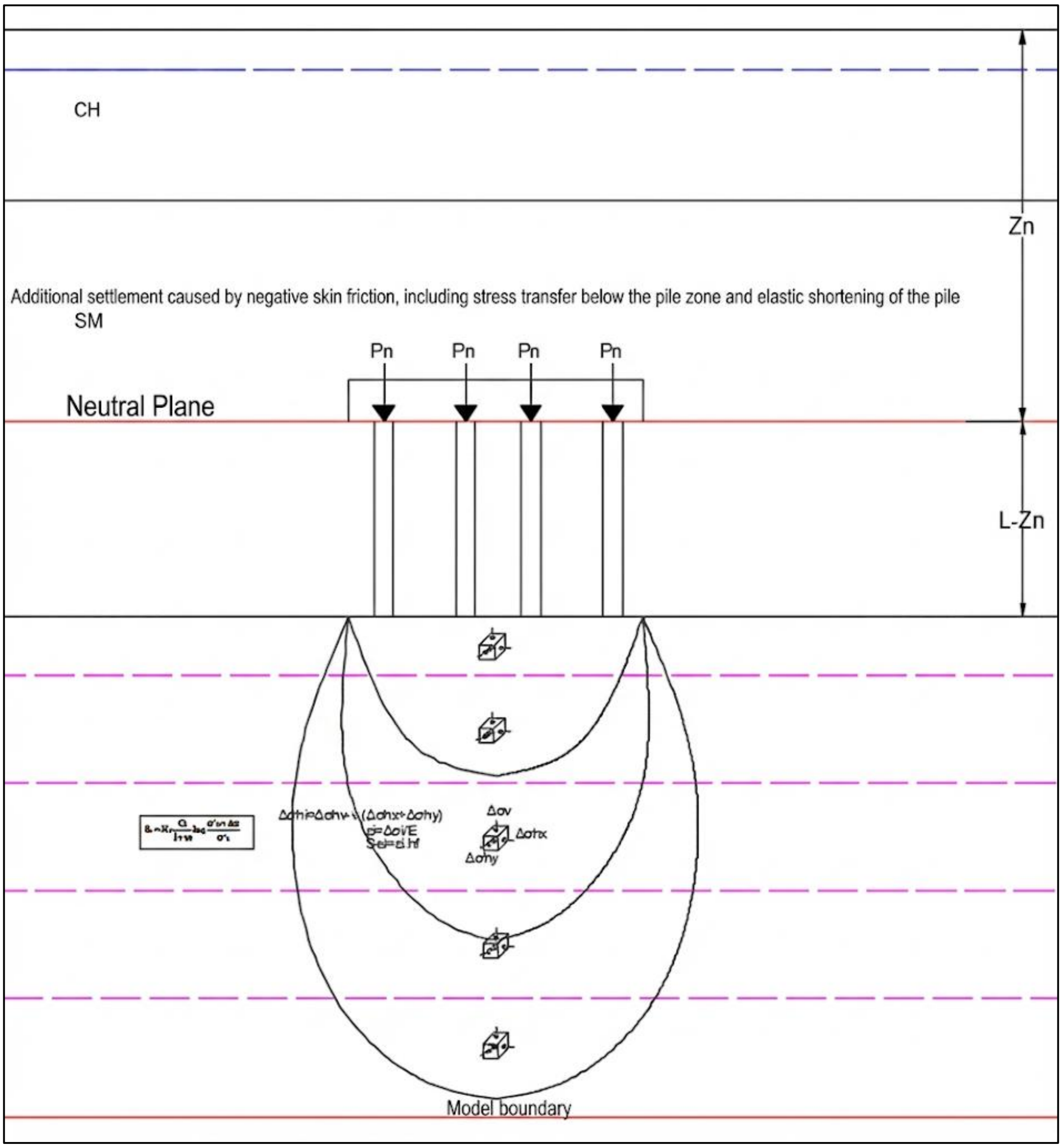


Figure 106. Additional Settlements due to Negative Skin Friction (Mindlin–Geddes Method)

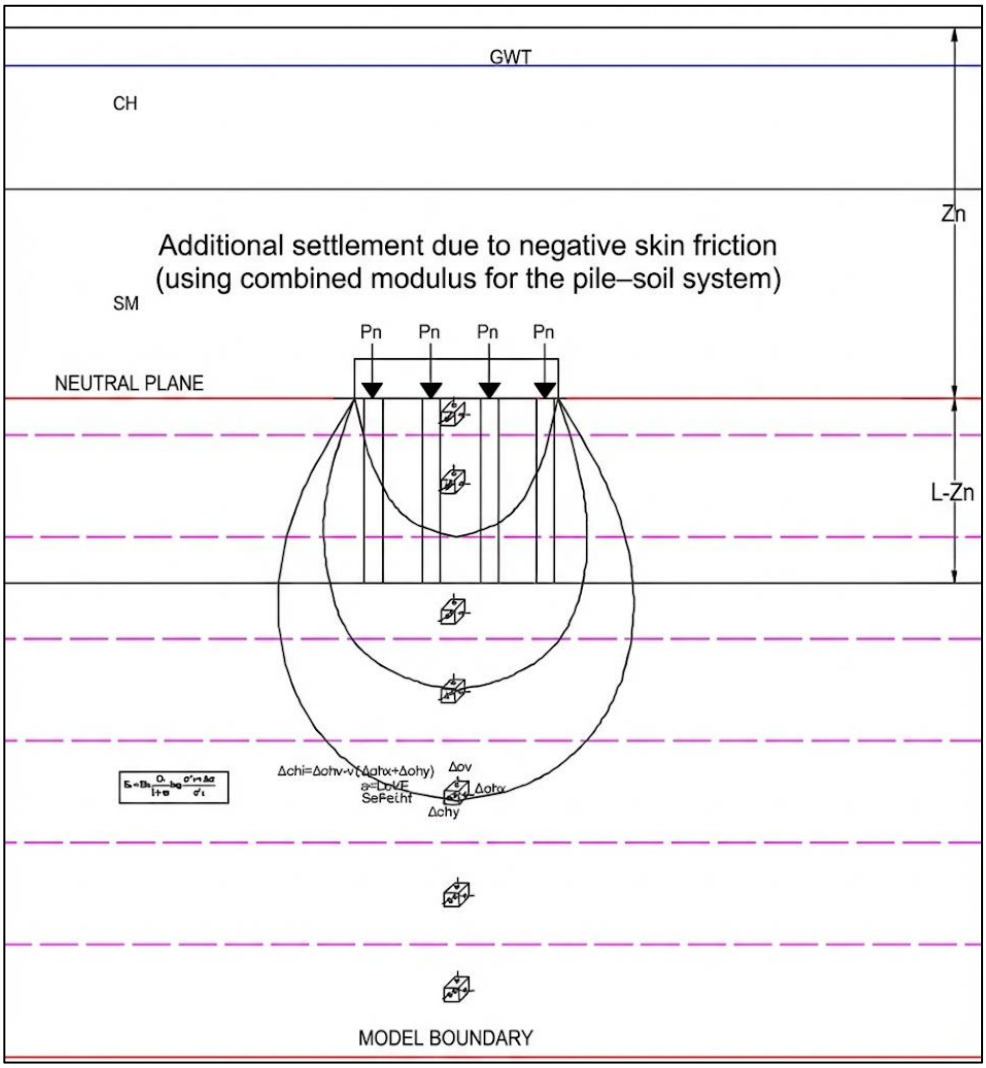


Figure 107. Additional Settlements due to Negative Skin Friction (Boussinesq Method with Conjugate Modulus in Soil–Column Zone

16.9. Pressuremeter (MPM) Corrections

The **soil properties** measured in the soil profile by the **pressuremeter test** are converted to **net limit pressures** using the following equation:

$$P_L^* = P_L - \sigma_h \tag{161}$$

where:

P_L: Limit preesure

σ_h: Horizontal total stress

P_L*: Net limit pressure

16.10. SPT Corrections

E_m: Efficiency of hammer

C_B: Borehole factor

C_S: Sampler factor

C_R: Length of rods coefficient

C_E: Energy ratio correction coefficient

z: Depth

σ'_{vo}: Vertical effective stress

SPTN: Number of hammer blows required to achieve a penetration of 300 mm, after the initial 150 mm seating penetration

C_N: Correction coefficient for overburden effect

N_1: Number of blows corrected with overburden correction coefficient

N_{160}: Corrected number of blows

The corrections to the SPT N-values are made using the following relations:

$$C_N = 9.78\sqrt{\frac{1}{\sigma'_{vo}}} \leq 1.70 \qquad (162)$$

$$N_1 = C_N \times SPTN \qquad (163)$$

$$N_{1,60} = \left(E_m \times C_B \times C_S \times C_R \times C_N \times SPTN\right)/0,60 \qquad (164)$$

$$C_E = E_m/0,60 \qquad (165)$$

16.11. Check for Liquefaction

The **liquefaction potential** in **sands and gravels** is determined according to the **Turkish Earthquake Code (TBDY 2018)**, whereas it is evaluated according to the **Adapazarı** and

Ishihara criteria for **fine-grained soils**. All points in an **SPT profile** are examined for liquefaction potential. The usual depth range for liquefaction assessment extends **from the groundwater table to a maximum depth of 20 m** below the ground surface.

16.11.1. Adapazarı Criterion

All of the following conditions must be satisfied for a **fine-grained soil (FC > 50%)** to liquefy:

- Liquidity index $I_L > 0,9$ or $(w_n/w_L)>0.9$
- Liquid limit $w_L \leq \%33$
- Plasticity index $I_p < \%15$
- Clay content $C < \%10$

According to the **Ishihara criterion**, if the **liquefiable layer** lies at a sufficient depth, its effect will **not reach the surface**. This can be evaluated using the **chart in** Figure 108, where the **thickness of the non-liquefying layer (H_1)** is plotted vertically and the **thickness of the liquefiable layer below (H_2)** horizontally. If the plotted point lies **on or beyond the M_w = 7.5 curve**, the likelihood of the liquefaction effect reaching the surface is **low or negligible**.

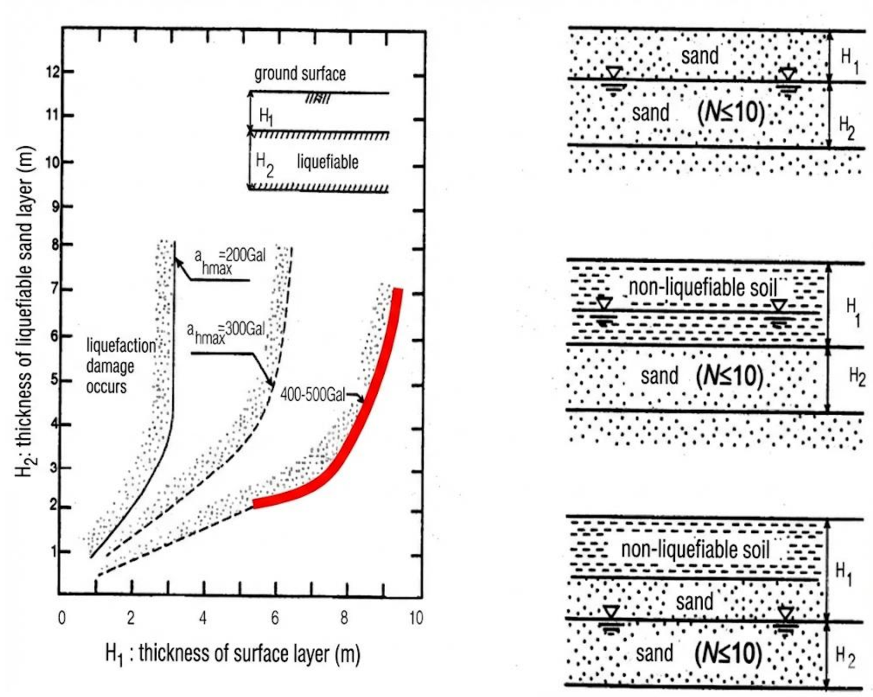


Figure 108. Ishihara Criterion – Definition of a Non-Liquefying Layer above a Liquefiable Layer

16.11.2. Liquefaction Check according to TBDY 2018

According to **TBDY 2018**, liquefaction may occur in **coarse-grained soils** when the following conditions are satisfied:

- $I_p \leq$ %12
- **Sand**, **gravelly sand**, **silty or clayey sand**, **non-plastic silts (NP)**, and **silt–sand mixtures**
- $N_{1,60} < 30$
- For sites classified as **Earthquake Design Class (DTS) = 4:**
    (a) Clay fraction C ≤ %20
    (b) Fines content FC ≤ %35 and $N_{1,60} < 20$ for **sandy soils**

16.11.3. Liquefaction Evaluation in Coarse-grained Soils

The liquefaction evaluation for coarse-grained soils is performed using the method described in **TBDY 2018, Appendix 16B**, based on the **SPT profile** of the selected borehole.

