DEEPXCAV

A SOFTWARE FOR ANALYSIS AND DESIGN OF RETAINING WALLS

THEORY MANUAL

RELEASE 9.1.1.9 - November 2011
## Table of Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Introduction</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>General Analysis Methods</td>
<td>4</td>
</tr>
<tr>
<td>3</td>
<td>Ground water analysis methods</td>
<td>4</td>
</tr>
<tr>
<td>4</td>
<td>Undrained-Drained Analysis for clays</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>Active and Passive Coefficients of Lateral Earth Pressures</td>
<td>5</td>
</tr>
<tr>
<td>5.1</td>
<td>Active and Passive Lateral Earth Pressures in Conventional Analyses</td>
<td>6</td>
</tr>
<tr>
<td>5.2</td>
<td>Active and Passive Lateral Earth Pressures in PARATIE module</td>
<td>6</td>
</tr>
<tr>
<td>5.3</td>
<td>Passive Pressure Equations</td>
<td>10</td>
</tr>
<tr>
<td>5.4</td>
<td>Classical Earth Pressure Options</td>
<td>11</td>
</tr>
<tr>
<td>5.4.1</td>
<td>Active &amp; Passive Pressures for non-Level Ground</td>
<td>11</td>
</tr>
<tr>
<td>5.4.2</td>
<td>Peck 1969 Earth Pressure Envelopes</td>
<td>13</td>
</tr>
<tr>
<td>5.4.3</td>
<td>FHWA Apparent Earth Pressures</td>
<td>14</td>
</tr>
<tr>
<td>5.4.4</td>
<td>FHWA Recommended Apparent Earth Pressure Diagram for Soft to Medium Clays</td>
<td>16</td>
</tr>
<tr>
<td>5.4.5</td>
<td>FHWA Loading for Stratified Soil Profiles</td>
<td>16</td>
</tr>
<tr>
<td>5.4.6</td>
<td>Modifications to stiff clay and FHWA diagrams</td>
<td>19</td>
</tr>
<tr>
<td>5.4.7</td>
<td>Verification Example for Soft Clay and FHWA Approach</td>
<td>22</td>
</tr>
<tr>
<td>5.4.8</td>
<td>Custom Trapezoidal Pressure Diagrams</td>
<td>24</td>
</tr>
<tr>
<td>5.4.9</td>
<td>Two Step Rectangular Pressure Diagrams</td>
<td>25</td>
</tr>
<tr>
<td>5.5</td>
<td>Vertical wall adhesion in undrained loading</td>
<td>26</td>
</tr>
<tr>
<td>6</td>
<td>Eurocode 7 analysis methods</td>
<td>27</td>
</tr>
<tr>
<td>6.1</td>
<td>Safety Parameters for Ultimate Limit State Combinations</td>
<td>28</td>
</tr>
<tr>
<td>6.2</td>
<td>Automatic generation of active and passive lateral earth pressure factors in EC7 type approaches.</td>
<td>35</td>
</tr>
<tr>
<td>6.3</td>
<td>Determination of Water Pressures &amp; Net Water Pressure Actions in the new software (Conventional Limit Equilibrium Analysis)</td>
<td>36</td>
</tr>
<tr>
<td>6.4</td>
<td>Surcharges</td>
<td>37</td>
</tr>
<tr>
<td>6.5</td>
<td>Line Load Surcharges</td>
<td>38</td>
</tr>
<tr>
<td>6.6</td>
<td>Strip Surcharges</td>
<td>40</td>
</tr>
<tr>
<td>6.7</td>
<td>Other 3D surcharge loads</td>
<td>40</td>
</tr>
<tr>
<td>7</td>
<td>Analysis Example with EC7</td>
<td>42</td>
</tr>
<tr>
<td>8</td>
<td>Ground anchor and helical anchor capacity calculations</td>
<td>61</td>
</tr>
<tr>
<td>8.1</td>
<td>Ground Anchor Capacity Calculations</td>
<td>61</td>
</tr>
<tr>
<td>8.2</td>
<td>Helical anchor capacity calculations</td>
<td>67</td>
</tr>
<tr>
<td>Section</td>
<td>Title</td>
<td>Page</td>
</tr>
<tr>
<td>---------</td>
<td>-----------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>9</td>
<td>Geotechnical Safety Factors</td>
<td>71</td>
</tr>
<tr>
<td>9.1</td>
<td>Introduction</td>
<td>71</td>
</tr>
<tr>
<td>9.1.1</td>
<td>Introduction</td>
<td>71</td>
</tr>
<tr>
<td>9.1.2</td>
<td>Cantilever Walls (conventional analysis)</td>
<td>72</td>
</tr>
<tr>
<td>9.1.3</td>
<td>Walls supported by a single bracing level in conventional analyses.</td>
<td>73</td>
</tr>
<tr>
<td>9.1.4</td>
<td>Walls supported by a multiple bracing levels (conventional analysis)</td>
<td>73</td>
</tr>
<tr>
<td>9.2</td>
<td>Clough Predictions &amp; Basal Stability Index</td>
<td>74</td>
</tr>
<tr>
<td>9.3</td>
<td>Ground surface settlement estimation</td>
<td>76</td>
</tr>
<tr>
<td>10</td>
<td>Handling unbalanced water pressures in Paratie</td>
<td>78</td>
</tr>
<tr>
<td>11</td>
<td>Wall Types - Stiffness and Capacity Calculations</td>
<td>79</td>
</tr>
<tr>
<td>12</td>
<td>Seismic Pressure Options</td>
<td>83</td>
</tr>
<tr>
<td>12.1</td>
<td>Selection of base acceleration and site effects</td>
<td>84</td>
</tr>
<tr>
<td>12.2</td>
<td>Determination of retaining structure response factor R</td>
<td>85</td>
</tr>
<tr>
<td>12.3</td>
<td>Seismic Thrust Options</td>
<td>87</td>
</tr>
<tr>
<td>12.3.1</td>
<td>Semirigid pressure method</td>
<td>87</td>
</tr>
<tr>
<td>12.3.2</td>
<td>Mononobe-Okabe</td>
<td>88</td>
</tr>
<tr>
<td>12.3.3</td>
<td>Richards Shi method</td>
<td>89</td>
</tr>
<tr>
<td>12.3.4</td>
<td>User specified external</td>
<td>89</td>
</tr>
<tr>
<td>12.3.5</td>
<td>Wood Automatic method</td>
<td>90</td>
</tr>
<tr>
<td>12.3.6</td>
<td>Wood Manual</td>
<td>90</td>
</tr>
<tr>
<td>12.4</td>
<td>Water Behavior during earthquakes</td>
<td>90</td>
</tr>
<tr>
<td>12.5</td>
<td>Wall Inertia Seismic Effects</td>
<td>91</td>
</tr>
<tr>
<td>12.6</td>
<td>Verification Example</td>
<td>92</td>
</tr>
<tr>
<td>13</td>
<td>Verification of free earth method for a 10ft cantilever excavation</td>
<td>96</td>
</tr>
<tr>
<td>14</td>
<td>Verification of 20 ft deep single-level-supported excavation</td>
<td>100</td>
</tr>
<tr>
<td>15</td>
<td>Verification of 30 ft excavation with two support levels</td>
<td>105</td>
</tr>
<tr>
<td>Appendix A</td>
<td>APPENDIX: Verification of Passive Pressure Coefficient Calculations</td>
<td>109</td>
</tr>
<tr>
<td>Appendix B</td>
<td>APPENDIX: Sample Paratie Input File Generated by New Software Program</td>
<td>113</td>
</tr>
</tbody>
</table>
1. **Introduction**
   This document briefly introduces the new DeepXcav-Paratie combined software features, analysis methods, and theoretical background. The handling of Eurocode 7 is emphasized through an example of a simple single anchor wall.

2. **General Analysis Methods**
   The combined DeepXcav-Paratie software is capable of analyzing braced excavations with “conventional” limit-equilibrium-methods and beam on elastic foundations (i.e. the traditional PARATIE engine). An excavation can be analyzed in one of the following sequences:
   a) Conventional analysis only
   b) Paratie analysis only
   c) Combined “Conventional”-Paratie Analysis: 1st Conventional analysis with traditional safety factors stored in memory. Once the traditional analysis is completed, then the Paratie analysis is launched.

3. **Ground water analysis methods**
   The software offers the following options for modeling groundwater:
   a) **Hydrostatic:** Applicable for both conventional and Paratie analysis. In Paratie, hydrostatic conditions are modeled by extending the “wall lining” effect to 100 times the wall length below the wall bottom.
   b) **Simplified flow:** Applicable for both conventional and Paratie analysis. This is a simplified 1D flow around the wall. In the Paratie analysis mode, the traditional Paratie water flow option is employed.
   c) **Full Flow Net analysis:** Applicable for both conventional and Paratie analysis. Water pressures are determined by performing a 2D finite difference flow analysis. In PARATIE, water pressures are then added by the UTAB command. The flownet analysis does not account for a drop in the phreatic line.
   d) **User pressures:** Applicable for both conventional and Paratie analysis. Water pressures defined by the user are assumed. In PARATIE, water pressures are then added by the UTAB command.

   In contrast to PARATIE, conventional analyses do not generate excess pore pressures during undrained conditions for clays.

4. **Undrained-Drained Analysis for clays**
   Clay behavior depends on the rate of loading (or unloading). When “fast” stress changes take place then clay behavior is typically modeled as “Undrained” while slow stress changes or long term conditions are typically modeled as “Drained”.