The **corrected SPT blow count (N_{160f})** is obtained according to the fines content:

$$N_{1,60f} = \alpha + \beta N_{1,60} \tag{166}$$

$\alpha = 0$; $\beta = 1$ $\quad (IDI \leq \%5)$

$\alpha = \exp\left[1{,}76 - \left(190/IDI^2\right)\right]$; $\beta = 0{,}99 + IDI^{1,5}/100$ $\quad (\%5 < IDI < \%35)$

$\alpha = 0{,}5$; $\beta = 1{,}2$

The **resistance to liquefaction** is then evaluated as:

$$\tau_R = CRR_{M7,5} C_M \sigma'_{v0} \tag{167}$$

The **cyclic resistance ratio (CRR)** corresponding to an earthquake of **moment magnitude Mw = 7.5** is determined from:

$$CRR_{M7,5} = \frac{1}{34 - N_{1,60f}} + \frac{N_{1,60f}}{135} + \frac{50}{\left[10N_{1,60f} + 45\right]^2} - \frac{1}{200} \tag{168}$$

The **magnitude correction factor** is applied as:

$$C_M = \frac{10^{2,24}}{M_w^{2,56}} \tag{169}$$

Shear stress created in the soil:

$$\tau_{deprem} = 0{,}65 s_{v0} \left(0{,}4 S_{DS}\right) r_d \tag{170}$$

Stress reduction factors:

$r_d = 1{,}0 - 0{,}00765z$	$z \leq 9{,}15$
$r_d = 1{,}174 - 0{,}0267z$	$9{,}15m < z < 30m$
$r_d = 0{,}744 - 0{,}008z$	$23m < z < 30m$
$r_d = 0{,}50$	$z > 30m$

The **factor of safety against liquefaction** is then checked from:

$$\frac{\tau_R}{\tau_{deprem}} \geq 1{,}10 \tag{171}$$

The **post-liquefaction settlement** at the surface is calculated using the method proposed by **Tokimatsu and Seed (1987)**.

16.11.4. Surface Settlement after Liquefaction

Sands subjected to earthquake loads tend to densify after the event. The compression of the lower layers is reflected at the ground surface in the form of **additional settlement**. **SETAF2018** calculates this settlement using the **Tokimatsu and Seed (1987)** method. The **unit volumetric strain (ε_h)** corresponding to different **SPT $N_{1,60}$** values is given as:

- $N_{1,60} = 2{,}5 \rightarrow \varepsilon_h = \%10$
- $N_{1,60} = 5 \rightarrow \varepsilon_h = \%5$
- $N_{1,60} = 7{,}5 \rightarrow \varepsilon_h = \%4$
- $N_{1,60} = 10 \rightarrow \varepsilon_h = \%3$
- $N_{1,60} = 14 \rightarrow \varepsilon_h = \%2$
- $N_{1,60} = 28 \rightarrow \varepsilon_h = \%1$

- $N_{1,60}$= 32 → ε_h= %0,5
- 2,5< $N_{1,60}$ <5 ε_h = -2 $N_{1,60}$ +15
- 5< $N_{1,60}$ <10 → ε_h = -0,4 $N_{1,60}$ +7
- 10< $N_{1,60}$ <14 → ε_h = -0,25 $N_{1,60}$ +5,5
- 14< $N_{1,60}$ <28 → ε_h = -0,071 $N_{1,60}$ +2,99
- 28< $N_{1,60}$ <32 → ε_h = -0,125 $N_{1,60}$ +4,5

Settlement at the surface:

$$S_i = \frac{\varepsilon_h}{100} \times \Delta z$$

$$S_t = \sum_{taban}^{YASS} Si$$

Δz: Effective soil thickness

S_i: Settlement value corresponding to SPTN

S_t: Total settlement at the ground surface

16.11.5. Liquefaction Evaluation on Improved Soils

The liquefaction resistance in soils improved with rigid columns — such as Jet Grouting (JG), Deep Soil Mixing (DSM), concrete or reinforced concrete piles, and micropiles — is evaluated using SPT test results.

According to **Baez (1995)**, rigid columns installed within the soil will experience **the same shear strain** as the surrounding soil during an earthquake, and the **earthquake-induced shear stresses** will be **shared** between the columns and the soil. Due to the stiffness contrast, a greater portion of the shear stress will be concentrated on the more rigid columns, as expressed by:

$$\gamma_s = \gamma_c \tag{172}$$

$$\frac{\tau_s}{G_s} = \frac{\tau_c}{G_c} \tag{173}$$

where:

γ_s: Shear strain in soil

γ_c: Shear strain in rigid column

τ_s: Shear stress in soil

τ_c: Shear stress in rigid column

G_s: Shear modulus of soil

G_c: Shear modulus of column

According to the **principle of force equilibrium**, under seismic loading, the total shear force acting on the **composite system (column + soil)** is equal to the sum of the shear forces in the soil and in the columns:

$$\tau_d A_t = \tau_s \times A_s + \tau_c \times A_c \tag{174}$$

τ_d: Shear stress generated by earthquake

A: Total area

A_s: Area of the soil

A_c: Total cross-sectional area of the columns

From these parameters, the **area ratio (a_r)** and **shear modulus ratio (G_r)** are calculated as:

$$a_r = \frac{a_c}{A} \tag{175}$$

$$G_r = \frac{G_c}{G_s} \tag{176}$$

Baez (1995) defined **a reduction factor K_g** as the ratio of the earthquake-induced shear stresses in the improved ground to those in the original untreated soil, representing the reduction in seismic shear stresses achieved by the installation of rigid columns against liquefaction.

$$K_g = \frac{\tau_s}{\tau_d} \tag{177}$$

Further studies defined the **seismic strain ratio (γ_r)**—the ratio of the column shear strain to the soil shear strain—and expressed the relationship for cases where soil and column strains are **not compatible** as:

$$\gamma_r = 1.04 G_r^{-0.65} - 0.04 \leq 1 \tag{178}$$

Analytically, the **reduction factors** can be determined as follows:

For the **compatible strain condition (γ_r = 1)**:

$$K_g = \frac{1}{G_r a_r + (1 - a_r)} \tag{179}$$

For the **non-compatible strain condition**:

$$K_g = \frac{1}{\gamma_r G_r a_r + (1 - a_r)} \tag{180}$$

In **SETAF2018**, both **compatible (γ_r = 1)** and **non-compatible deformation** conditions are analyzed.

16.12. Correlations

In SETAF2018, material data can be entered manually or obtained through correlations based on physical and mechanical properties.

Compression Index C_c

Skempton, 1944 $0{,}009\left(w_L - 10\right)$ (181)

Azzoz et al, 1976 $0{,}40\left(e_0 - 0{,}25\right)$ (182)

Azzoz et al, 1976 $0{,}01\left(w_n - 5\right)$ (183)

Azzoz et al, 1976 $0{,}37\left(e_0 + 0{,}003 w_L - 0{,}34\right)$ (184)

Wood and Wroth, 1978 $0{,}5 G_s \left(I_p / 100\right)$ (185)

Nagaraj and Murthy, 1986 $0{,}00234 \times w_L \times G_s$ (186)

Recompression Index C_r

Azzouz et al, 1976 $0{,}15\left(e_0 + 0{,}007\right)$ (187)

Azzouz et al, 1976 $0{,}003\left(w_n + 7\right)$ (188)

Azzouz et al, 1976 $0{,}126\left(e_0 + 0{,}003 w_L - 0{,}06\right)$ (189)

Nagaraj and Murthy, 1986 $0{,}000463w_L.G_s$ (190)

$C_c/7$ (191)

OCR

Kulhawy and Mayne, 1990 $\left(patm/\sigma_z^{'}\right)\times 10^{(1{,}11\text{-}1{,}62I_L)}$ (192)

Preconsolidation pressure σ′c

Nagaraj and Srinivasa Murthy (1985,1986)

$$\log_{10}\sigma_c^{'} = 5{,}97 - 5{,}32\cdot\left(\frac{w_n}{w_L}\right) - 0{,}25\cdot\log_{10}\sigma_c^{'} \quad (193)$$

$$\sigma_c^{'} = 3.78S_u - 2{,}9 \ \text{(units: kPa)} \quad (194)$$

Undrained Shearing Strength S_u

NC Clays Skempton 1957 $\sigma_v^{'}\left(0{,}11+0{,}0037I_p\right)$ (195)

NC Clays Ladd 1957 $\sigma_v^{'}\left(0{,}129+0{,}00435\times I_p\right)OCR^{0{,}8}$ (196)

Shear Strength Parameters c′,ϕ′

$$c' = 0{,}10\cdot c_u \quad (197)$$

$$\phi' = 45 - \left(I_p / \left(0{,}5 + 0{,}04I_p\right)\right) \quad (198)$$

Shear Modulus G

$$\frac{G_{sec}}{P_{atm}} \cong \frac{2{,}5}{1+\nu}N_{1,60} \quad (199)$$

$$\frac{G_{sec}}{P_{atm}} \cong \frac{5}{1+\nu}N_{1,60} \quad (200)$$

$$\frac{G_{sec}}{P_{atm}} \cong \frac{7{,}5}{1+\nu}N_{1,60} \quad (201)$$

PI(%)		0	20	40	60	80	≥100
A		0	0,81	0,30	0,41	0,48	0,5

$$\frac{G_{max}}{P_{atm}}=321\frac{(2{,}97\text{-}e)^2}{1+e}OCR^A\left(\frac{\rho_0^{'}}{P_{atm}}\right)^{0,5} \quad (202)$$

Deep Soil Mixing (DSM)

Wet mix $E_{dm} = 300 \times q_u$ (U.S. Department of Transportation, 2013) (203)

Dry mix $E_{dm} = 150 \times q_u$ (U.S. Department of Transportation, 2013) (204)

Jet Grouting (JG)

Sand $E_c = 7 + 8.1(w{:}c)^2$ (205)

Clay $E_c = 2 + 3.6(w:c)^2$ (206)

Sand $E_c = 800q_u^{0.5}$ (207)

Clay $E_c = 500q_u^{(\frac{2}{3})}$ (208)

$E_c = \beta_e \times 200$

16.13. Excavation Support Structures

In this module, analyses of excavation support systems with **reinforced concrete piles** and **reinforced concrete diaphragm (curtain) walls** are performed.

The analysis includes the following calculations and checks:

- Wall displacement control
- For anchors:
  - Grout–soil interface pull-out check
  - Tendon–grout interface pull-out check
  - Tensile rupture and internal stability checks
- Reinforcement calculations in the wall

- External stability(collapse) control

Additionally, for steel struts:

- Buckling check
- Verification of strut–concrete support connection details completes the design process.

16.13.1. Numerical Analysis Model

The program uses the **Dependent Pressures Method** to determine the pressures acting on the wall. The thrusts applied to the structure are derived as a function of the wall displacement ($K_0 \rightarrow K_a$ or $K_0 \rightarrow K_p$), allowing a realistic modeling of behavior and an economical design solution. The analysis accounts for the gradual development of wall deformations and the subsequent stressing of the anchors. The use of the Dependent Pressures Method requires defining the **modulus of subgrade reaction**, which may vary linearly or nonlinearly.

The program also enables the user to verify the **internal stability of the anchorage system**.

The analysis is performed using the **matrix displacement method**. Displacements, internal forces, and subgrade moduli are evaluated at discrete nodal points.

The structure is discretized into finite elements using the following methodology:

- Nodes are placed at all topological points of the structure, including start and end points, anchor levels, excavation stages, and positions where soil parameters change.
- Additional nodes are introduced until all elements reach approximately equal lengths.

Each element is assigned a **modulus of subgrade reaction**, assuming the soil behaves as an **elastic Winkler spring**. Supports are added after the wall deformation develops, each representing a prescribed displacement applied to the structure.

During the construction stage when **prestressed anchors** are installed, the anchors are modeled as **forces** (Case I in Figure 109). In subsequent construction stages, anchors are modeled as **springs** with stiffness *k* and an associated force (Case II in Figure 109).

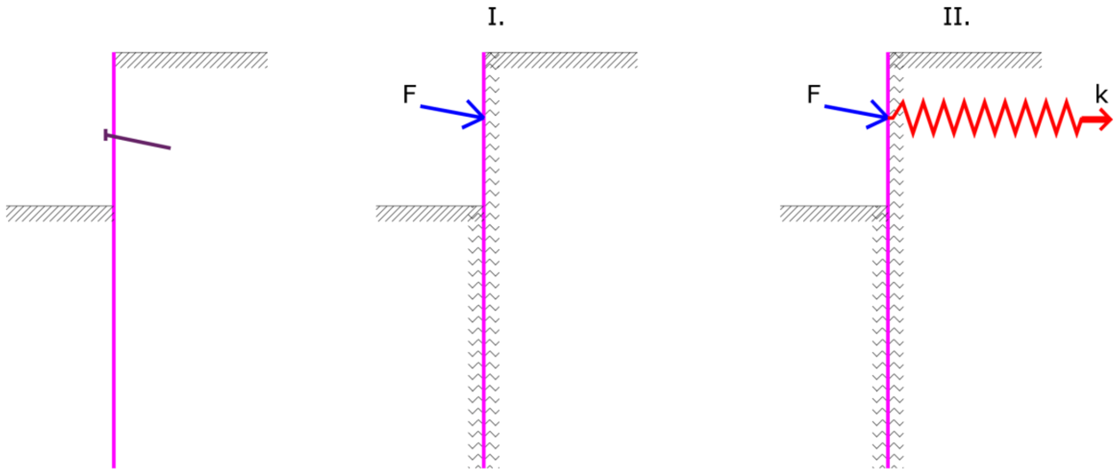


Figure 109. Prestressed Anchorage Model

Anchors without prestressing force are modeled as springs at all stages of construction. The force in each anchor is calculated from the deformation of the structure and the stiffness of the anchor.