In the software, the default behavior of a clay type soil is set as “Undrained”. **However, the final Drained/Undrained analysis mode is controlled from the Analysis tab.** The software offers the following Drained/Undrained Analysis Options:

a) **Drained:** All clays are modeled as drained. In this mode, conventional analysis methods use the effective cohesion $c'$ and the effective friction angle $\varphi'$ to determine the appropriate lateral earth pressures. For clays, the PARATIE analysis automatically determines the effective cohesion from the stress state history and from the peak and constant volume shearing friction angles ($\varphi_{peak}'$ and $\varphi_{cv}'$ respectively) and it does not use the defined $c'$ in the soils tab.

b) **Undrained:** All clays are modeled as undrained. In this mode, conventional analysis methods use the Undrained Shear Strength $S_u$ and assume an effective friction angle $\varphi'=0^\circ$ to determine the appropriate lateral earth pressures. For clays, the PARATIE analysis automatically determines the Undrained Shear Strength from the stress state history of the clay element and from the peak and constant volume shearing friction angles ($\varphi_{peak}'$ and $\varphi_{cv}'$ respectively), but limits the upper $S_u$ to the value in the soils tab.

c) **Undrained for only initially undrained clays:** Only clays whose initial behavior is set to undrained (Soils form) are modeled as undrained as described in item b) above. All other clays are modeled as drained.

**In contrast to PARATIE, conventional analyses do not generate excess pore pressures during undrained conditions for clays.**

5. **Active and Passive Coefficients of Lateral Earth Pressures**

The new software offers a number of options for evaluating the “Active” and “Passive” coefficients that depend on the analysis method employed (Paratie or Conventional). An important difference with PARATIE is that the old concept of “Uphill” and “Downhill” side has been changed to “Driving side” and “Resisting Side”. Sections 5.1 and 5.2 present the methods employed in determining active and passive coefficients/pressures in Conventional and Paratie analyses respectively. However, in all cases the methods listed in Table 1 available for computing active and passive coefficients.

<table>
<thead>
<tr>
<th>Method</th>
<th>Active Coefficient</th>
<th>Passive Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Available</td>
<td>Surface angle</td>
</tr>
<tr>
<td>Rankine</td>
<td>Yes</td>
<td>No$^1$</td>
</tr>
<tr>
<td>Coulomb</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Caquot-Kerisel Tabulated</td>
<td>No</td>
<td>-</td>
</tr>
<tr>
<td>Caquot-Kerisel Tabulated</td>
<td>No</td>
<td>-</td>
</tr>
<tr>
<td>Lancellotta</td>
<td>No</td>
<td>-</td>
</tr>
</tbody>
</table>

**Notes:**

1. Rankine method automatically converts to Coulomb if a surface angle or wall friction is included.
2. Seismic effects are added as separately.
5.1 **Active and Passive Lateral Earth Pressures in Conventional Analyses**

In the conventional analysis the software first determines which side is generating driving earth pressures. Once the driving side is determined, the software examines if a single ground surface angle is assumed on the driving and on the resisting sides. If a single surface angle is used then the exact theoretical equation is employed as outlined in. If an irregular ground surface angle is detected then the program starts performing a wedge analysis on the appropriate side. Horizontal ground earth pressures are then prorated to account for all applicable effects including wall friction. It should be noted that the active/passive wedge analyses can take into account flownet water pressures if a flownet is calculated.

The computed active and passive earth pressures are then modified if the user assumes another type of lateral earth pressure distribution (i.e. apparent earth pressure diagram computed from active earth pressures above subgrade, divide passive earth pressures by a safety factor, etc.).

All of the above Ka/Kp computations are performed automatically for each stage. The user has only to select the appropriate wall friction behavior and earth pressure distribution.

5.2 **Active and Passive Lateral Earth Pressures in PARATIE module**

Paratie 7.0 incorporates the active and passive earth pressure coefficients within the soil data. Hence, in the existing Paratie 7.0 even though Ka/Kp are in the soil properties dialog, the user has to manually compute Ka/Kp and include wall friction and other effects (such as slope angle, wall friction). If a slope angle surface change takes place on a subsequent stage, then the existing Paratie user has to manually compute and change Ka/Kp to properly account for all required effects. The new software offers a different, more rationalized approach.

In the new SW the default Ka/Kp (for both \( \phi_{peak}' \) and \( \phi_{cv}' \)) defined in the soils tab are by default computed with no wall friction and for a horizontal ground surface. The user still has the ability to use the default PARATIE engine Ka/Kp by selecting a check box in the settings (Tabulated Butee values). This new approach offers the benefit that the same soil type can easily be reused in different design sections without having to modify the base soil properties. Otherwise, while strongly not recommended, wall friction and ground surface angle can be incorporated within the default Ka/Kp values in the Soil Data Dialog. In general the layout logic in determining Ka and Kp is described in Figure 1.
DeepXcav theory manual: Developed by Ce.A.S. srl, Italy and Deep Excavation LLC, U.S.A.

### Soil Type Dialog/Base Ka-Kp

<table>
<thead>
<tr>
<th>Options</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Default Ka, Kp = Rankine (RECOMMENDED)</td>
</tr>
<tr>
<td>2</td>
<td>Default engine Ka/Kp (Butee) for zero wall friction and horizontal ground gives same numbers as Rankine</td>
</tr>
<tr>
<td>3</td>
<td>User defined Ka/Kp that can include slope and wall friction (NOT RECOMMENDED)</td>
</tr>
</tbody>
</table>

Default KaBase, KpBase defined for each soil type

Enable automatic readjustment of Ka/Kp for slope angle, wall friction etc?

1. Default Option (YES)
   SW automatically determines slope angle, wall friction, and other effects

Options:
- A. Enable Kp changes for seismic effects (Default = Yes)
- B. Enable Ka/Kp changes for slope angle (Default = Yes)
- C. Enable wall friction adjustments (Default = Yes)

For each stage then Options 1.1 and 1.2 are available:

**Sub option 1.1: Prorate base Ka/Kp for slope and other effects (Default)**

- \( \text{Ka} = \text{Kabase} \times \text{Ka}(\text{selected method, slope angle, wall friction}) \)
  - Ka Rankine (i.e. ground slope =0, wall friction = 0)

- \( \text{Kp} = \text{Kpbase} \times \text{Kp}(\text{selected method, slope angle, wall friction, EQ}) \)
  - Kp Rankine (i.e. ground slope =0, wall friction = 0)

**Sub option 1.2: Use Actual Ka/Kp as determined from Stage Methods and Equations (see Table 1)**

- \( \text{Ka} = \text{Ka}(\text{selected method, slope angle, wall friction}) \)
- \( \text{Kp} = \text{Kp}(\text{selected method, slope angle, wall friction, EQ}) \)

3. Examine material changes. The latest Material change property will always override the above equations.

**IMPORTANT LIMITATIONS**

A) Ka/Kp for irregular surfaces is not computed and is treated as horizontal.
B) Seismic thrusts are not included in the default Ka calculations.

*note wall friction can be independently selected on the driving or the resisting side. However, basic wall friction modeling is limited to three options a) Zero wall friction, b) % of available soil friction, and c) set wall friction angle.*

**Figure 1: Ka/Kp determination options for Paratie module in new software**
When the software detects that the user is running an analysis with non-horizontal soil layers (i.e. custom line mode is turned on), the software will calculate the appropriate active and passive lateral earth pressure coefficients by performing a series of wedge analysis. Each wedge analysis is performed at the bottom elevation of each layer by assuming a linear wedge failure with no wall friction. Then, if wall friction is assumed, the Ka and Kp values are prorated by the ratio of the horizontal layer Ka with the selected method (Coulomb, Caquot, etc) to the Rankine Ka or Kp values. Last, the computed Ka and Kp coefficients are multiplied or divided by the appropriate partial safety factors if an EC7 type approach is selected. For clays, the wedge analysis is performed for both the peak and the constant volume friction angle Ka and Kp values (KaCV, KpCV, KaPeak, KpPeak). More information about the wedge analysis equations is presented in sections 5.4 and 5.5.

In most cases this approach yields good rough approximations to the actual Ka and Kp values in complex geological stratigraphies. However, results should be more closely inspected as in some conditions more conservative coefficients may be generated when block type failures are initiated. In these cases, it might be more appropriate to define a custom increased Ka and decreased Kp from the soils input dialog and the Resistance Tab. The program offers a way to quickly inspect the wedge analysis values (without the wall friction prorating) by typing the following commands in the Command Prompt text box:

- GEN WEDGES 0 LEFT = Generates the equivalent Ka and Kp for the left side of the left wall
- GEN WEDGES 0 RIGHT = Generates the equivalent Ka and Kp for the right side of the left wall
- GEN WEDGES 1 LEFT = Generates the equivalent Ka and Kp for the left side of the right wall
- GEN WEDGES1 RIGHT = Generates the equivalent Ka and Kp for the right side of the right wall

**Note:** The commands can be executed only when the excavation section has been analyzed at-least once.

The following example presents a case where the previous commands were verified with no wall friction.

![Figure 2.1.a: Wedge analysis example for non-linear analysis](image-url)
In this example the layer properties are:

<table>
<thead>
<tr>
<th>Name</th>
<th>$g_{tot}$</th>
<th>$g_{dry}$</th>
<th>Frict</th>
<th>C'</th>
<th>Su</th>
<th>Frp</th>
<th>FrpCV</th>
<th>Elact</th>
<th>Eur</th>
<th>kAp</th>
<th>kPp</th>
<th>kAcv</th>
<th>kPcv</th>
<th>Vary</th>
<th>Spring</th>
<th>Color</th>
</tr>
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<tbody>
<tr>
<td>Fill San</td>
<td>125</td>
<td>121.02</td>
<td>30</td>
<td>0</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>300</td>
<td>900</td>
<td>0.33</td>
<td>3</td>
<td>N/A</td>
<td>N/A</td>
<td>True</td>
<td>EXP</td>
<td></td>
</tr>
<tr>
<td>Clay</td>
<td>125</td>
<td>120</td>
<td>23</td>
<td>1000</td>
<td>1000</td>
<td>19.5</td>
<td>28</td>
<td>400</td>
<td>1200</td>
<td>0.5</td>
<td>2</td>
<td>0.56</td>
<td>2.77</td>
<td>True</td>
<td>EXP</td>
<td></td>
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<tr>
<td>Sand</td>
<td>155</td>
<td>130</td>
<td>35</td>
<td>0</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>400</td>
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<td>3.59</td>
<td>N/A</td>
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<td>Weath</td>
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<td>N/A</td>
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<td>Linear</td>
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<td>4000</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>900</td>
<td>2700</td>
<td>0.49</td>
<td>2.04</td>
<td>N/A</td>
<td>N/A</td>
<td>True</td>
<td>Linear</td>
<td></td>
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</tbody>
</table>

For the Gen Wedges 0 Right command the following message will be produced:

![Wedge Ka Test](image)

For sand layers the reported KaWedge and KpWedge is also reported in the CV and Peak Values. For clays, the KaWedge and KpWedge represent the values used in the limit equilibrium analysis, while the KaCV, KpCV, KaPK, KpPK represent the values used in the non-linear analysis (Paratie engine). Negative and zero values are reported when a layer is not intersected by the wall. Upon closer inspection of the previously presented results one can see that the calculated Ka and Kp values are very close to the theoretical horizontal Ka and Kp Rankine values. Some expected small differences are also observed but these are expected because the typically used Ka and Kp equations for multilayered soils assume a step-wise wedge failure (which is also a rough approximation) whereas the wedge analysis assumes a single angle from the wall to the surface.
5.3 Passive Pressure Equations

This section outlines the specific theoretical equations used for determining the passive lateral earth pressure coefficients within the software.

a) Rankine passive earth pressure coefficient: This coefficient is applicable only when no wall friction is used with a flat passive ground surface. This equation does not account for seismic effects.

\[ K_p = \frac{1 + \sin(\varphi)}{1 - \sin(\varphi)} \]

b) Coulomb passive earth pressure coefficient: This coefficient can include effects of wall friction, inclined ground surface, and seismic effects. The equation is described by Das on his book “Principles of Geotechnical Engineering”, 3rd Edition, pg. 430 and in many other textbooks:

\[ K_p = |\cos^2(\varphi + \theta - \beta)(1 - ay)\cos^2(\theta)\cos(\beta - \theta + \beta)| \left[ 1 - \frac{\sin(\beta + \varphi)\sin(\beta + \alpha - \beta)}{\cos(\beta + \alpha - \beta)\cos(\alpha - \theta)} \right] \]

Where
- \( \alpha = \) Slope angle (positive upwards)
- \( \beta = \) Seismic effects = \( \tan^{-1} \left( \frac{ax}{1 - ay} \right) \) with
- \( ax = \) horizontal acceleration (relative to \( g \))
- \( ay = \) vertical acceleration, +upwards (relative to \( g \))
- \( \theta = \) Wall angle from vertical (0 radians wall face is vertical)

\[ K_{ph} = K_p \cdot \cos(\theta - \theta) \]

c) Lancellotta: According to this method the passive lateral earth pressure coefficient is given by:

\[ K_{ph} = K_{ps} \cdot \theta \cdot \cos(\alpha - \beta) \]

\[ K_{ps} = \left[ \frac{\cos(\beta)(\cos(\beta) + \sqrt{\sin^2(\beta) - \sin^2(\alpha - \beta)})}{\cos(\alpha - \beta) - \sqrt{\sin^2(\beta) - \sin^2(\alpha - \beta)}} \right] \cdot \tan(\varphi) \]

Where
- \( \beta = \) \( \sqrt{(1 - ay)^2 + (1 + ax)^2} \)

And
- \( 2\beta = \sin^{-1} \left( \frac{\sin(\varphi)}{\sin(\varphi)} \right) + \sin^{-1} \left( \frac{\sin(\alpha - \beta)}{\sin(\varphi)} \right) + \theta + (\alpha - \beta) + 2\beta \)

d) Caquot-Kerisel (Tab-buttee): Refer to manual by Paratie and tabulated values.
5.4 Classical Earth Pressure Options

5.4.1 Active & Passive Pressures for non-Level Ground

Occasionally non-level ground surfaces and benches have to be constructed. The current version of DeepXcav can handle both single angle sloped surfaces (i.e. single 10 degree slope angle) and complex benches with multiple points. DeepXcav automatically detects which condition applies. For single angle slopes, DeepXcav will determine use the theoretical Rankine, Coulomb, or Caquot-Kerisel active, or passive lateral thrust coefficients (depending on user preference).

For non-level ground that does not meet the single slope criteria, DeepXcav combines the solutions from a level ground with a wedge analysis approach. Pressures are generated in a two step approach: a) first, soil pressures are generated pretending that the surface is level, and then b) soil pressures are multiplied by the ratio of the total horizontal force calculated with the wedge method divided by the total horizontal force generated for a level ground solution. This is done incrementally at all nodes throughout the wall depth summing forces from the top of the wall. Wall friction is ignored in the wedge solution but pressures with wall friction according to Coulomb for level ground are prorated as discussed.

This approach does not exactly match theoretical wedge solutions. However, it is employed because it is very easy with the iterative wedge search (as shown in the figure below) to miss the most critical wedge. Thus, when lateral active or passive pressures have to be backfigured from the total lateral force change a spike in lateral pressure can easily occur (while the total force is still the same). Hence, by prorating the active-passive pressure solution a much smoother pressure envelope is generated. In most cases this soil pressure envelope is very close to the actual critical wedge solution. The wedge methods employed are illustrated in the following figures.

Surcharge loads are not considered in the wedge analyses since surcharge pressures are derived separately using well accepted linear elasticity equations.
Figure 2.1: Active force wedge search solution according to Coulomb.

Figure 2.2: Passive force wedge search solution according to Coulomb.
5.4.2 Peck 1969 Earth Pressure Envelopes

After observation of several braced cuts Peck (1969) suggested using apparent pressure envelopes with the following guidelines:

\[ p = 0.65 K a \gamma h \quad p = \gamma h \left(1 - \frac{4c}{\gamma h}\right) \quad p = 0.3 \gamma h \]

\[ \text{for} \quad \frac{4c}{\gamma h} > 1 \]

\( \gamma \) is taken as the effective unit weight while water pressures are added separately (private communication with Dr. Peck).

**Figure 2.3: Apparent Earth Pressures as Outlined by Peck, 1969**

For mixed soil profiles (with multiple soil layers) DeepXcav computes the soil pressure as if each layer acted only by itself. After private communication with Dr. Peck, the unit weight \( g \) represents either the total weight (for soil above the water table) or the effective weight below the water table. For soils with both frictional and undrained behavior, DeepXcav averages the "Sand" and "Soft clay" or "Stiff Clay" solutions. Note that the \( K a \) used in DeepXcav is only for flat ground solutions. The same effect for different \( K a \) (such as for sloped surfaces), can be replicated by creating a custom trapezoidal redistribution of active soil pressures.
5.4.3 FHWA Apparent Earth Pressures

The current version of DeepXcav also includes apparent earth pressure with FHWA standards (Federal Highway Administration). The following few pages are reproduced from applicable FHWA standards.

\[
p = \frac{\text{TOTAL LOAD}}{2/3 \ H} \approx K_a \gamma H
\]

(a) Walls with one level of ground anchors

\[
p = \frac{\text{TOTAL LOAD}}{H - 1/3 \ H_1 - 1/3 \ H_{n+1}}
\]

(b) Walls with multiple levels of ground anchors

- \( H_1 \) = Distance from ground surface to uppermost ground anchor
- \( H_{n+1} \) = Distance from base of excavation to lowermost ground anchor
- \( T_{hi} \) = Horizontal load in ground anchor \( i \)
- \( R \) = Reaction force to be resisted by subgrade (i.e., below base of excavation)
- \( p \) = Maximum ordinate of diagram

**Figure 2.4:** Recommended apparent earth pressure diagram for sands according to FHWA
TOTAL LOAD (kN/m/meter of wall) = 3H² to 6H² (H in meters)

**Figure 2.5: Recommended apparent earth pressure diagram for stiff to hard clays according to FHWA.**

In both cases for figures 2.4 and 2.5, the maximum pressure can be calculated from the total force as:

a. For walls with one support: \( p = \frac{2 \times \text{Load}}{H + H/3} \)
b. For walls with more than one support: \( p = \frac{2 \times \text{Load}}{2H - 2(H_3 + H_{n+1})/3} \)
5.4.4 FHWA Recommended Apparent Earth Pressure Diagram for Soft to Medium Clays

Temporary and permanent anchored walls may be constructed in soft to medium clays (i.e. $N_s>4$) if a competent layer of forming the anchor bond zone is within reasonable depth below the excavation. Permanently anchored walls are seldom used where soft clay extends significantly below the excavation base.