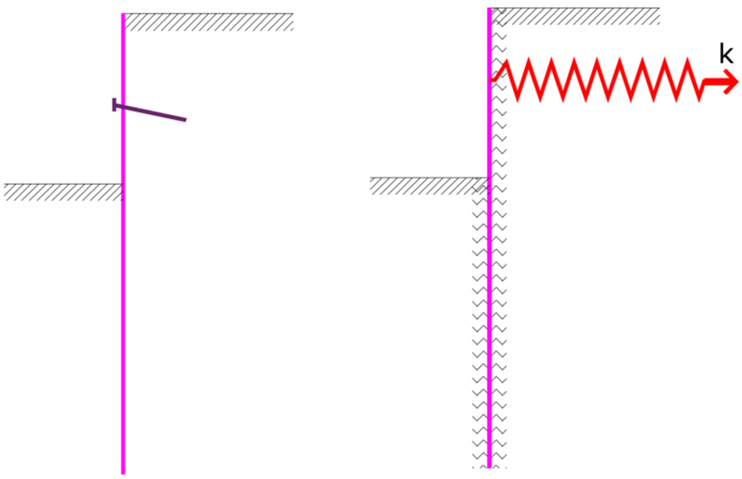


Figure 110. Non-Prestressed Anchorage Model

The variation of anchor force with deformation is expressed as:

$$\Delta F = \frac{k.\nu.\Delta w}{\cos\alpha} \tag{209}$$

$$k = \frac{E.A}{L} \tag{210}$$

where:

v : Horizontal spacing of anchors

Δw : Increment of deformation at the anchor point

E : Modulus of elasticity of the anchor

A : Cross-sectional area of the anchor

l : Anchor length

k : Anchor stiffness

α : Anchor inclination

16.13.2. Dependent Pressures Method

The fundamental assumption of the method is that the soil or rock surrounding the wall behaves as an **ideal elasto-plastic Winkler material**. This material is characterized by the **horizontal subgrade reaction modulus** *kh* and certain **limiting deformation values**. When these limiting deformations are exceeded, the material behaves as perfectly plastic.

The following assumptions apply:

- The pressure acting on the wall may take any value **between the active and passive earth pressure limits**, but cannot exceed them.
- The **at-rest earth pressure** acts on a **non-deformed** wall ($w = 0$).

For a deformed structure, the pressure is calculated as:

$$\sigma = \sigma_r - k_h.w \qquad (211)$$

If $\sigma < \sigma_a \rightarrow \sigma = \sigma_a$

If $\sigma > \sigma_p \rightarrow \sigma = \sigma_p$

Where:

- $\boldsymbol{\sigma_r}$: Pressure at rest state
- $\mathbf{k_h}$: Horizontal subgrade reaction modulus
- **w** : Wall deformation
- $\boldsymbol{\sigma_a}$: Active earth pressure

- σ_p : Passive earth pressure

Calculation procedure:

- A horizontal subgrade reaction modulus k_h is assigned to all elements, and the structure is loaded by the **pressure at rest** (Figure 111).

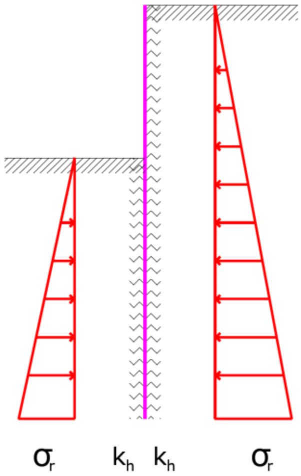


Figure 111. Analysis Model Prior to First Iteration

- The analysis is performed, and the resulting pressures acting on the wall are checked against allowable limits. Where these limits are exceeded, the program **sets the corresponding element to plastic behavior** by **removing the elastic reaction component (kh = 0)** and assigns the **active or passive pressure** accordingly (Figure 112).

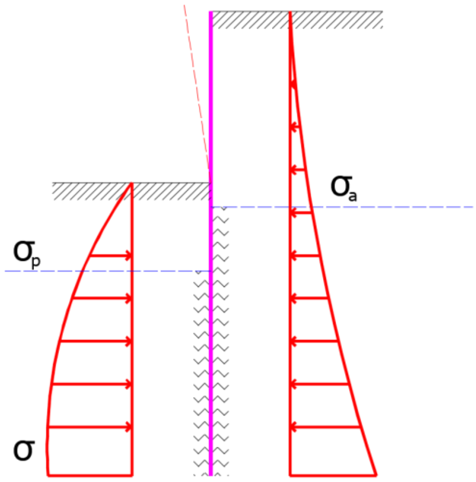


Figure 112. Analysis Model during Iterations

- This iterative procedure continues **until horizontal pressure equilibrium is achieved** along the wall.

During the analysis of subsequent construction stages, the program also accounts for **plastic deformations accumulated in earlier stages**. This is why defining the construction stages realistically is essential, as it influences the resulting wall deformations and earth pressures.

16.13.3. Horizontal Subgrade Reaction Moduli

Several approaches may be used to determine the horizontal subgrade reaction modulus in numerical analysis. Depending on the user input, the program calculates the modulus using the **Schmitt** or **Chadeisson** methods.

16.13.3.1. Schmitt Method

The horizontal subgrade reaction modulus is computed from the **oedometric modulus of the soil** and the **bending rigidity of the wall**, using:

$$k_h = 2{,}1\left(\frac{E_{oed}^{\,4/3}}{(EI)^{1/3}}\right) \tag{212}$$

E: Elastic modulus of the wall [F/L^2],

I: Moment of inertia of the wall section [L^4],

E_{oed}: Oedometric modulus of the soil [F/L^2],

16.13.3.2. Chadeisson Method

$$k_h = \left[20EI\left(\frac{K_p\gamma\left(1-\frac{K_o}{K_p}\right)}{0{,}015}\right)^4\right]^{\frac{1}{5}} + A_p c' \frac{\tanh\left(\frac{c'}{30}\right)}{0{,}015} \tag{213}$$

EI : Bending rigidity of the wall [F.L^2/L]

γ : Unit weight of the soil [F/L^3]

K_p : Passive earth pressure coefficient

K_0: Coefficient of earth pressure at rest

c′ : Effective cohesion [F/L^2]

A_p : Cohesion influence coefficient (typically between **1 and 15**)

16.13.4. Earth Pressures

When calculating the pressures acting on the wall, the at-rest pressure **σ_r**, active pressure **σ_a**, and passive pressure **σ_p** are computed both behind and in front of the wall for all construction stages, assuming the wall is initially at rest. According to the **Dependent Pressures Method**, the pressures acting on the wall derived from displacements **cannot exceed** the limiting values **σ_a** and **σ_p**. Earth pressures are obtained using **Rankine** or **Coulomb** earth pressure theories.

For effective stress analysis, the active and passive earth pressures behind or in front of the wall are:

$$\sigma_a = \sigma'_z . K_a - 2.c' . \sqrt{K_a} \qquad (214)$$

$$\sigma_p = \sigma'_z . K_p + 2.c' . \sqrt{K_p} \qquad (215)$$

For total stress analysis:

- Horizontal active total pressure:

$$\sigma_{x,a} = \sigma_z - K_u . c_u \qquad (216)$$

- Horizontal passive total pressure:

$$\sigma_{x,p} = \sigma_z + K_u . c_u \qquad (217)$$

Horizontal earth pressure at rest:

$$\sigma_r = \sigma'_z \times K_r \qquad (218)$$

Where:

- **K_r** : Coefficient of earth pressure at rest

- $\mathbf{K_a}$: Active earth pressure coefficient
- $\mathbf{K_p}$: Passive earth pressure coefficient
- $\mathbf{K_u}$: Active and passive pressure coefficient for total stress analysis

16.13.5. Rankine Theory

If **“Rankine Theory”** is selected in the analysis settings, the active pressures behind and in front of the wall are computed using this theory. The active earth pressure coefficient is determined based on the ground slope **β** and the effective angle of internal friction **ϕ′** as:

$$K_a = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \tag{219}$$

The passive earth pressure coefficient is:

$$K_p = \cos\beta \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}} \tag{220}$$

Since the Rankine theory assumes **no friction between the wall and the backfill**, the inclination of the active thrust is taken equal to the ground surface slope.

For **β = 0**, the equations reduce to:

$$K_a = \tan^2(45 - \frac{\varphi}{2}) \tag{221}$$

$$K_p = \tan^2(45 + \frac{\varphi}{2}) \tag{222}$$

16.13.6. Coulomb Theory

The Coulomb theory computes the active and passive earth pressure coefficients based on:

- The inclination of the back of the wall relative to the vertical, **α**
- The ground slope, **β**
- The effective angle of shearing resistance, **ϕ′**

- The angle of friction between back of the wall and the backfill, δ

$$K_a = \frac{\cos^2(\phi-\alpha)}{\cos^2\alpha.\cos^2(\alpha+\delta).\left(1+\sqrt{\dfrac{\sin(\phi+\delta).\sin(\phi-\beta)}{\cos(\alpha+\delta).\cos(\alpha-\beta)}}\right)^2} \tag{223}$$

$$K_p = \frac{\cos^2(\phi+\alpha)}{\cos^2\alpha.\cos^2(\delta-\alpha).\left(1+\sqrt{\dfrac{\sin(\phi+\delta).\sin(\phi+\beta)}{\cos(\delta-\alpha).\cos(\beta-\alpha)}}\right)^2} \tag{224}$$

For **α = 0** and **δ = β**, the Coulomb and Rankine solutions coincide.

16.13.7. Earth Pressure Coefficients in Total Stress Analysis

For total stress conditions, the active and passive earth pressure coefficients are computed from:

$$K_u = 2.\sqrt{1+\frac{\alpha_u}{c_u}} \tag{225}$$

Where:

α_u: Adhesion

$S_u(c_u)$: Undrained shear strength

When total stress analysis is performed, both the **undrained shear strength c_u** of the soil and the **adhesion α** between the wall and the soil must be considered. Adhesion **α** is usually taken as a portion of **c_u**, as defined in Equations (109) and (110).

16.13.8. Coefficient of Earth Pressure at Rest

If the coefficient of earth pressure at rest **K_r** is not entered by the user, it is determined as follows:

- For **overconsolidated soils**:

$$K_r = 0.5\times\sqrt{OCR} \tag{226}$$

- For **coarse-grained soils**:

$$K_r = 1 - \sin\phi' \tag{227}$$

- For **fine-grained soils**:

$$K_r = \frac{\nu}{1-\nu}$$

16.13.9. Surcharge Loads (External Loads)

Point, line, strip, and area-type surcharge loads may be applied behind the wall. The horizontal stress increments induced by these surcharge loads at the wall–soil interface are computed and added to the earth pressures σ_a, σ_p, and σ_r.

16.13.9.1. Point Load

A point load **F** applied behind the wall produces a horizontal stress increment at the wall–soil interface, calculated using the Boussinesq equation (Section 16.2.1).