For soft to medium clays and for deep excavations (and undrained conditions), the Terzaghi-Peck diagram shown in figure 2.5 has been used to evaluate apparent earth pressures for design of temporary walls in soft to medium clays. For this diagram apparent soil pressures are computed with a “coefficient”:

$$R_a = 1 - m \frac{4.5}{yH}$$

Where $m$ is an empirical factor that accounts for potential base instability effects in deep excavations is soft clays. When the excavation is underlain by deep soft clay and $N_s$ exceeds 6, $m$ is set to 0.4. Otherwise, $m$ is taken as 1.0 (Peck, 1969). Using the Terzaghi and Peck diagram with $m=0.4$ in cases where $N_s>6$ may result in an underestimation of loads on the wall and is therefore not conservative. In this case, the software uses Henkel’s equation as outlined in the following section.

An important realization is that when $N_s>6$ then the excavation base essentially undergoes basal failure as the basal stability safety factor is smaller than 1.0. In this case, significant soil movements should be expected below the excavation that are not captured by conventional limit equilibrium analyses and may not be included in the beam-on-elastoplastic simulation (Paratie).

The software in the case of a single soil layer will use the this equation if $N_s>4$ and $N_s<=6$.

5.4.5 FHWA Loading for Stratified Soil Profiles

The apparent earth pressure diagrams described above were developed for reasonably homogeneous soil profiles and may therefore be difficult to adapt for use in designing walls in stratified soil deposits. A method based on redistributing calculated active earth pressures may be used for stratified soil profiles. This method should not be used for soil profiles in which the critical potential failure surface extends below the base of the excavation or where surcharge loading is irregular. This method is summarized as follows:

- Evaluate the active earth pressure acting over the excavation height and evaluate the total load imposed by these active earth pressures using conventional analysis methods for evaluating the active earth pressure diagram assuming full mobilization of soil shear strength. For an irregular ground surface the software will perform a trial wedge stability analysis to evaluate the total active thrust.
- The total calculated load is increased by a factor, typically taken as 1.3. A larger value may be used where strict deformation control is desired.
- Distribute the factored total force into an apparent pressure diagram using the trapezoidal distribution shown in Figure 2.4.

Where potential failure surfaces are deep-seated, limit equilibrium methods using slope stability may be used to calculate earth pressure loadings.

The Terzaghi and Peck (1967) diagrams did not account for the development of soil failure below the bottom of the excavation. Observations and finite element studies have demonstrated that soil failure below the excavation bottom can lead to very large movements for temporary retaining walls in soft clays. For \( N_s > 6 \), relative large areas of retained soil near the excavation base are expected to yield significantly as the excavation progresses resulting in large movements below the excavation, increased loads on the exposed portion of the wall, and potential instability of the excavation base. In this case, Henkel (1971) developed an equation to directly obtain \( K_a \) for obtaining the maximum pressure ordinate for soft to medium clays apparent earth pressure diagrams (this equation is applied when FHWA diagrams are used and the program examines if \( N_s > 6 \)):

\[
K_a = 1 - m \cdot \frac{4 \cdot S_u}{\gamma H} + 2 \sqrt{2} \frac{d}{H} \left( 1 - \frac{5.14 S_{ub}}{\gamma H} \right)
\]

Where \( m = 1 \) according to Henkel (1971). The total load is then taken as:

\[
F = 0.8 \cdot K_a \cdot \gamma \cdot H^2
\]

Figure 2.6: Henkel’s mechanism of base failure

Figure 2.7 shows values of \( K_a \) calculated using Henkel’s method for various \( d/H \) ratios. For results in this figure \( S_u = S_{ub} \). This figure indicates that for \( 4 < N_s < 6 \), the Terzaghi and Peck envelope with \( m = 0.4 \) is overly conservative relative to Henkel. Also, for \( N_s < 5.14 \) the Henkel equation is not valid and apparent earth pressures calculated using \( m = 1.0 \) in the Terzaghi and Peck envelope are unrealistically low. For the range \( 4 < N_s < 5.14 \), a constant value of \( K_a = 0.22 \) should be used to evaluate the maximum pressure ordinate for the soft to medium clay apparent earth pressure envelope. At the transition between stiff-hard clays to soft-medium clays, i.e. \( N_s \approx 4 \), the total load
using the soft to medium apparent earth pressure diagram with Ka= 0.22 is 0.193 $\gamma$H$^2$ resulting in a maximum pressure $p=0.26$ $\gamma$H. Use of Ka= 0.22, according to FHWA, represent a rational transition value for these cases.

Henkel’s method is limited to cases where the clays soils on the retained side of the excavation and below the excavation can each be reasonably characterized using a constant value for undrained shear strength. Where a more detailed shear strength profile is required, limit equilibrium methods may be used to evaluate the earth pressure loadings on the wall described in section 5.7.3 of the FHWA manual (not performed within the software).

![Figure 2.7: Comparison of apparent lateral earth pressure coefficients with basal stability index (FHWA 2004).](image)

For clays the stability number is defined as:

$$N_s = \frac{\gamma_{total}H}{S_u}$$

Please note that software uses the effective vertical stress at subgrade to find an equivalent soil unit weight. Water pressures are added separately depending on water condition assumptions. This is slightly different from the approach recommended by FHWA, however, after personal communication with the late Dr. Peck, has confirmed that users of apparent earth pressures should use the effective stress at subgrade and add water pressures separately.

By ignoring the water table, or by using custom water pressures, the exact same numerical solution as with the original FHWA method can be obtained.
5.4.6 Modifications to stiff clay and FHWA diagrams

Over the years various researchers and engineers have proposed numerous apparent lateral earth pressure diagrams for braced excavations. Unfortunately, most lateral apparent pressure diagrams have been taken out of context or misused. Historically, apparent lateral earth pressure diagrams have been developed from measured brace reactions. However, apparent earth pressure diagrams are often arbitrarily used to also calculate bending moments in the wall.

In excavations supporting stiff clays, many researchers have observed that the lower braces carried smaller loads. This has misled engineers to extrapolate the apparent lateral earth pressure to zero at subgrade. In this respect, many apparent lateral earth pressure diagrams carry within them a historical unconservative oversight in the fact that the lateral earth pressure at subgrade was never directly or indirectly measured. Konstantakos (2010) has proven that the zero apparent lateral earth pressure at the subgrade level assumption is incorrect, unconservative, and most importantly unsubstantiated. This historical oversight, can lead to severe underestimation of the required wall embedment length and of the experienced wall bending moments.

If larger displacements can be tolerated or drained conditions are experienced the apparent earth pressure diagrams must not, at a minimum, drop below the theoretical active pressure, unless soil arching is carefully evaluated. Alternatively, in these cases, for fast calculations or estimates, an engineer can increase the apparent earth pressure from 50% at midway between the lowest support level and the subgrade to the full theoretical apparent pressure or the active pressure limit at the subgrade level (see Figure 2.8). As always, these equations represent a simplification of complex conditions.

If tighter deformation control is required or when fully undrained conditions are to be expected, then the virtual reaction at the subgrade level has to take into account increased lateral earth pressures that can even reach close to fifty percent of the total vertical stress at the subgrade level. The initial state of stress has to be taken into consideration as overconsolidated soil strata will tend to induce larger lateral earth stresses on the retaining walls. In such critical cases, a design engineer must always compliment apparent earth pressure diagram calculations with more advanced and well substantiated analysis methods.

The above modifications can be applied within the software by double clicking on the driving earth pressure button when the FHWA or Peck method is selected.
Figure 2.8: Minimum lateral pressure option for FHWA and Peck apparent pressure diagrams (check box).
Figure 2.9: Proposed modifications to stiff clay and FHWA apparent lateral earth pressure diagrams (Konstantakos 2010).

**Simplified Apparent Reaction At Subgrade**

Large displacements tolerated for clays and/or partially drained conditions experienced

| Case (A): Rsub = 0.75p x H4/3 = 0.25 p H4 |
| Case (B): Rsub depends on dimensions |
| Case (C): Rsub = 0.75p x 0.125 H |
| Rsub = 0.094 p H |

Undrained conditions and/or strict deformation control Case (D)

| Average: Rsub = 0.5 p H4 |
| Lower limit: Rsub = 0.417 p H4 |
| Upper limit: Rsub = (0.275 \( \gamma \) H - 0.083p) H4 |

\( \gamma = \) Saturated soil unit weight

Limitations:
1. To be used as a first order approximation
2. Excavations fully dewatered for clays on both wall sides.
5.4.7 Verification Example for Soft Clay and FHWA Approach

A 10m deep excavation is constructed in soft clays and the wall is embedded 2m in a second soft clay layer. The wall is supported by three supports at depths of 2m, 5m, and 8m from the wall top. Assumed soil properties are:

- Clay 1: From 0 to 10m depth, $S_u = 50$ kPa, $\gamma = 20$ kN/m$^3$
- Clay 2: From 10m depth and below $S_u = 30$ kPa, $\gamma = 20$ kN/m$^3$

The depth to the firm layer from the excavation subgrade is assumed as $d=10$ m (which is the model base, i.e. the bottom model coordinate).

The total vertical stress at the excavation subgrade is:

$$\sigma'_v = 20 \text{ kN/m}^3 \times 10 \text{m} = 200 \text{ kPa}$$

The basal stability safety factor is then:

$$FS = \frac{\sigma'_v}{20 \text{ kPa}} = 0.855 \text{ (verified from Fig. 2.10)}$$

Then according to Henkel $K_a$ is calculated as ($m=1$):

$$K_a = 1 - \frac{4 \cdot 5 \text{ kPa}}{200 \text{ kPa}} + 2 \sqrt{\frac{d}{H}} \left(1 - \frac{5.14 \cdot S_u}{\gamma \cdot H}\right)$$

$$K_a = 1 - \frac{4 \times 5 \text{ kPa}}{200 \text{ kPa}} + 2 \sqrt{\frac{10 \text{m}}{10 \text{m}}} \left(1 - \frac{5.14 \times 30 \text{ kPa}}{200 \text{ kPa}}\right) = 0.647$$

The total thrust above the excavation is then: $P_{\text{total}} = 0.5 \cdot K_a \cdot \sigma'_v \cdot H = 647 \text{ kN/m}$

The maximum earth pressure ordinate is then:

$$p = 2 \times \text{Load} / (2H - 2(H_1 + H_{n+1})/3)) = 2 \times 647 \text{ kN/m} / (2 \times 10 \text{m} - 2 \times (2 \text{m} + 2 \text{m})/3) = 74.65 \text{ kPa}$$
The software calculates 74.3 kPa and essentially confirms the results as differences are attributed to rounding errors.

The tributary load in the middle support is then 3m x 74.3kPa = 222.9 kN/m (which is confirmed by the program). When performing only conventional limit equilibrium analysis it is important to properly select the number of wall elements that will generate a sufficient number of nodes. In this example, 195 wall nodes are assumed. In general it is recommended to use at least 100 nodes when performing conventional calculations while 200 nodes will produce more accurate results.

Now examine the case if the soil was a sand with a friction angle of 30 degrees.

Figure 2.11: Verification example for FHWA soft clay analysis

In this case, the total active thrust is calculated as: \( P_{total} = 0.65 \times K_A \times \sigma' \times H = 432.9 \text{ kN/m} \)

The maximum earth pressure ordinate is then:

\[
p = 2 \times \text{Load} / (2 \times H - 2(H_1 + H_{n+1})/3) = 2 \times 432.9 \text{ kN/m} / (2 \times 10m - 2 (2m + 2m)/3) = 49.95 \text{ kPa}
\]

This apparent earth pressure value is confirmed by the software.

Next, we will examine the same excavation with a mixed sand and clay profile.

Sand: From 0 to 5m depth, \( \phi = 30^\circ \) \( \gamma = 20 \text{ kN/m}^3 \)
Clay 1: From 5 to 10m depth, \( Su = 50 \text{ kPa} \) \( \gamma = 20 \text{ kN/m}^3 \)
Clay 2: From 10m depth and below \( Su = 30 \text{ kPa} \) \( \gamma = 20 \text{ kN/m}^3 \)

In this example, \( N_s =6.67 \). As a result we will have to use Henkel's equation but average the effects of soil friction and cohesion. This method is a rough approximation and should be used with caution.

From 0m to 5m the friction force on a vertical face in Sand 1 can be calculated as:

\[
F_{\text{friction}} = 0.5 \times 20 \text{ kN/m}^3 \times 5m \times \tan(30 \text{ degrees}) \times 5m = 144.5 \text{ kN/m}
\]
The available side cohesion on layer Clay 1 is: 5m x 50 kPa = 250 kN/m

The total side resistance on the vertical face is then: 250 kN/m + 144.5 kN/m = 395.5 kN/m

The average equivalent cohesion can be computed as:

\[ S_{u, ave} = \frac{395.5 \text{ kN/m} \times 10 \text{ m}}{10 \text{ m}} = 39.55 \text{ kPa} \]

Then according to Henkel Ka is calculated as (m=1):

\[
K_a = 1 - \frac{4 \times S_{u, ave}}{\gamma H} + 2 \sqrt{2} \frac{d}{H} \left( 1 - \frac{5.14 S_{u, ave}}{200 \text{ kPa}} \right) = 0.849
\]

The total thrust above the excavation is then: \( P_{total} = 0.5 K_a \sigma'_v \times H = 849 \text{ kN/m} \)

The maximum earth pressure ordinate is then:

\[ p = 2 \times \text{Load} / \{2 \times (H - 2(H_1 + H_{n+1})/3)\} = 2 \times 647 \text{ kN/m} / \{2 \times 10 \text{ m} - 2 \times (2 \text{ m} + 2 \text{ m})/3\} = 98 \text{ kPa} \]

This result is confirmed by the software that produces 99.2 kPa.

---

**Figure 2.12:** Verification example for FHWA mixed soil profile with soft clay and sand

### 5.4.8 Custom Trapezoidal Pressure Diagrams

With this option the apparent earth pressure diagram is determined as the product of the active soil thrust times a user defined factor. The factor should range typically from 1.1 to 1.4 depending on the user preferences and the presence of a permanent structure. The resulting horizontal thrust is then redistributed as a trapezoidal pressure diagram where the top and bottom triangular pressure heights are defined as a percentage of the excavation height.
5.4.9 Two Step Rectangular Pressure Diagrams

Very often, especially in the US, engineers are provided with rectangular apparent lateral earth pressures that are defined with the product of a factor times the excavation height. Two factors are usually defined M1 for pressures above the water table and M2 for pressures below the water table. M1 and M2 should already incorporate the soil total and effective weight. Use of this option should be carried with extreme caution. The following dialog will appear if the rectangular pressure option is selected in the driving pressures button.

![Calculation Options]

Figure 2.10: Two step rectangular earth pressure coefficients.
5.5 **Vertical wall adhesion in undrained loading**

Short term total stress conditions (i.e. undrained loading) represent the state in the soil before the pore water pressures have had time to dissipate i.e. immediately after construction in a cohesive soil. For total stress the horizontal active and passive pressures are calculated using the following equations:

\[ p_a = K_a \left( \gamma z + q \right) - S_u K_{ac} \]
\[ p_p = K_p \left( \gamma z + q \right) + S_u K_{pc} \]

Where:
- \( (\gamma z+q) \) represents the total overburden pressure
- \( K_a = K_p = 1.0 \) for cohesive soils.

Design \( s_u = s_{umd}/FSS_u \) where \( FSS_u \) is typically 1.5. The earth pressure coefficients, \( K_{ac} \) and \( K_{pc} \), make an allowance for wall /soil adhesion and are derived as follows:

\[ K_{ac} = K_{pc} = 2 \left( 1 + Sw_{max} / S_{udr} \right)^{0.5} \]

According to the Piling Handbook by Arcelor (2005), the limiting value of wall adhesion \( Sw_{max} \) at the soil/sheet pile interface is generally taken to be smaller than the design undrained shear strength of the soil, \( S_{udr} \), by a factor of 2 for stiff clays i.e. \( Sw_{max} = \alpha \times S_{udr} \), where \( \alpha = 0.5 \). Lower values of wall adhesion, however, may be realized in soft clays. In any case, the designer should refer to the design code they are working to for advice on the maximum value of wall adhesion they may use. Currently, these modifications can be used only in conventional limit equilibrium analyses.

<table>
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<th>( \alpha = \frac{Sw_{max}}{S_{ud}} )</th>
<th>Values of ( K_{ac} ) and ( K_{pc} )</th>
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6. **Eurocode 7 analysis methods**

In the US practice excavations are typically designed with a service design approach while a Strength Reduction Approach is used in Europe and in many other parts of the world. Eurocode 7 (strength design, herein EC7) recommends that the designer examines a number of different Design Approaches (DA-1, DA-2, DA-3) so that the most critical condition is determined. In Eurocode 7 soil strengths are readjusted according to the material “M” tables, surcharges and permanent actions are readjusted according to the action “A” tables, and resistances are modified according to the “R” tabulated values. Hence, in a case that may be outlined as “A2” + “M2” + “R2” one would have to apply all the relevant factors to “Actions”, “Materials”, and “Resistances”. A designer still has to perform a service check in addition to all the ultimate design approach cases. Hence, a considerable number of cases will have to be examined unless the most critical condition can be easily established by an experienced engineer. In summary, EC7 provides the following combinations where the factors can be picked from the tables in section 6.1:

- **Design Approach 1, Combination 1:** A1’ “+” M1 “+” R1
- **Design Approach 1, Combination 2:** A2’ “+” M2 “+” R1
- **Design Approach 2:** A1’ “+” M1 “+” R2
- **Design Approach 3:** A1’ “+” A2’ “+” M2 “+” R3

  A1’ = For structural actions or external loads, and A2’ = for geotechnical actions
  EQK (from EC8): M2 “+” R1

(The Italian code DM08 uses DA1-1, DA1-2, and EQK design approach methods only).

In the old Paratie (version 7 and before), the different cases would have been examined in many “Load Histories”. The term “Load History” has been replaced in the new software with the concept of “Design Section”. Each design section can be independent from each other or a Design Section can be linked to a Base Design Section. When a design section is linked, the model and analysis options are directly copied from the Base Design Section with the exception of the Soil Code Options (i.e, Eurocode 7, DM08 etc).

In Eurocode 7, various equilibrium and other type checks are examined:

- **a)** STR: Structural design/equilibrium checks
- **b)** GEO: Geotechnical equilibrium checks
- **c)** HYD: Hydraulic heave cases
- **d)** UPL: Uplift (on a structure)
- **e)** EQU: Equilibrium states (applicable to seismic conditions?)

The new software handles a number of STR, GEO, and HYD checks while it gives the ability to automatically generate “all” Eurocode 7 cases for a model. Unfortunately, Eurocode 7 as a whole is mostly geared towards traditional limit equilibrium analysis. In more advanced analysis methods (such as in Paratie), Eurocode 7 can be handled according to “the letter of the code” only when equal groundwater levels are assumed in both wall sides. However, much doubt exists as to the
most appropriate method to be employed when different groundwater levels have to be modeled. Section 6.1 presents the safety/strength reduction parameters that the new software uses.