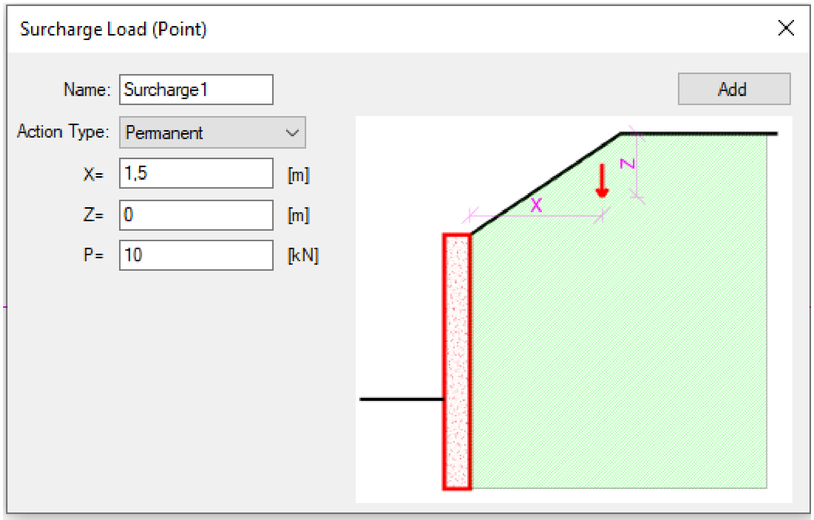


Figure 113. Point Load

16.13.9.2. Line Load

A line load of infinite length (F/L), parallel to the wall, is applied behind the wall. The horizontal stress increment along the wall–soil interface is computed using the integrated form of the Boussinesq equation.

$$\sigma_x = \frac{2p}{\pi} \cdot \frac{x^2.z}{R^4} \tag{228}$$

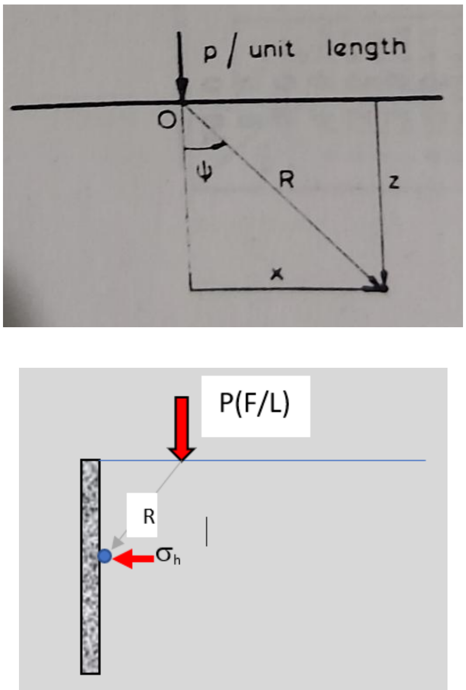


Figure 114. Horizontal stress increases at the wall–soil interface due to a line load

16.13.9.3. Area Load

A distributed load (F/L²) acting over a rectangular area of width **B** and length **L** behind the wall is defined. The Boussinesq equation given in Section 16.2.1. is numerically integrated to compute horizontal stress increase.

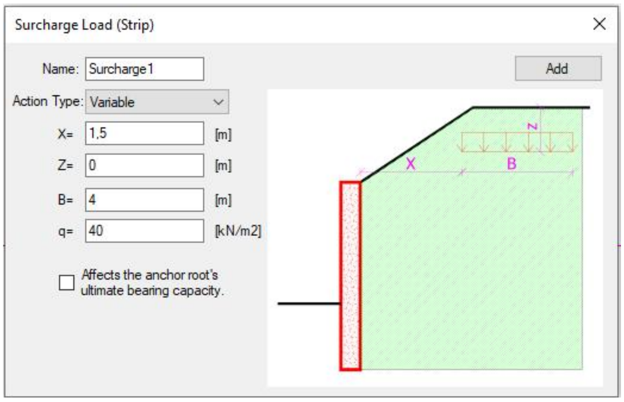


Figure 115. Area Load

16.13.9.4. Strip Load

A uniformly distributed load (F/L²), acting over an infinitely long strip of width **B** parallel to the wall, is applied behind the wall. The horizontal stress increment is computed using:

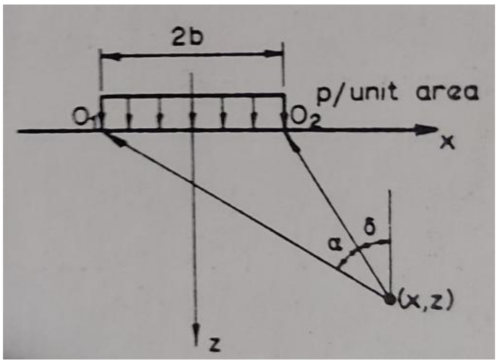


Figure 116. Horizontal Stress Increase due to a Strip Load

$$\sigma_x = \frac{P}{\pi} \cdot \left[\alpha - \sin\alpha\cos(\alpha + 2\delta)\right] \tag{229}$$

where **α** and **δ** are in radians.

16.13.9.5. Trapez Load

Programda şekildeki arazi tipinde duvar üstündeki zemin yüksekliği trapez yük olarak dikkate alınır. Ground surface geometry behind the wall is modeled as a trapezoidal load.

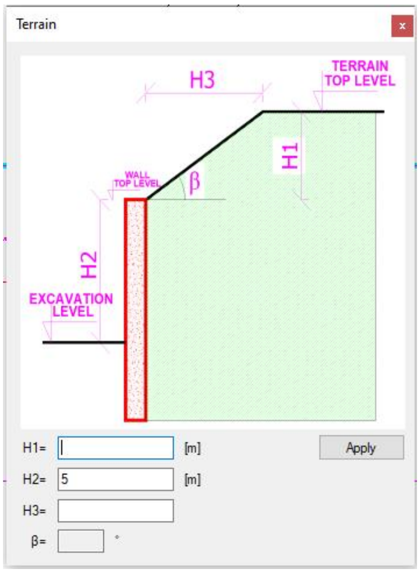
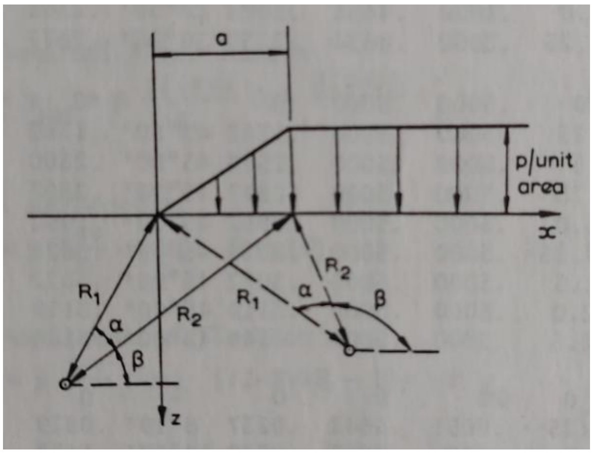


Figure 117. Terrain Type Affected as Trapezoidal Load

For a semi-infinite medium, the horizontal stress increment due to trapezoidal loading (α and β in radians) is:

$$\sigma_x = \frac{p}{\pi a}\left[\alpha\beta + x\alpha + 2z\log_e\frac{R_2}{R_1}\right] \tag{230}$$



Şekil 118. Stress Increase in a Semi-infinite Medium due to Trapezoidal Loading

16.13.10. Earthquake Effects

The equivalent static earthquake load is computed by multiplying the weight of a defined soil wedge behind the wall with the horizontal seismic acceleration coefficient. The dependent pressures method remains valid.

Two seismic analysis options are available:

1. Methods determining seismic earth pressures
2. Quasi-static application of the equivalent earthquake load

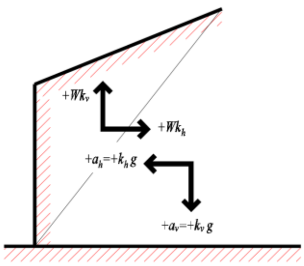


Figure 119. Directions of Earthquake Effects

The lateral seismic coefficient k_h is always assumed positive (unfavorable). The vertical seismic coefficient k_v may take positive or negative values. If the equivalent acceleration acts downward, the inertial force $k_v \cdot W_s$ tends to lift the soil wedge upward. In this case, the equivalent acceleration a_v (and therefore the inertial force $k_v \cdot W_s$) is taken as positive. As shown in Figure 119, inertial forces act in the direction opposite to the acceleration. If the acceleration acts upward ($-a_v = -k_v \cdot g$), then the inertial forces push the soil wedge downward ($-k_v \cdot W_s$). The effect of vertical acceleration may be neglected in excavation support systems, and k_v may be taken as zero.

The seismic inertia angle is:

$$\psi = \tan^{-1}\left(\frac{k_h}{1-k_v}\right) \tag{231}$$

Increase in Active Pressure due to Earthquake:

$$\sigma_{ae,i} = \sigma_{0,i}(K_{ae,i} - K_{a,i}) \tag{232}$$

$$\sigma_{0,i} = \sum_0^H \gamma_i h_i (1-k_v) \tag{233}$$

Where:

γ_i: Unit weight of the i-th soil layer

$K_{ae,i}$: Static–seismic active pressure coefficient

$K_{a,i}$: Coulomb active pressure coefficient

h_i: Thickness of soil layer

Reduction in Passive Resistance due to Earthquake:

$$\sigma_{pe,i} = \sigma_{0,i}(K_{pe,i} - K_{p,i}) \tag{234}$$

$$\sigma_{0,i} = \sum_0^H \gamma_i h_i (1-k_v) \tag{235}$$

16.13.10.1. Mononobe–Okabe Method

The Mononobe–Okabe method increases active pressures and reduces passive pressures during seismic loading. The dependent pressures method applies.

Active pressure coefficient:

$$K_{ae} = \frac{\cos^2(\phi-\psi-\alpha)}{\cos\psi\cos^2\alpha\cos(\psi+\alpha+\delta).\left(1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\psi-\beta)}{\cos(\delta+\psi+\alpha)\cos(-\beta+\alpha)}}\right)^2} \tag{236}$$

Passive pressure coefficient:

$$K_{pe} = \frac{\cos^2(\phi-\psi+\alpha)}{\cos\psi\cos^2\alpha\cos(\psi-\alpha+\delta).\left(1-\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\psi+\beta)}{\cos(\delta+\psi-\alpha)\cos(\beta-\alpha)}}\right)^2} \tag{237}$$

Where:

γ: Unit weight of soil

H: Wall height

ϕ: Soil angle of shearing resistance

δ: Wall–soil friction angle

α: Inclination of the back of the wall

β: Ground slope behind the wall

ψ: Seismic inertia angle

ψ for active pressure must satisfy: $\psi \leq (\phi - \beta)$

If ψ is larger, the program uses $\psi = \phi - \beta$.

For passive pressure: $\psi \leq (\phi + \beta)$

16.13.10.2. Static–Equivalent Earthquake Loading

The equivalent static earthquake force derived from the weight of a soil wedge is applied at a height of **0.66H**. This load is distributed to all wall nodes. The dependent pressures method applies. Active and passive pressures are computed using Rankine or Coulomb theory.

16.13.11. Failure of Anchorage Grout Body–Soil Interface by Pullout

A pullout check is performed using the anchorage forces obtained from the analysis.

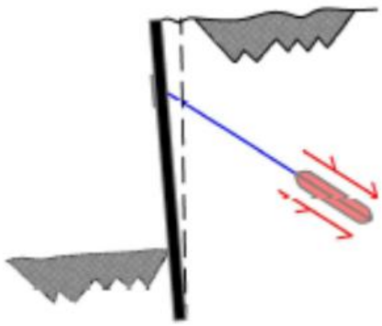


Figure 120. Pullout Mechanism of the Anchorage Grout Body along the Soil Interface

The ultimate pullout capacity of the anchorage is calculated using:

$$T_f = \pi . D . L_{tb} . \tau_f \tag{238}$$

Where:

D: Effective diameter of the anchorage root

L_{tb}: Tendon (cable) root length

τ_f: Ultimate skin friction

The pullout resistance is either entered by the User or computed from the ultimate skin friction. Ultimate skin friction $\boldsymbol{\tau_f}$ may be determined from effective or total stress conditions.

For **effective stresses**:

$$\tau_f = K_1 . \sigma_v' . \tan\phi' \tag{239}$$

where:

- **K_1**: Earth pressure coefficient for low-pressure grouted anchors (typically 1.4–2.3). For fine sands and silts, values range between 1.0 and 1.5 depending on soil relative density (D_r).
- **σ'_v**: vertical effective stress at the midpoint of the anchor root.

For **total stresses**:

$$\tau_f = \alpha_a . S_u \tag{240}$$

α_a: Adhesion factor

S_u: Average undrained shear strength of soil along the anchor root

Ultimate skin friction may also be taken empirically from the tables:

Table 3. Ultimate Skin Friction Values for Anchors in Cohesive Soils – τ_f

Anchor Type / Soil Type	**Average Ultimate Shaft Resistance for Soil/Grout Interface – τf [MPa]**
Low-Pressure Grouted Anchors	**0.03 – 0.07**
Pressure-Grouted Anchors	
Soft Silty Clay	0.03 – 0.07
Silty Clay	0.03 – 0.07
Medium–High Plasticity Stiff Clay	0.03 – 0.10

Anchor Type / Soil Type	Average Ultimate Shaft Resistance for Soil/Grout Interface – τf [MPa]
Medium–High Plasticity Very Stiff Clay	0.07 – 0.17
Medium Plasticity Stiff Clay	0.10 – 0.25
Medium Plasticity Very Stiff Clay	0.14 – 0.35
Medium Plasticity Very Stiff Sandy Silt	0.28 – 0.38

Table 4. Ultimate Skin Friction Values for Anchors in Cohesionless Soils – τ_f

Anchor Type / Soil Type	Average Ultimate Shaft Resistance for Soil/Grout Interface – τf [MPa]
Low-Pressure Grouted Anchors	**0.07 – 0.14**
Pressure-Grouted Anchors	
Medium Dense – Dense, Fine to Medium Sand	0.08 – 0.38
Medium Dense, Medium to Coarse Gravelly Sand	0.11 – 0.66
Dense – Very Dense, Medium to Coarse Gravelly Sand	0.25 – 0.97

Anchor Type / Soil Type	Average Ultimate Shaft Resistance for Soil/Grout Interface – τf [MPa]
Silty Sand	0.17 – 0.41
Dense Glacial Till	0.30 – 0.52
Medium Dense – Dense Sandy Gravel	0.21 – 1.38
Dense – Very Dense Sandy Gravel	0.28 – 1.38

Table 5. Ultimate Skin Friction Values for Anchors in Rock – τ_f

Rock Type	Average Ultimate Shaft Resistance for Rock/Grout Interface – τf [MPa]
Granite – Basalt	1.7 – 3.1
Dolomitic Limestone	1.4 – 2.1
Soft Limestone	1.0 – 1.4
Slate and Hard Shales	0.8 – 1.4
Soft Shales	0.2 – 0.8
Sandstone	0.8 – 1.7
Weathered Sandstone	0.7 – 0.8
Chalk	0.2 – 1.1

Rock Type	Average Ultimate Shaft Resistance for Rock/Grout Interface – τf [MPa]
Weathered Marl	0.15 – 0.25
Concrete	1.4 – 2.8

User-defined anchorage capacity T_f may also be supplied directly.

16.13.12. Tensile Failure of Tendons

The tensile resistance F_u of an anchor is:

$$F_t = A_t \times f_u \tag{241}$$

$$F_u = (\text{Number of tendons}) \times F_t \tag{242}$$

d_t: Nominal diameter of a tendon

A_t: Nominal cross-sectional area of a tendon

D_s: Diameter of an anchor

f_u: Nominal tensile strength of a tendon [F/L^2]

F_t: Rupture load of a tendon [F]

F_u: Tensile resistance of an anchor [F]

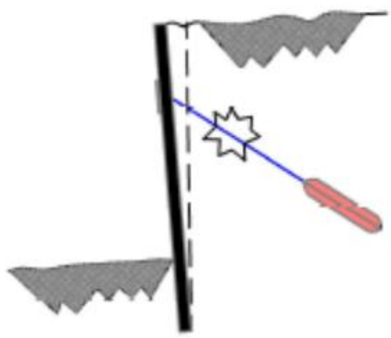


Figure 121. Tensile Failure of the Anchor Tendon

16.13.13. Pullout of Tendon From the Grout Body

This check applies to strand-type (steel tendon) anchors.

Pullout resistance F_u is:

$$F_u = \pi \times D_s \times L_k \times \tau \tag{243}$$

f_c: Compressive strength of grout

τ: Bond strength

D_s: Diameter of anchorage

L_k: Bond length (grouted length)

Bond strength according to **TS500**:

$$\tau = C_1 \times f_{ctd} \tag{244}$$

$$C_1 = \frac{1}{4 \times C_0} \tag{245}$$

$$f_{ctd} = 0.35 \times \sqrt{f_c} \tag{246}$$

f_{ctd}: Tensile strength of grout

For ribbed bars: C_0= 0.24

Bond strength according to **ACI**:

$$\tau = 3.3 \times \sqrt{f_c} \tag{247}$$

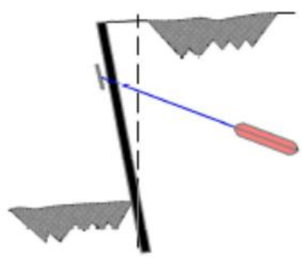


Figure 122. Pullout of Tendon from the Grout Body

16.13.14. Vertical Displacements Behind the Support Structure

Vertical displacements occurring on the ground surface behind the wall are calculated based on the horizontal movements of the wall.

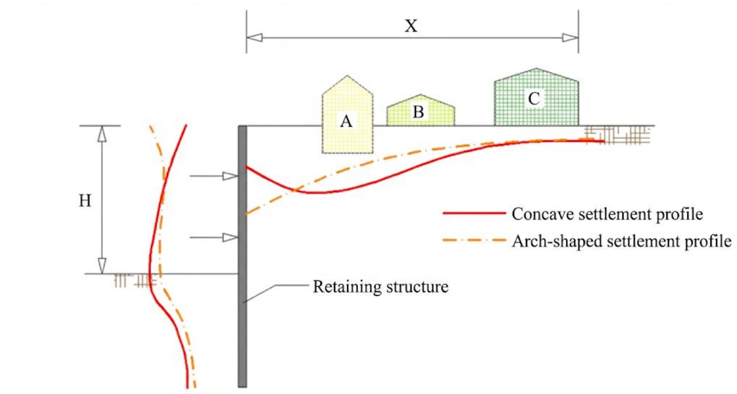


Figure 123. Vertical Settlements Occurring behind the Support Structure due to Wall Displacement

16.13.15. Calculation of Socket (Embedment) Length

The portion of the vertical support element that remains below the final excavation level is defined as the **socket (embedment) length**. The required socket length is determined using methods based on **plastic equilibrium**, and the selected method must be appropriate for the soil conditions to which it is applied. The socket length ensuring stability is computed through an iterative procedure for all construction stages. The software increases the socket length starting from zero, using successive increments. The default increment is **0.50 m**, and reducing this increment increases the accuracy of the result.

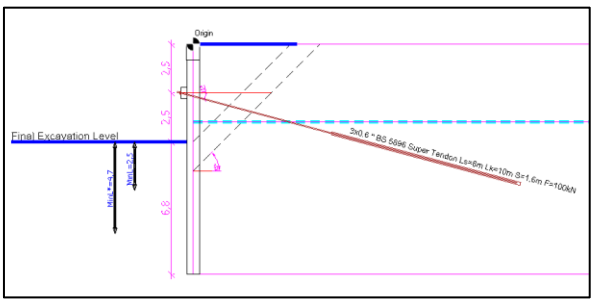


Figure 124. Determination of the Required Socket (embedment) Length for the Support Element

16.13.16. Internal Stability of Anchors

The internal stability of anchored excavation support systems is evaluated by analysing the equilibrium of soil wedges that may form at each construction stage. A block analysis is performed for every active anchor at the relevant stage. During the stability check of an anchor, the forces of other anchors are included in the block analysis as long as they influence the equilibrium of the anchor being evaluated.

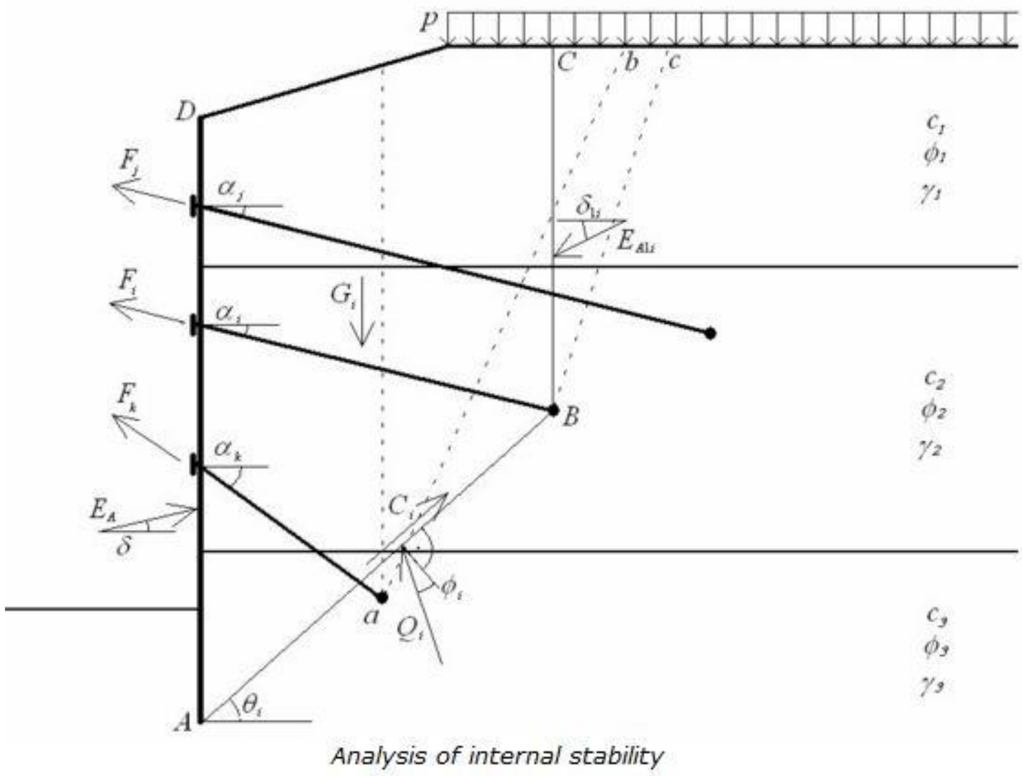


Figure 125. Internal Stability Analysis of an Anchored Support System

The wedge for the checked anchor is defined by connecting the theoretical bottom point of the wall (A) to the midpoint of the anchor root (B), and extending a vertical line from this point to the ground surface (C). At the theoretical bottom point, the resultant of horizontal forces acting on the wall below the excavation level is zero.

E_a: Resultant of active earth pressures acting between A and D [F/L]

E_{ai}: Resultant of the active thrust acting on the root of the anchor being checked [F/L]

W_i: Weight of block ABCD [F/L]

C_i: Cohesion-derived shear resistance acting along sliding surface AB [F/L]

$F_{j,k..}$: Forces of additional anchors included in the block analysis [F/L]

Determination of Additional Anchors to Include in the Block Analysis

- Anchors below the checked anchor:
  A potential sliding surface is defined by drawing a line inclined at **$(45-\phi_n/2)$** from the bottom point to the anchor root midpoint, then extending it to the ground surface. If the root of a lower anchor lies **outside** this surface, that anchor is included in the analysis.
- Anchors above the checked anchor:
  For the checked anchor, a sliding surface at **$(45-\phi_n/2)$** is constructed in the same way. If the root of an upper anchor lies **within** this sliding surface, it is included in the analysis.
  φ_n: Average shearing resistance angle along the root of upper or lower anchors described above.

Q_i: Reaction force on sliding surface AB

F_i: Maximum allowable force on the checked anchorage [F/L]

The equilibrium of block ABCD is solved by writing the horizontal and vertical force equilibrium equations:

$$\begin{bmatrix} \cos\alpha_i & \cos(90+\phi_i-\theta_i) \\ \sin\alpha_i & \sin(90+\phi_i-\theta_i \end{bmatrix}\begin{bmatrix} F_i \\ Q_i \end{bmatrix}=\begin{bmatrix} E_a\times\cos\delta+C_i\times\cos\theta_i-E_{ai}\times\cos\delta_i-\sum_{j,k..}F_{j,k..}\times\cos\alpha_{j,k..} \\ W_i+E_{ai}\times\sin\delta_i-C_i\times\sin\theta_i-E_a\times\sin\delta-\sum_{j,k..}F_{j,k..}\times\sin\alpha_{j,k..} \end{bmatrix} \quad (248)$$

From these equations, the maximum allowable force **F_i** is obtained. The **Factor of Safety** is computed as: FS= F_i/F

16.13.17. Effects on Steel Pipe Supports

Fixed and variable effects shall be determined according to the load combinations (LC) defined below. Thermal loading is considered the primary variable load in all cases.

Load combinations:

$$LC1: 1.4G_k+G_{k,GEO}+Q_{k,temp} \quad (249)$$

$$LC2: 1.2G_k + G_{k,GEO} + 1.6Q_{k,temp} \quad (250)$$

$$LC3: G_k + G_{k,GEO} + 0.5Q_{k,temp} + 1.6Q_{k,tesadufi} \quad (251)$$

$G_{k,GEO}$: The characteristic geotechnical load is considered as a permanent load.

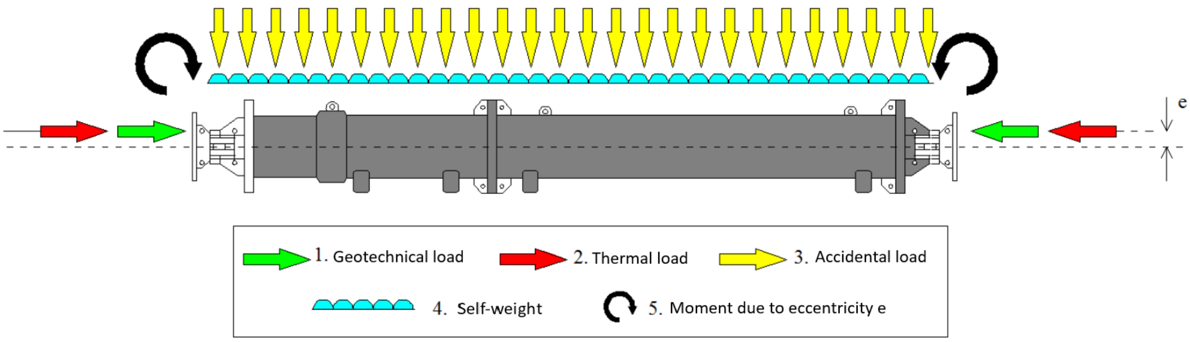


Figure 126. Loads Acting on the Horizontal Support Element

$Q_{k,temp}$: The characteristic thermal load is calculated based on the expected temperature change and is considered as a variable load depending on the resistance provided by the excavation support system.