6.1 Safety Parameters for Ultimate Limit State Combinations
Table 2.1 lists all safety factors that are used in the new software and also provides the used safety factors according to EC7-2008. The last 4 table columns list the code safety factors for each code case/scenario (i.e. in the first row Case 1 refers to M1, Case 2 refers to M2). Table 2.2 lists the same factors for the Italian code NTC08, while tables 2.3, 2.4, and 2.5 list the safety factors for the Greek, the French, and the German codes respectively.

Last, table 2.6 presents the load combinations employed by AASHTO LRFD 5th edition (2010). AASHTO slightly differs from European standards in that soil strength is not factored.
## Table 2.1: List of safety factors for strength design approach according to Eurocode 7

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<th>Description</th>
<th>Type</th>
<th>Eurocode Parameter</th>
<th>Eurocode Reference</th>
<th>Eurocode Checks</th>
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<th>Case 2 DA-1: comb2. A2+M2+R1</th>
<th>Case 3 DA-2: A1=M1+R2</th>
<th>Case 4 DA-1: A1=M1+R1</th>
<th>EQU: M2+R1</th>
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<td>γₗₚ</td>
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<td>γₗ_Gₑ</td>
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<td>UPL</td>
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<td>A</td>
<td>γₗ_Gₑ on actions</td>
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### Table 2.2: List of safety factors for strength design approach according to Italian NTC 2008 code

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<th>Type</th>
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<th>Eurocode Reference</th>
<th>Eurocode Checks</th>
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<th>Case 4</th>
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<td>γ_M</td>
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<tr>
<td>F_Su</td>
<td>Safety factor on Su</td>
<td>M</td>
<td>γ_M</td>
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<td>γ_G</td>
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<td>γ_G</td>
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</tr>
<tr>
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<tr>
<td>F_ANCH_T</td>
<td>Safety factor for temporary anchors</td>
<td>R</td>
<td>γ_\text{at}</td>
<td>Section A.3.3.4</td>
<td>STR-GEO</td>
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<tr>
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<td>γ_\text{ap}</td>
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<td>F_RES</td>
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<td>R</td>
<td>γ_R,e</td>
<td>Section A.3.3.5 Table</td>
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<td>γ_G</td>
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<td>γ_G</td>
<td>Table A.3. pg 130</td>
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<td>1.00</td>
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<tr>
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<td>A</td>
<td>γ_G,dst</td>
<td>Section A.5</td>
<td>HYD</td>
<td>1.35</td>
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<tr>
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<td>A</td>
<td>γ_G,stab</td>
<td>Table A.17. pg 136</td>
<td>HYD</td>
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<td>-</td>
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<tr>
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<td>UPL</td>
<td>γ_G,dst</td>
<td>Section A.4</td>
<td>UPL</td>
<td>1.10</td>
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<tr>
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<td>Factor for favorable permanent STABILIZING action</td>
<td>UPL</td>
<td>γ_G,stab</td>
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<td>UPL</td>
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<td>multiplication factor applied to driving earth pressures</td>
<td>A</td>
<td>γ_G on actions</td>
<td>Table A.3</td>
<td>STR-GEO</td>
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<tr>
<td>F_Wall</td>
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<td>STR</td>
<td>Model factor for wall capacity</td>
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**Note:** F_Wall is not defined in EC7. These parameters can be used in an LRFD approach consistent with USA codes.
Table 2.3: List of safety factors for strength design approach Greek design code 2007

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<th>Description</th>
<th>Type</th>
<th>Eurocode Parameter</th>
<th>Eurocode Reference</th>
<th>Eurocode Checks</th>
<th>Code Case</th>
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<tr>
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<td>Safety factor on tanFR</td>
<td>M</td>
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<td>STR-GEO</td>
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<tr>
<td>F_C</td>
<td>Safety factor c’</td>
<td>M</td>
<td>$\gamma_m$</td>
<td>Table A.4 pg. 130</td>
<td>STR-GEO</td>
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<tr>
<td>F_Su</td>
<td>Safety factor on Su</td>
<td>M</td>
<td>$\gamma_m$</td>
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<td>1.40</td>
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<td>A</td>
<td>$\gamma_0$</td>
<td>Section A.3.1</td>
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<td>STR-GEO</td>
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<td>STR-GEO</td>
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<td>$\gamma_G$ on actions</td>
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<td>Model factor for wall capacity</td>
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Note: For slope stability the design approach is equivalent to EQU
Table 2.4: Partial safety factors for strength design approach with French codes XP240 and XP220

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<th>Description</th>
<th>Type</th>
<th>Eurocode Parameter</th>
<th>Eurocode Reference</th>
<th>Eurocode Checks</th>
<th>Code Case</th>
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<tr>
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<td>M</td>
<td>γ_m</td>
<td>Section A.3.2</td>
<td>STR-GEO</td>
<td>1.25 1.25 1.25 1.25 1.25 1.25 1.25</td>
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<td>Safety factor c’</td>
<td>M</td>
<td>γ_m</td>
<td>Table A.4 pg 130</td>
<td>STR-GEO</td>
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<tr>
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<td>γ_m</td>
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<td>STR-GEO</td>
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<td>γ_0</td>
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<tr>
<td>F_EQ</td>
<td>Safety factor for seismic pressures</td>
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Note: French code standards are particularly important for soil nailing walls.
Table 2.5: Partial safety factors for strength design approach with German DIN 2005

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Note:  
1. Case 5, 6, and 7 are only used in slope stability analysis in conjunction with cases 1, 2, and 3 respectively.  
2. At-rest earth pressure factor is used in multiplying earth pressures only in limit equilibrium analyses.

Page | 33
Table 2.6: Partial safety factors for strength design approach with AASHTO LRFD 5th edition 2010

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<tr>
<th>Internal SW Parameter</th>
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Note:
1. AASHTO recommends that slope stability analysis is performed only with the Service I combination.
2. At-rest and active earth pressure factors are used in multiplying earth pressures only in limit equilibrium analyses when the user has selected a relevant method.
6.2 **Automatic generation of active and passive lateral earth pressure factors in EC7 type approaches.**

Figure 3 outlines the calculation logic for determining the active and passive lateral earth pressure coefficients. In conventional analyses, the resistance factor is applied by dividing the resisting lateral earth pressures with a safety factor.

1: **Get Base soil strength parameters**

(Slope, wall friction, etc)

2. **Modify soil properties according to the code 'M' case**

3: **Determine base Ka & Kp according**

Section 5.1 for Conventional Analysis
Section 5.2 for Paratie Analysis

4. **Multiply/Divide Ka and Kp by Appropriate Factor**

4.1 **PARATIE ANALYSIS**

\[ Ka_{used} = Ka_{Base} \times FS_{DriveEarth} \]
\[ Kp_{used} = \frac{Kp_{Base}}{F_{RES}} \]

In DA1-1 the software uses internally \( FS_{DriveEarth}=1 \) and standardizes the external loads by \( FS_{DriveEarth} \). Then, at the end of the analysis, wall moments, shear forces, and support reactions are multiplied by \( FS_{DriveEarth} \) and the ultimate design values are obtained:

- Wall moment \( M_{ULT} = M_{CALC} \times FS_{DriveEarth} \)
- Wall Shear \( V_{ULT} = V_{CALC} \times FS_{DriveEarth} \)
- Support Reaction \( R_{ULT} = R_{CALC} \times FS_{DriveEarth} \)

4.2 **CONVENTIONAL ANALYSIS**

\[ Ka_{used} = Ka_{Base} \]
\[ Kp_{used} = Kp_{Base} \]

Determine Initial Driving and Resisting Lateral Earth Pressures

4.2.a: **Final Driving Lateral Earth Pressures** = Initial \( \times FS_{DriveEarth} \)

\( (FS_{DriveEarth} = 1 \text{ EC7, DM08}) \)

4.2.b: **Final Resisting Lateral Earth Pressures** = Initial \( /F_{RES} \)

---

**Figure 3**: Calculation logic for determining \( K_a \) and \( K_p \) and driving and resisting lateral earth pressures.
6.3 Determination of Water Pressures & Net Water Pressure Actions in the new software (Conventional Limit Equilibrium Analysis)

The software program offers two possibilities for determining water actions on a wall when EC7 is employed. In the current approach, the actual water pressures or water levels are not modified.

Option 1 (Default): Net water pressure method

In the default option, the program determines the net water pressures on the wall. Subsequently, the net water pressures are multiplied by F_WaterDR and then the net water pressures are applied on the beam action. The net water pressure results are then stored for reference checks. Hence, this method can be outlined with the following equation:

\[ W_{net} = (W_{drive} - W_{resist}) \times F_{WaterDR} \]

Option 2: Water pressures multiplied on driving and resisting sides (This Option is not yet enabled.)

In this option, the program first determines initial net water pressures on the wall. Subsequently, the net water pressures are determined by multiplying the driving water pressures by F_WaterDR and by multiplying the resisting water pressures. The net water pressure results are then stored for reference checks. Hence, this method can be outlined with the following equation:

\[ W_{net} = W_{drive} \times F_{WaterDR} - W_{resist} \times F_{WaterRES} \]
6.4 Surcharges
The new software enables the user to use a number of different surcharge types. Some of these surcharges are common with PARATIE, however, most surcharge types are not currently included in the Paratie Engine. Table 3 lists the available types of surcharges.