$Q_{k,random}$: In addition to the self-weight of the excavation support element, a vertical line load of 1.0 kN/m acting on the support should be considered.

G_k : The total characteristic self-weight of the support is assumed to be uniformly distributed along its length and is considered as a permanent load.

Since the connection between the support and the wall is not designed as a non-moment-transmitting joint, an eccentric load equal to one-sixth of the steel pipe diameter is considered to account for the resulting moment.

Characteristic thermal load:

$$Q_{k,temp} = \alpha_t \cdot \Delta t \cdot E \cdot A \cdot (\beta / 100) \quad (252)$$

The terms in the equation:

α_t : Coefficient of thermal expansion of the support material

Δt : Temperature change relative to the installation temperature

E : Modulus of elasticity of the support material

A : Cross-sectional area of the support element

β : Represents the percentage of axial length change that is restrained in the support element.

The horizontal support is designed for the most unfavorable loading case. The value of $G_{k;GEO}$ is determined based on the greater of the SLS or ULS load cases.

For the SLS case:

$$G_{k,GEO} = \gamma_G \cdot P_{SLS} \cdot \gamma_{SD} \tag{253}$$

Explanations:

P_{SLS} : Represents the effects obtained from stress-strain analyses or limit equilibrium methods under SLS conditions.

γ_G : Takes the values 1.4, 1.2, and 1.0 for LC1, LC2, and LC3, respectively.

γ_{SD} : Represents the stress redistribution factor (taken as 1.0 in analyses that account for changes in horizontal earth pressure as excavation stages progress).

For the ULS case:

$$G_{k,GEO} = P_{ULS} \cdot \gamma_{SD} \tag{254}$$

Where,

P_{ULS} : Represents the design effects under ULS conditions obtained from stress-strain analyses, increased by partial safety factors.

16.13.18. Design of the Horizontal Support Element with Steel Pipe Cross-Section

The design of the steel structural element is carried out using the Load and Resistance Factor Design (LRFD) approach.

Cross-section classification for local buckling under axial compression force:

$$\lambda = \frac{D}{t} \tag{255}$$

$$\lambda_r = 0.11\frac{E}{F_y} \tag{256}$$

$$\begin{bmatrix} \lambda \le \lambda_r \rightarrow \text{Non-slender section} \\ \lambda > \lambda_r \rightarrow \text{Slender cross-section} \end{bmatrix} \tag{257}$$

D : Outer diameter of the pipesection

t : Wall thickness of the pipesection

λ_r : Limit value for width / thickness ratio

E : Modulus of elasticity of steel

F_y : Yield stress of steel

Slenderness ratio:

$$L_c = K \cdot L \tag{258}$$

L_c : Buckling length of the steel pipe support

L : Length of the steel pipe support

K : Effective length factor

Slenderness limit:

$$\frac{L_c}{i} \le 200 \tag{259}$$

i : Radius of gyration of the pipe section

Effective area in slender pipe sections under axial compression:

$$\frac{D}{t} \le 0.11\frac{E}{F_y} \rightarrow A_e = A_g \tag{260}$$

$$0.11\frac{E}{F_y} < \frac{D}{t} < 0.45\frac{E}{F_y} \rightarrow A_e = \left(\frac{0.038E}{F_y\left(D/t\right)} + \frac{2}{3}\right)A_g \tag{261}$$

A_g : Gross cross-sectional area

A_e : Effective area

Nominal axial compressive strength in flexural buckling limit state:

$$F_e = \frac{\pi^2 E}{\left(\frac{L_c}{i}\right)^2} \tag{262}$$

$$\frac{L_c}{i} \leq 4.71\sqrt{\frac{E}{F_y}} \rightarrow F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y \tag{263}$$

$$\frac{L_c}{i} > 4.71\sqrt{\frac{E}{F_y}} \rightarrow F_{cr} = 0.877F_e \tag{264}$$

For non-slender sections:

$$P_n = F_{cr}A_g \tag{265}$$

For slender sections:

$$P_n = F_{cr}A_e \tag{266}$$

$$P_c = \phi_c P_n \tag{267}$$

F_e : Elastic buckling stress

F_{cr} : Critical buckling stress

P_n : Nominal axial compressive strength

ϕ_c : Resistance factor for axial compression

P_c : Design axial compressive strength

Cross-section classiffication for local buckling under bending moment:

$$\lambda_{r,bending} = 0.31\frac{E}{F_y} \tag{268}$$

$$\lambda_p = 0.07\frac{E}{F_y} \tag{269}$$

$$\begin{bmatrix} \lambda \le \lambda_p \rightarrow \text{Compact cross} - \text{section} \\ \lambda > \lambda_p \text{ve} \lambda \le \lambda_{r,bending} \rightarrow \text{Non} - \text{compact cross} - \text{section} \\ \lambda > \lambda_{r,bending} \rightarrow \text{Slender cross - section} \end{bmatrix} \tag{270}$$

λ_p : Limiting width-to-thickness ratio

Nominal flexural strength in the yielding limit state:

$$M_n = M_p = F_y W_p \tag{271}$$

If the non-compact section condition is satisfied:

$$M_n = \left[\frac{0.021E}{\left(\frac{D}{t}\right)} + F_y\right] W_e \tag{272}$$

If the slender section condition is satisfied:

$$M_n = F_{cr} W_e \tag{273}$$

Critical stress:

$$F_{cr} = \frac{0.33E}{\left(\frac{D}{t}\right)} \tag{274}$$

Design bending moment strength:

$$M_c = \phi_b M_n \tag{275}$$

W_p : Plastic section modulus

W_e : Elastic section modulus

ϕ_b : Resistance factor for bending

Interaction between flexural moment and axial compression in the steel pipe suppor telement:

$$\frac{P_r}{P_c} \geq 0.2 \rightarrow \frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1 \quad (276)$$

$$\frac{P_r}{P_c} < 0.2 \rightarrow \frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1 \quad (277)$$

P_r : Required axial strength under load combinations

M_{rx}, M_{ry} : Required flexural strengths

Nominal shear strength of steel pipe sections:

$$V_n = \frac{F_{cr} A_g}{2} \quad (278)$$

For shear buckling limit state, critical shear stress **F_{cr}** is taken as the larger of the values calculated from Equations (279) and (280).

$$F_{cr} = \frac{1.60E}{\sqrt{\frac{L_v}{D}}\left(\frac{D}{t}\right)^{5/4}} \leq 0.6F_y \quad (279)$$

$$F_{cr} = \frac{0.78E}{\left(\frac{D}{t}\right)^{3/2}} \leq 0.6F_y \quad (280)$$

L_v : Distance between the point of zero shear and the point of maximum shear

Design shear strength:

$$V_d = \phi_v V_n \quad (281)$$

ϕ_v : Resistance factor for shear

Check for design shear strength:

$$\frac{V_u}{V_d} \le 1 \tag{282}$$

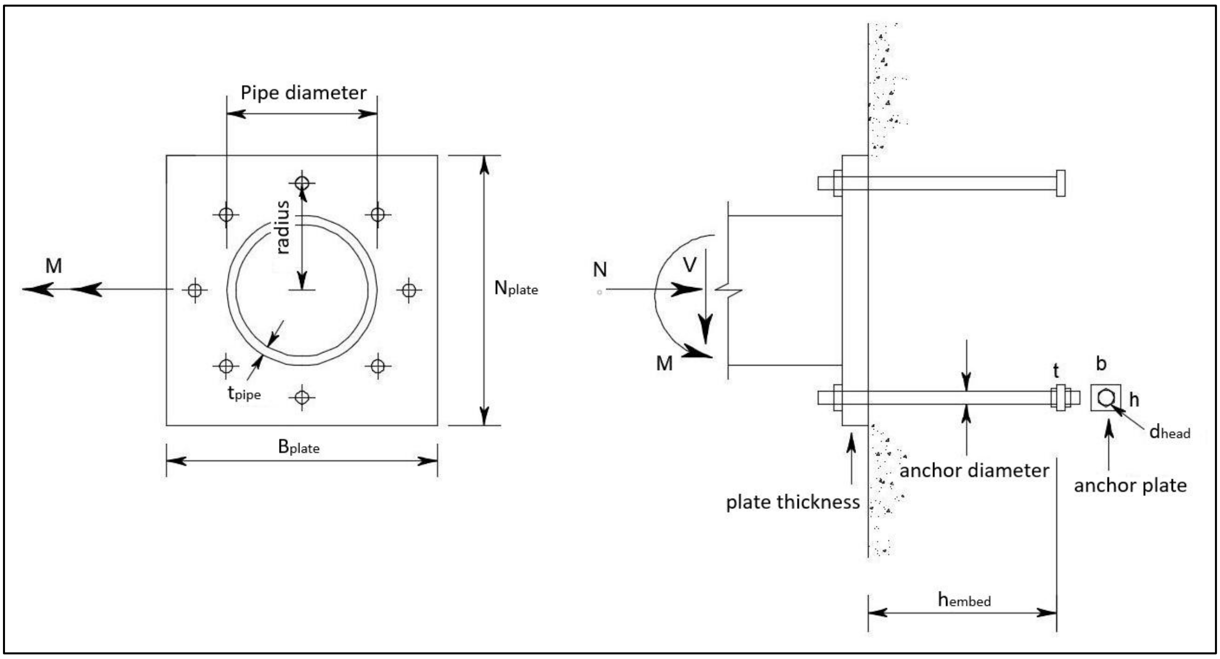
V_u : Required shear strength under load combinations

16.13.19. Design of Steel Pipe Support - Wall Connection

The design is performed for the connection of the steel pipe support to a reinforced concrete wall or beam by anchorage

Plate surface area:

$$A_1 = N \cdot B \tag{283}$$



Şekil 127. Connection of Support Element to Concrete Bearing Surface

N : Plate dimension perpendicular to the bending moment vector

B : Plate dimension parallel to the bending moment vector

Concrete bearing area under the plate:

$$A_2 = N_2 \cdot B_2 \tag{284}$$

N_2 : Concrete bearing surface dimension perpendicular to the bending moment vector

B_2 : Dimension of the concrete bearing surface under the plate in the direction parallel to the bending moment vector

First cantilever length on the plate:

$$m = \frac{N - 0.95d}{2} \tag{285}$$

D : Outside diameter of the pipe

Second cantilever length on the plate:

$$n = \frac{B - 0.8b_f}{2} \tag{286}$$

b_f : The outer diameter of the pipe is taken.

Maximum concrete stress shall be taken as the lesser of the values calculated by Equation (287) and (288).

$$f_{p,maks} = \phi 0.85 f_c \sqrt{\frac{A_2}{A_1}} \tag{287}$$

$$f_{p,maks} = 1.7 f_c \tag{288}$$

f_c : Concrete compressive strength

Maximum uniform load on concrete:

$$q_{maks} = f_{p,maks} B \tag{289}$$

Eccentricity:

$$e = \frac{M_r}{P_r} \tag{290}$$

Critical eccentricity:

$$e_{cr} = \frac{N}{2} - \frac{P_r}{2 f_{p,max} B} \tag{291}$$

P_r : Required axial strength under load combinations

M_r : Required flexural moment capacity under load combinations

Y : Adjusted plate dimension in the direction perpendicular to the bending moment vector due to eccentricity

$$e \le e_{cr} \rightarrow Y = N - 2e \tag{292}$$

$$e > e_{cr} \rightarrow Y = \left(f + \frac{N}{2}\right) - \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_r(e+f)}{q_{maks}}} \tag{293}$$

Stress on the concrete under the base plate:

$$f_p = \frac{P_r}{B \cdot Y} \tag{294}$$

Plate dimension adequacy check:

$$\left(f + \frac{N}{2}\right)^2 \ge \frac{2P_r(e+f)}{q_{maks}} \tag{295}$$

f : Distance from the outer bolt to the center of the plate

Concrete bearing stress check:

$$\frac{f_p}{f_{p,maks}} < 1 \tag{296}$$

Tensile force:

$$T_u = q_{max} \times Y - P_r \tag{297}$$

Required thickness of the base plate is determined according to the following conditions.

For Y>m condition:

$$t_{p,req} = 1.49m\sqrt{\frac{f_p}{F_y}} \tag{298}$$

For Y≤m condition:

$$t_{p,req} = 2.11\sqrt{\frac{f_{p,maks}Y\left(m-\frac{Y}{2}\right)}{F_y}} \quad (299)$$

For Y>n condition:

$$t_{p,req} = 1.49n\sqrt{\frac{f_p}{F_y}} \quad (300)$$

For Y≤n condition:

$$t_{p,req} = 2.11\sqrt{\frac{f_{p,maks}Y\left(n-\frac{Y}{2}\right)}{F_y}} \quad (301)$$

$$t_{p,req} = 2.11\sqrt{\frac{T_u x}{BF_y}} \quad (302)$$

T_u : Tensile force

x: Distance between the outermost bolt under tension and the pipe section

The required plate thickness will be taken as the maximum of the values calculated from equations (298), (299), (300), (301) and (302).

Plate thickness check:

$$\frac{t_{p,gerekli}}{t_p} \leq 1 \quad (303)$$

Anchor Design Capacity

Diameter of anchor rod:

$$d \geq \sqrt{\frac{4T_{u,1}}{\pi\phi F_{nt}}} \quad (304)$$

F_{nt} : Anchor tensile strength

$T_{u,1}$: Tensile force acting on a single anchor

Anchor rod diameter shall not be less than 22mm.

Tensile limit state of anchor group in tension zone:

$$\phi N_{sa} = \phi . n . A_{se,N} . f_{uta} \tag{305}$$

ϕN_{sa} : Reduced nominal tensile strength of anchor group in tension zone

ϕ: Strength reduction factor

n: Number of anchor rods in tension zone

$A_{se,N}$: Effective tensile area of anchors in tension zone

f_{uta} : Specified tensile strength of anchor steel

Concrete cone breakout check:

$$A_{Nc} = \left(1.5h_e + x_d\right)\left(2 \times 1.5h_e + s_1\right) \tag{306}$$

$$A_{Nco} = 9 \times h^2_{embedded} \tag{307}$$

If embedment depth 280mm ≤ h_e ≤ 635mm:

$$N_b = 3.9\lambda\sqrt{f_{ck}}\, h_e^{5/3} \tag{308}$$

If h_e is not within the limits of 280 mm and 635 mm:

$$N_b = 10\lambda\sqrt{f_{ck}}\, h_e^{1.5} \tag{309}$$

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \tag{310}$$

N_b : Concrete tensile breakout strength for a single anchor

f_{ck} : Characteristic compressive strength of concrete

h_e : Anchor embedment depth

N_{cbg} : Characteristic breakout strength for anchor group in tension

x_d : Distance from the outermost bolt to the edge of the plate in the tension zone

s_1 : Distance between edge bolts in tension zone

A_{Nc} : Projected concrete breakout area for anchor group in tension

A_{Nco} : Projected concrete breakout area for a single anchor in tension

$\psi_{ec,N}$: Modifier for eccentric loading

$\psi_{ed,N}$: Modifier for proximity to concrete edge

$\psi_{c,N}$: Modifier for presence of cracks in concrete

$\psi_{cp,N}$: Modifier for post-installed anchors in uncracked concrete

Concrete pullout strength:

$$A_{brg} = b \cdot h - \frac{\pi d_b^{\ 2}}{4} \tag{311}$$

Tensile pullout strength for one anchor:

$$N_p = 8 \cdot A_{brg} \cdot f_c \tag{312}$$

Design tensile pullout strength of anchor group:

$$N_{pn} = 0.40 \cdot 0.75 \cdot \phi \cdot n \cdot \psi_{c,P} \cdot N_p \tag{313}$$

A_{brg} : Net bearing area on anchor plate

d_b : Diameter of anchor bolt head

b : Dimension of anchor end plate

h : Dimension of anchor end plate

N_{pn} : Design pullout strength of anchor group in tension

ϕ: Strength reduction factor

n: Number of anchors

$\psi_{c,P}$: Modifier for concrete cracking under pullout

N_p : Pullout strength of single anchor in tension

Bearing check of anchor end plate:

$$W_{p,h} = \frac{t_h^{\,2}}{4} \tag{314}$$

$$M_{n,h} = F_{y,h} W_{p,h} \tag{315}$$

$$M_{p,h} = 0.4 \times 0.9 \times M_{n,h} \tag{316}$$

$$L_{p,h} = \frac{b - L_b}{2} \tag{317}$$

$$w_u = \frac{(T_u / n)}{A_{brg}} \tag{318}$$

$$M_u = \frac{w_u \cdot L^2_{p,h}}{2} \tag{319}$$

$W_{p,h}$: Plastic section modulus of anchor end plate

t_h : Thickness of anchor end plate

$F_{y,h}$: Yield strength of anchor end plate

$M_{n,h}$: Nominal moment capacity of anchor end plate

$M_{p,h}$: Design moment capacity of anchor end plate

$L_{p,h}$: Cantilever length of anchor end plate

L_b : Height of anchor bolt head

w_u : Distributed load per unit width due to tension force

M_u : Moment acting on anchor plate

Weld design capacity:

$$L_e = \pi D \tag{320}$$

$$R_{nw} = 0.75 \times 0.60 \times 0.9 \times F_{exx} \times w \times (L_e - 2w) \tag{321}$$

$$W_{p,w} = \pi D^2 w \tag{322}$$

$$M_{p,w} = W_{p,w} \times 0.6 \times 0.9 \times F_{exx} \tag{323}$$

L_e : Length of weld

R_{nw} : Shear capacity of weld

w : Weld thickness

F_{exx} : Tensile strength of weld metal

$W_{p,w}$: Plastic section modulus of weld

$M_{p,w}$: Flexural capacity of weld

16.14. Stability of Slopes and Excavated Faces

Stability of slopes and excavated faces is evaluated using limit equilibrium methods. The factor of safety is computed using the **OMS–Fellenius** and **Bishop** slice methods.

The user defines the soil layers according to the desired geometry. These layers may be included in the analysis using **effective stresses** or **total stresses**. In addition, the effects of **ground anchors with steel tendons** and **soil nails** can be incorporated into the system.

16.14.1. Anchors

Anchors are defined by their **head location**, **free length** l, **bond (root) length** l_k, and **inclination** α. The anchor force is entered as the **prestressing force** applied to the anchor. If the model is converted from an excavation support analysis, the anchor force must be the force obtained from the wall analysis.

The head of the anchor is always at the ground surface, and the anchor force acts toward the soil mass. The component of the anchor force acting on the **normal force at the base of the slice** is included, increasing the resisting forces of the slice. However, the component acting **opposite to the driving force** along the slice base is **neglected**, keeping the analysis on the safe side.

Only anchors whose **root ends remain behind the sliding surface** are considered in the analysis.

- If the sliding surface intersects the **free length**, the **full anchor force** is applied.
- If the sliding surface intersects the **bond (root) zone**, the anchor force is assumed to be **full at the top of the root**, **zero at the root tip**, and **linearly decreasing** in between.

This approach is particularly used when evaluating the stability of slopes supported by **existing anchors**, since anchors that do not adequately intersect the critical sliding surface contribute little to system stability.

16.14.2. Soil Nails

Soil nails generally consist of steel tendons placed within a grout body. These tendons may be:

- ribbed reinforcing bars (rebar)
- prestressed steel strands
- fully threaded steel bars

In FHWA literature, the terms **“tendon”** and **“bar”** are used interchangeably.

The tendons carry tensile forces while embedded in the soil, and their **tensile rupture load (R_t)** determines the ultimate capacity of the soil nailing system. Therefore, the tendon rupture load is taken as the limiting value in soil nail analyses.

The capacity of a nail depends on where it intersects the sliding surface:

- If the nail is entirely located **in front of** the sliding surface, it is not included in the analysis.

- If the sliding surface intersects the nail, its capacity is determined as follows:

$$F = Min(T_p.x, R_t) \tag{324}$$

x: Length of the nail behind the sliding surface [L]

T_p: Pullout resistance at the nail–soil interface [F/L]

R_t: Tendon kopma yükü [F] (Bölüm 16.13.12)

$$T_p = D \times f_s \tag{325}$$

D: Grout body diameter

f_s: Ultimate unit skin friction at the grout–soil interface [F/L^2]

Using effective stresses:

$$f_s = K_1.\sigma_v^{'}.\tan\phi^{'} \tag{326}$$

16.14.3. Surcharge Loads

The applied surcharge load is spread to the base of the slice using a 1:2 load-distribution (spreading) assumption and added to the weight of that slice.

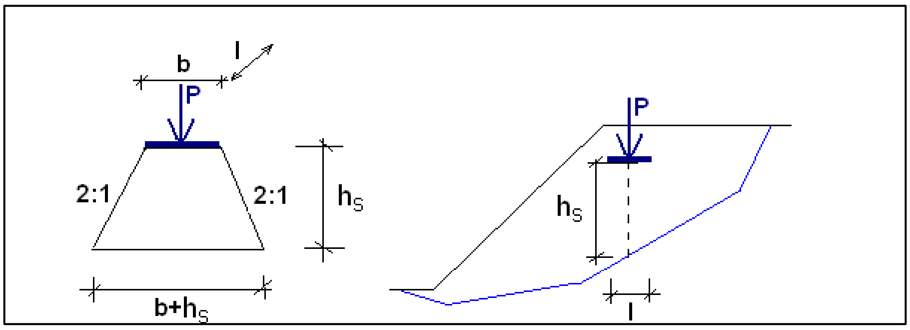


Figure 128. Transformation of the surcharge load to the slice base using a 1:2 load-spreading assumption

$$p_i = \frac{P}{(b+z)(1+z)} \tag{327}$$

$$M_a = p_i \times L \tag{328}$$

P: Point load

b: Width of surcharge load

l: Length of surcharge load

L: Width of slice base

p_i: Pressure acting on the slice base

M_a: Additional surcharge force added to slice weight

The surcharge load entered as P represents the point load. For line, strip and area loads, the applied surcharge is converted into an equivalent point load.

16.14.4. OMS/Fellenius Method

This method is the earliest slice-based limit equilibrium solution developed in the literature and is also known as the Swedish method.

$$FS = \frac{\sum_i^n c_i.L_i + N_i \tan\phi_i}{\sum_i^n W_i \operatorname{Sin}\alpha_i} = \frac{\sum_i^n R_s}{\sum_i^n F_s} \tag{329}$$

$$N_i = \left(W_i - u_i \times L_i\right) \times \cos\alpha \tag{330}$$

The weight of each slice is resolved into components parallel and perpendicular to the slice base. The normal component acting on the base is used to compute the available shear resistance, while the parallel component represents the driving shear force. The factor of safety is calculated by taking moments around a reference point on the trial slip surface.

Where:

R_s: Resisting forces

F_s: Mobilized driving forces

W: Slice weight

α : Base inclination angle

c_i: Cohesion at slice base (c′ for effective stress; c_u for total stress analysis)

ϕ_i': Angle of shearing resistance at slice base

N: Normal force on slice base

16.14.5. Bishop Simplified Method

Bishop developed an equilibrium relationship by summing vertical forces in order to determine the normal force at the base of each slice. As a result, the factor of safety appears on both sides of the equation, making the equation nonlinear. Therefore, an iterative procedure is required to compute the factor of safety.

$$FS = \frac{\dfrac{\sum_i c_i.L_i + N_i \tan\phi_i}{m_\alpha}}{\sum_i W_i \,\mathrm{Sin}\,\alpha_i} = \frac{\dfrac{\sum_i R_s}{m_\alpha}}{\sum_i F_s} \tag{331}$$

$$m_\alpha = \cos\alpha + \frac{\sin\alpha.\tan\phi}{FS} \tag{332}$$

$$N_i = \left(W_i - u_i * L_i\right) * \cos\alpha \tag{333}$$

16.15. Laboratory Tests

Laboratory testing is performed to obtain the engineering properties of soils. SETAF2018 uses parameters from the results of testing to present calculations and graphical analyses in accordance with related theories.

16.15.1. Consolidation Test

This test is performed on a fully saturated, undisturbed cylindrical specimen whose lateral expansion is prevented. The amount and rate of compression under a linearly increasing vertical load are monitored while drainage is allowed from the top and bottom.

Compressibility and the rate of consolidation are calculated from the data obtained during the test. The calculated values enable the prediction of the settlement of structures.

Data Entered for Computation

Sample Information:

- Specimen identification
- Specific gravity of solids
- Depth of sampling

Oedometer Data:

- Micrometer(Dial Gauge) factor
- Halka ağırlığı Weight of oedometer ring
- Height of ring
- Diameter of ring

Data before test:

- Weight of ring+Sample

Data during test:

- Dial gauge readings at the end of 24-hour loading
- Dial gauge readings during each loading step

Value determined after the end of the test:

- Weight of oven-dried sample

SETAF2018 calculates and reports soil properties separately for both pre-test and post-test based on the entered data. The following results are obtained in this context:

- Weight of sample
- Weight of water
- Mass of dry soil
- Moisture content
- Natural unit weight

- Dry Unit weight
- Degree of saturation
- Height of sample
- Equivalent solid height
- Equivalent void height
- Equivalent water height
- Void ratio
- Change in void height
- Virgin line
- Time-compression curves
- Consolidation parameters

Constructing the Virgin Line(curve)

A compression reading is taken after 24 hours for each test to indicate the completion of consolidation at that specific pressure. Frequent readings are taken during the 24 hours to monitor the rate of consolidation.