<table>
<thead>
<tr>
<th>Surcharge Type</th>
<th>Permanent/Temporary (P/T)</th>
<th>Exists in Paratie Engine</th>
<th>Exists in Conventional Analysis</th>
<th>Conventional Analysis Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface Line load</td>
<td>P &amp; T</td>
<td>No</td>
<td>Yes</td>
<td>Theory of elasticity. Can include both Horizontal and Vertical components.</td>
</tr>
<tr>
<td>Line load</td>
<td>P &amp; T</td>
<td>No</td>
<td>Yes</td>
<td>Same as above</td>
</tr>
<tr>
<td>Wall Line Load</td>
<td>P &amp; T</td>
<td>No</td>
<td>Yes</td>
<td>Same as above</td>
</tr>
<tr>
<td>Surface Strip Surcharge</td>
<td>P &amp; T</td>
<td>Yes</td>
<td>Yes</td>
<td>Same as above</td>
</tr>
<tr>
<td>Wall strip Surcharge</td>
<td>P &amp; T</td>
<td>Yes</td>
<td>Yes</td>
<td>Same as above</td>
</tr>
<tr>
<td>Arbitrary Strip Surcharge</td>
<td>P &amp; T</td>
<td>No</td>
<td>Yes</td>
<td>Theory of elasticity. Vertical Direction only.</td>
</tr>
<tr>
<td>Footing (3D)</td>
<td>P</td>
<td>No</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>Building (3D)</td>
<td>P</td>
<td>No</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>3D Point Load</td>
<td>P &amp; T</td>
<td>No</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>Vehicle (3D)</td>
<td>T</td>
<td>No</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>Area Load (3D)</td>
<td>P &amp; T</td>
<td>No</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>Moment/Rotation</td>
<td>-</td>
<td>Yes</td>
<td>No</td>
<td></td>
</tr>
</tbody>
</table>

When EC7 (or DM08) is utilized, the following items are worth noting:

a) In the PARATIE module: In the default option the program does not use the Default Paratie Engine for determining surcharge actions, but calculates all surcharges according to the conventional methods.

If the Paratie Simplified Load Options are enabled (Figure 4.1), then all conventional loads are ignored. Only loads that match the Paratie engine criteria are utilized.

b) Unfavorable Permanent loads are multiplied by F\_LP while favorable permanent loads are multiplied by 1.0.

c) Unfavorable Temporary loads are multiplied by F\_LV while favorable temporary loads are multiplied by 0.

The software offers great versatility for calculating surcharge loads on a wall. Surcharges that are directly on the wall are always added directly to the wall. In the default setting, external loads that are not directly located on the wall are always calculated using theory of elasticity equations.

Most formulas used are truly applicable for certain cases where ground is flat or the load is within an infinite elastic mass. However, the formulas provide reasonable approximations to otherwise
extremely complicated elastic solutions. When Poisson's ratio is used the software finds and uses the applicable Poisson ratio of $v$ at each elevation.

![Figure 4.1: Simplified Paratie load options](image1)

![Figure 4.2: Elasticity surcharge options](image2)

### 6.5 Line Load Surcharges

Line loads are defined with two components: a) a vertical $P_y$, and b) a horizontal $P_x$. It is important to note that the many of the equations listed below are, only by themselves, applicable for a load in an infinite soil mass. For this reason, the software multiplies the obtained surcharge by a factor $m$ that accounts for wall rigidity. The software assumes a default value $m=2$ that accounts for full surcharge “reflection” from a rigid behavior. However, a value $m=1.5$ might be a reasonably less conservative assumption that can account for limited wall displacement.

For line loads that are located on the surface (or the vertical component strip loads, since strip loads are found by integrating with line load calculations), equations that include full wall rigidity can be included. This behavior can be selected from the Loads/Supports tab as Figure 4.2 illustrates. In this case, the calculated loads are not multiplied by the $m$ factor.

**For vertical line loads on the surface:** When the *Use Equations with Wall Rigidity* option is not selected, the software uses the Boussinesq equation listed in Poulos and Davis, 1974, Equation 2.7a

\[
D_{R\delta} = \left( D_{x^2} + D_{y^2} \right)^2
\]

Horizontal Surcharge

\[
q_{x\delta} = \frac{2 \cdot P_x \cdot D_y \cdot D_{x^2}}{\pi \cdot D_{R\delta}}
\]
For a vertical surface line load, when the Use Equations with Wall Rigidity option is selected, the software uses the Boussinesq equation as modified by experiment for rigid walls (Terzaghi, 1954).

\[
v_H = 0.20 \frac{Q}{H} \left(1 + \frac{n}{H(0.16 + n^2)^2}\right) \quad \text{(for } m \leq 0.4)\]

\[
P_H = 0.55 Q, \text{ resultant force}
\]

\[
\sigma_H = 1.28 \frac{Q}{H} \left(1 + \frac{n}{H(0.16 + n^2)^2}\right) \quad \text{(for } m > 0.4)\]

\[
P_H = \frac{0.64 Q}{(m^2 + 1)} \text{ resultant force}
\]

**For vertical line loads within the soil mass:** The software uses the Melan’s equation listed in Poulos and Davis, 1974, Equation 2.10b pg. 27

\[
r_1 = \sqrt{\left(\frac{a}{r}\right)^2 + x^2} \quad r_2 = \sqrt{\left(\frac{a}{r}\right)^2 + x^2} \quad \text{and } m = (1 - v)/v
\]

**Horizontal Surcharge**

\[
q_w = \frac{P}{\pi} \left[\frac{m + 1}{m} \left\{\frac{(a - d)x^2}{r_1^4} + \frac{(a + d)x^2 + 2d^2}{r_2^4} + \frac{8d^2(2 + 3)x^2}{r_2^6}\right\} + \frac{m - 1}{4m^3} \left(-\frac{(a - d)}{r_1^2} + \frac{a + 3d}{r_2^2} + \frac{4x^3}{r_2^4}\right)\right]
\]

For the horizontal component of a surface line load: The software uses the integrated Cerruti problem from Poulos and Davis Equation 2.9b

\[
DR4 = (Dx^2 + Dy^2)^2
\]

**Horizontal Surcharge**

\[
q_w = \frac{2 \cdot P}{n \cdot DR4}
\]

**For the horizontal component of a line load within the soil mass:** The software uses Melan’s problem Equation 2.11b pg. 27, from Poulos & Davis

\[
DR2 = Dx^2 + Dy^2
\]

**Horizontal Surcharge**
6.6 Strip Surcharges
Strip loads in the new software can be defined with linearly varying magnitudes in both vertical and horizontal directions. Hence, complicated surcharge patterns can be simulated. Surcharge pressures are calculated by dividing the strip load into increments where an equivalent line load is considered. Then the line load solutions are employed and numerically integrated to give the total surcharge at the desired elevation. The software subdivides each strip load into 50 increments where it performs the integration of both horizontal and vertical loads. On surface loads, the vertical load is calculated from integration along x and not along the surface line.

6.7 Other 3D surcharge loads
The software offers the possibility to include other 3-dimensional surcharges. In essence, all these loads are extensions/integrations of the 3D point vertical load solution.

For 3D footings, the surcharge on the wall can be calculated in two ways:
a) By integrating the footing bearing pressure over smaller segments on the footing footprint. In this case the footing is subdivided into a number of segments and the surcharge calculations are slightly more time consuming.
b) By assuming that the footing load acts as a 3D point load at the footing center coordinates.

For loads that are located on the surface: The software program uses the Boussinesq equation. Results from the following equations are multiplied by the elastic load adjustment factor m as previously described.

\[ r = \sqrt{x^2 + y^2} \]

\[ RR = \sqrt{r^2 + z^2} \]

The radial stress increment is then calculated as:

\[ q_{rr} = -\frac{P_y \cdot \frac{3 \cdot z \cdot r^2}{RR^3} + \frac{1 - 2 \cdot \nu}{RR \cdot (RR + z)}}{2 \cdot \pi} \]

The hoop stress is defined as:

\[ q_{\theta \theta} = -\frac{P_y \cdot (1 - 2 \cdot \nu) \cdot \left( \frac{z}{RR^3} - \frac{1}{RR \cdot (RR + z)} \right)}{2 \cdot \pi} \]

With the angles defined as:
Then, the horizontal component surcharge is:

\[ q_{h} = q_{ph} \cdot \cos(\alpha_{ph}) + q_{pg} \cdot \cos(\alpha_{pg}) \]

For vertical point loads within the soil mass: The software uses the Mindlin solution as outlined by Poulos and Davis, 1974 equations 2.4.a, and 2.4.g

\[ q_{vv} = -\left( \frac{P_{y}}{(S * \pi * (1 - 2 * v))} \right) \left( \frac{(1 - 2 * v) * (z - c) + c)}{R1^n 5 - 3 * x^n 2 + c} \right) \]

6.8 Load behavior and factors when a design approach is used

When an analysis uses design approach such as EC7, each external load must be categorized as favorable or unfavorable. In the default mode when no load combination is used, the software program automatically categorizes loads as favorable or unfavorable based on their location and direction relative to the wall and the excavation. Hence, loads that push the wall towards the excavation are treated as unfavorable, while loads that push the wall towards the retained soil are treated as favorable. In all design approach methods, favorable variable loads are ignored in the analysis while favorable permanent loads are multiplied by a safety factor equal to 1. Unfavorable loads get typically multiplied with factors ranging from 1 to 1.5 depending on the examined design approach and the load nature (permanent vs. variable).