Table 6. Calculation for Virgin Compression Curve (VCL)

No	σ [kPa]	Reading	ΔH	Actual [mm]	H_e [mm]	e	a_v [m²/kN]	m_v[m²/kN]
1	25	0	0	0	8,368	0,719	0	0
2	50	1	1	0,001	8,367	0,719	1,2E-05	6,9E-06
3	100	2	1	0,001	8,366	0,719	4,2E-06	2,4E-06
4	200	140	138	0,138	8,228	0,707	0,00012	6,8E-05
5	400	448	308	0,308	7,92	0,681	0,00013	7,7E-05
6	800	842	394	0,394	7,526	0,647	8,5E-05	5,1E-05
7	200	676	-166	-0,166	7,692	0,661	2,4E-05	1,4E-05
8	800	924	248	0,248	7,444	0,64	3,5E-05	2,1E-05
9	1600	1264	340	0,34	7,104	0,611	3,7E-05	2,2E-05
10	3200	1702	438	0,438	6,666	0,573	2,4E-05	1,5E-05

Table 6 illustrates the calculations used to obtain the virgin consolidation line (curve). A pressure interval of 25 to 3200 kPa was utilized for this purpose. Where:

- σ: Pressure

- Reading: Dial gauge(micrometer) reading at 24hours
- ΔH: Difference in dial gauge readings
- Actual: Converted value of micrometer readings to units used in calculation
- H_e: Equivalent void height
- e: Void ratio
- a_v: Coefficient of compression
- m_v: Volumetric coefficient of compression

Calculations proceed as follows:

$$\text{Actual} = \Delta H \times \text{Micrometer Reading} \tag{334}$$

$$\text{Equivalent solid height} = \frac{\text{Weight of dry sample}}{(\text{spec. gravity x area})} \tag{335}$$

$$\text{Equiv. water height} = \frac{\text{Weight of water}}{\text{area}} \tag{336}$$

$$\text{Change in void height} = \text{Last reading} - \text{Zero reading x Micrometer} \tag{337}$$

$$\text{Void Ratio} = \frac{\text{He}}{\text{Equivalent solid height}} \tag{338}$$

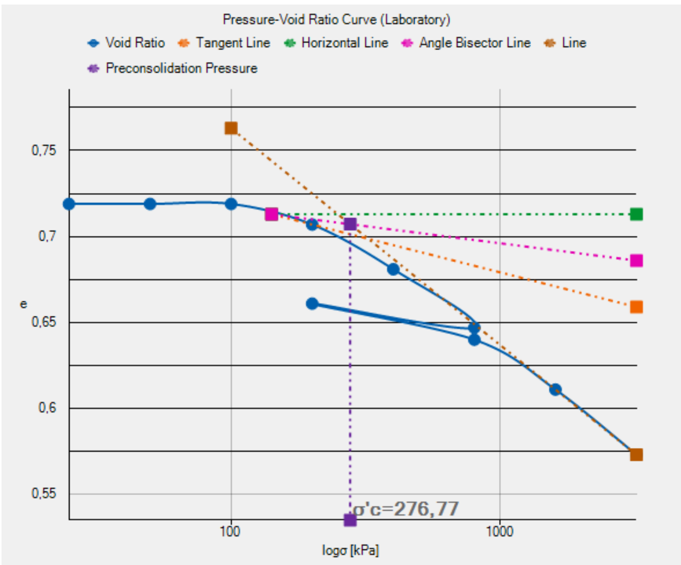


Figure 129. Determination of Preconsolidation Pressure with Casagrande Method

When the point of radius curvature on the curve is selected, the program applies the Casagrande method to determine the preconsolidation pressure and to calculate the compression indices and coefficients (Figure 129).

Calculation for Rate of Consolidation

The programme plots the time-compression curves required for the t_{50} and t_{90} parameters, which determine the speed of consolidation.

Table 7. Time-Compression Calculation

No	t [dk]	√t [dk]	Reading	Compression
1	0	0	140	0
2	0,07	0,265	202	62
3	0,25	0,5	208	68
4	0,5	0,707	214	74
5	1	1	224	84
6	2,25	1,5	234	94
7	4	2	268	128
8	9	3	300	160
9	16	4	326	186
10	36	6	360	220
11	64	8	384	244
12	121	11	420	280
13	420	20,494	444	304
14	1440	37,947	448	308

Coefficient of consolidation:

$$c_v = T_{v_{50}} \frac{(H/2)^2}{t_{50}} \tag{339}$$

In the equation, T_{v50}=0.197 is used for t_{50}, and T_{v50}=0.848 is used for t_{90}.

c_v: Coefficient of consolidation

t_{50}: Time required to reach 50% consolidation.

t_{90}: Time required to reach 90% consolidation

The Casagrande logarithmic and Taylor square root methods are used to estimate t_{50} and t_{90}, respectively.

Logarithmic Method

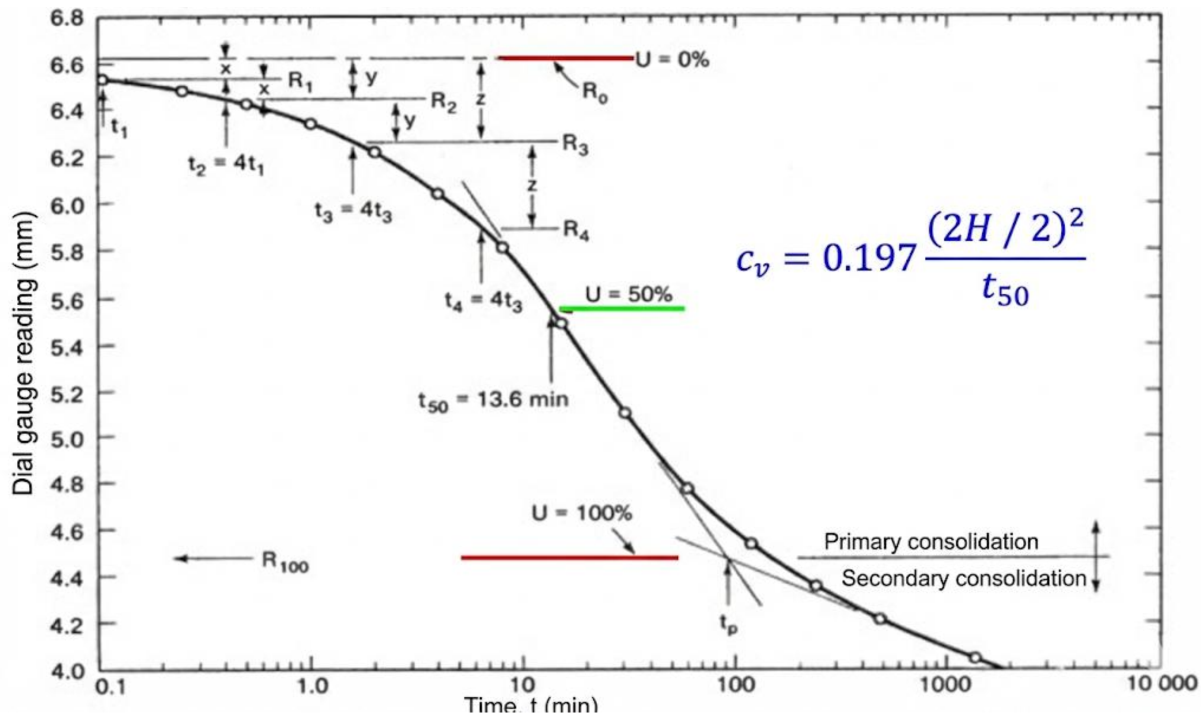


Figure 130. Estimating t_{50} with the Logarithmic Method

Basics of this method are illustrated in Figure 130**.** The program applies this method to the data when required points are selected on the compression curve.

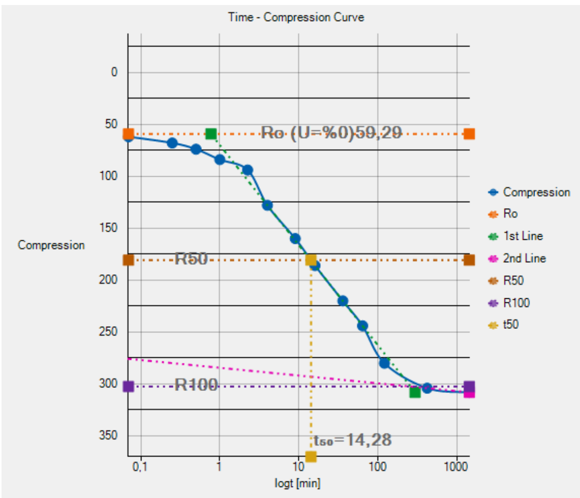


Figure 131. Estimation of t_{50} in the Program

Square Root (Taylor) Method

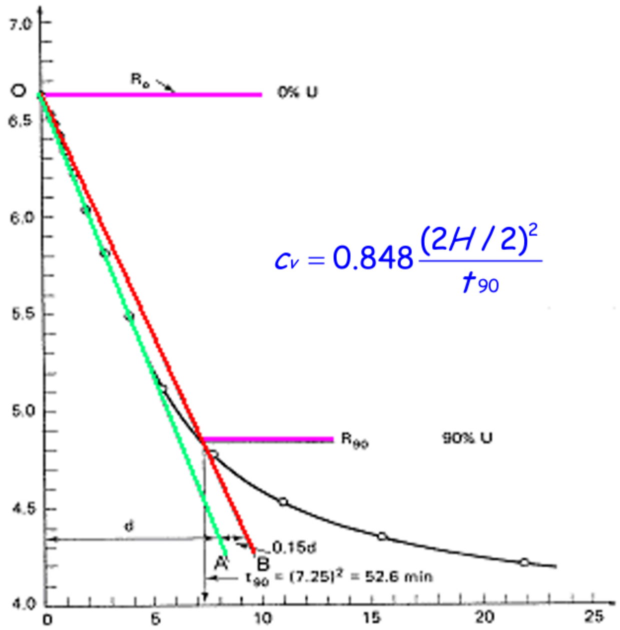


Figure 132. Square Root Method

Figure 132 depicts the square root method to determine the time to reach 90% of consolidation. The program automatically applies this method when necessary points on the compression curve are entered to determine the time taken to reach 90% consolidation (Figure 133).

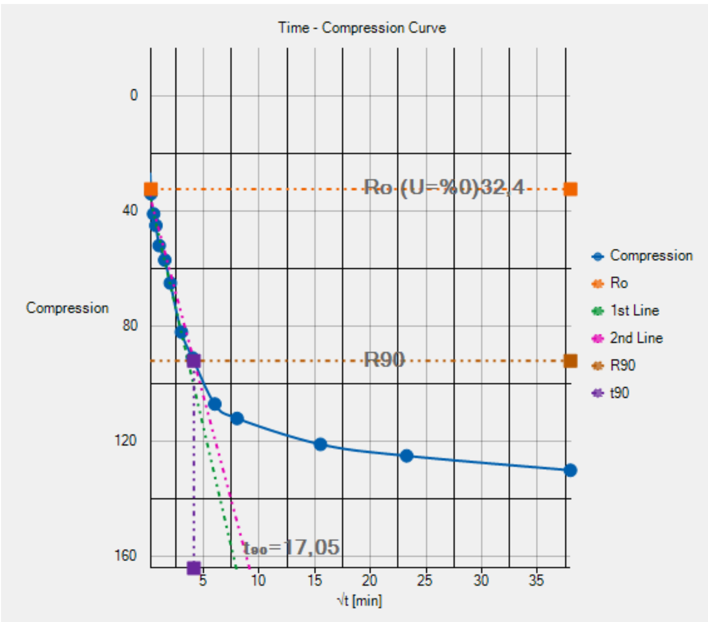


Figure 133. Estimating t_{90} value in the Program

The test report obtained from SETAF2018 provides the following parameter summary.

PARAMETER SUMMARY

Table 4. Consolidation Parameters

Definition	Parameter	Value
Pre-consolidation pressure	σc [kPa]	748,73
Effective Stress	σ'_o [kPa]	180
Overconsolidation ratio	OCR	4,16
Laboratory compression index	Cc	0,166
Laboratory recompression index	Cr	0,025
400kPa loading time for 50% consolidation [min]	t_{50}	5,51
400kPa loading coefficient of consolidation [m^2/day]	cv_{t50}	0,005151
400kPa loading time for 90% consolidation [min]	t_{90}	17,05
400kPa loading coefficient of consolidation [m^2/day]	cv_{t90}	0,007161
800kPa loading time for 50% consolidation [min]	t_{50}	10,82
800kPa loading coefficient of consolidation [m^2/day]	cv_{t50}	0,002621

Şekil 134. Summary Table Generated by the Program

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