When a load combination is used, the user has the option to manually select the behavior of each load.
7. **Analysis Example with EC7**

A simplified analysis example is presented in this section for the purpose of illustrating use of EC7 methods. The example involves the analysis of steel sheet pile wall supported by a single level of tiebacks with the following assumptions:

- Retained ground surface level (uphill side) El. +200
- Maximum excavation level (downhill side) El. +191
- Water level on retained side El. +195
- Water level on excavated side El. +191
- Water density $\gamma_{\text{WATER}} = 10 \text{kN/m}^3$
- Soil properties: $\gamma_{\text{TOTAL}} = 20 \text{kN/m}^3$, $\gamma_{\text{DRY}} = 19 \text{kN/m}^3$, $c' = 3 \text{kPa}$, $\phi = 32 \text{ deg}$, Exponential soil model: $E_{\text{LOAD}} = 15000 \text{ kPa}$, $E_{\text{RELOAD}} = 45000 \text{ kPa}$, $a_h = 1$, $a_v = 0$
  
  $K_{p\text{Base}} = 3.225$ (Rankine), $K_{a\text{Base}} = 0.307$ (Rankine)
  
  Ultimate Tieback bond capacity $q_{\text{ULT}} = 150 \text{ kPa}$
  
  User specified safety on bond values $FS_{\text{Geo}} = 1.5$

- **Tieback Data:**
  
  Elevation El. +197,
  
  Horizontal spacing = 2m
  
  Angle = 30 deg from horizontal
  
  Prestress = 400 kN (i.e. 200kN/m)

  Structural Properties: 4 strands/1.375 cm diameter each,

  Thus $A_{\text{STEEL}} = 5.94 \text{ cm}^2$

  Steel yield strength $F_y = 1862 \text{ MPa}$

  Fixed body length $L_{\text{FIX}} = 9 \text{ m}$

  Fixed body Diameter $D_{\text{FIX}} = 0.15 \text{ m}$

- **Wall Data:**

  Steel Sheet pile AZ36, $F_y = 355 \text{ MPa}$

  Wall top. El. +200

  Wall length 18m

  Moment of Inertia $I_{xx} = 82795.6 \text{ cm}^4/\text{m}$

  Section Modulus $S_{xx} = 3600 \text{ cm}^3/\text{m}$

- **Surcharge:**

  Variable triangular surcharge on wall

  Pressure 5kPa at El. +200 (top of wall)

  Pressure 0kPa at El. +195

The construction sequence is illustrated in Figures 4.1 through 4.4. For the classical analysis the following assumptions will be made:

- Rankine passive pressures on resisting side

- Cantilever excavation: Active pressures (Free earth analysis)
Final stage: Apparent earth pressures from active x 1.3, redistributed top from 0 kPa at wall top to full pressure at 25% of Hexc., Active pressures beneath subgrade.

Free earth analysis for single level of tieback analysis.

Water pressures: Simplified flow

Figure 5.1: Initial Stage (Stage 0, Distorted Scales)

Figure 5.2: Stage 1, cantilever excavation to El. +196.5 (tieback is inactive)

Figure 5.3: Stage 2, activate and prestress ground anchor at El. +197
Figure 5.4: Stage 3, excavate to final subgrade at El. +191

The first step will be to evaluate the active and passive earth pressures for the service case as illustrated in Figure 5.
Top triangular pressure height= 0.25 m
Hexc = 2.25 m
Hexc = 9 m
Apparent Earth Pressure Factor: 1.3 (times active)

**Eurocode Safety factors**

<table>
<thead>
<tr>
<th>SOIL</th>
<th>DRY UNIT WEIGHT (kPa)</th>
<th>WATER UNIT WEIGHT (kPa)</th>
<th>WATER TABLE ELEV. (m)</th>
<th>φ (deg)</th>
<th>Ka</th>
<th>Kp</th>
<th>c' (kPa)</th>
<th>WATER TABLE ELEV. (m)</th>
<th>Hydraulic travel length m</th>
<th>Hydraulic loss gradient i m/m</th>
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</thead>
<tbody>
<tr>
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<td>10</td>
<td>195</td>
<td>32</td>
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<td>3.255</td>
<td>3</td>
<td>195</td>
<td>22</td>
<td>0.1618</td>
</tr>
</tbody>
</table>

Modified for calculation/Strengthen Reductions

**LEFT EXCAVATION SIDE PRESSURES**

<table>
<thead>
<tr>
<th>ELEV. (m)</th>
<th>TOTAL VERTICAL STRESS (kPa)</th>
<th>WATER PRESSURE (kPa)</th>
<th>EFFECTIVE VERTICAL STRESS (kPa)</th>
<th>Active SOIL STRESS (kPa)</th>
<th>Apparent LATERAL SOIL STRESS (kPa)</th>
<th>Total LATERAL SOIL STRESS (kPa)</th>
<th>TOTAL LATERAL WATER STRESS (kPa)</th>
<th>EFFECTIVE VERTICAL WATER PRESSURE (kPa)</th>
<th>LATERAL WATER STRESS (kPa)</th>
<th>TOTAL LAT WATER STRESS (kPa)</th>
<th>NET LATERAL STRESS (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
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<td>10.82</td>
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<td>191</td>
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<td>0</td>
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</tr>
<tr>
<td>182</td>
<td>355</td>
<td>-106.4</td>
<td>248.64</td>
<td>-73.07</td>
<td>-73.07</td>
<td>-179.43</td>
<td>180</td>
<td>106.4</td>
<td>73.64</td>
<td>250.48</td>
<td>356.84</td>
</tr>
</tbody>
</table>

Total active earth force above subgrade:

\[ \Delta F_x = \sum_{Elev} \Delta F_x \]

From El. 200.00 to El. 199.43 0.0 kN/m
From El. 199.43 to El. 197.75 8.2 kN/m
From El. 197.75 to El. 195.00 49.1 kN/m
From El. 195.00 to El. 191.00 132.5 kN/m

Net Sum = 189.8 kN/m

Factored Force = 246.7 kN/m

Max. Apparent Earth Pressure = 31.3 kPa

**LATERAL STRESS (kPa)**

Figure 6: Calculation of lateral earth and water pressures for service case
As Figure 6 shows, the calculated maximum apparent earth pressure is 31.3 kPa which is very close to the 31.4 kPa apparent earth pressure envelope calculated from the software (Figure 7.1). All other pressure calculations are also very well confirmed (within rounding error accuracy).
Figure 7.2: Simplified flow groundwater pressures from conventional analysis

Figure 7.3: Simplified flow net groundwater pressures from conventional analysis
Figure 7.4: Wall surcharge pressures (unfactored)

Figure 7.5: Wall displacements from conventional analysis (last stage)
Figure 7.6: Shear and moment diagrams with support reaction and stress checks drawn (red lines on moment diagram show wall capacity).

Figure 7.7: Shear and moment diagram envelopes (for current design section only)
Next, the EC7 combination DA-3 approach will be examined in detail. However, all EC7 design approaches will be analyzed simultaneously. The model is linked to the base design section.

The corresponding safety factors are:
- \[ \text{FS}(\tan(\phi)) = 1.25 \]
- \[ \text{FS}(c') = 1.25 \]
- \[ \text{FS}(Su) = 1.5 \text{ (this is also used for the ultimate bond resistance)} \]
- \[ \text{FS}(\text{Actions temp}) = 1.3 \]
- \[ \text{FS}(\text{Anchors}) = 1.1 \]
- \[ \text{FS}(\text{Water Drive}) = 1.0 \]
- \[ \text{FS}(\text{Drive_Earth}) = 1.0 \]

Next the active and passive earth pressures, as well as the net water pressures for the DA3 approach will be calculated as illustrated in Figure 8.2. As Figures 8.3 through 8.4 demonstrate, the software calculates essentially the same lateral earth pressures as the spreadsheet shown in Figure 8.2.
DeepXcav theory manual:  Developed by Ce.A.S. srl, Italy and Deep Excavation LLC, U.S.A.

Top triangular pressure height= 0.25  Hexc = 2.5 m  Hexc= 9 m
Apparent Earth Pressure Factor: 1.3 (times active)

<table>
<thead>
<tr>
<th>SOIL UNIT WEIGHT (kPa)</th>
<th>DRY UNIT WEIGHT (kPa)</th>
<th>WATER UNIT WEIGHT (kPa)</th>
<th>WATER TABLE ELEV. (m)</th>
<th>( \phi ) (deg)</th>
<th>Ka</th>
<th>Kp</th>
<th>c' (kPa)</th>
<th>WATER ELEV. (m)</th>
<th>Hydraulic travel length ( m )</th>
<th>Hydraulic loss gradient ( i ) ( m/m )</th>
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Modified for calculations/Strength Reductions

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<tr>
<th>ELEV. (m)</th>
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<th>UNFACTORED WATER SOIL STRESS (kPa)</th>
<th>EFFECTIVE WATER PRESSURE (kPa)</th>
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<th>TOTAL LATERAL STRESS (kPa)</th>
<th>Net water pressure (factored) (kPa)</th>
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</table>

Total active earth force above subgrade:

\[ \Delta F_x \]

From El. 200.00 to El. 199.59  0.0 kN/m
From El. 199.59 to El. 197.75  12.3 kN/m
From El. 197.75 to El. 195.00  64.2 kN/m
From El. 195.00 to El. 191.00  169.4 kN/m

Sum: 340.1 kN/m

Factored Force: 319.7

Max. Apparent Earth Pressures: 40.8 kPa

---

**Figure 8.2:** Calculation of lateral earth and water pressures for DA3 Approach
Figure 8.2: Apparent lateral earth pressures for DA3 Approach (40.7 kPa pressure verified spreadsheet calculations)

Figure 8.3: Factored lateral surcharge pressures for DA3 Approach (7.5 kPa pressure = 5 kPa x 1.5)
Figure 8.4: Net Factored water pressures for DA3 Approach
32.73 kPa pressure = 32.73 kPa x 1.0, 32.7 kPa from Figure 6.3
Spreadsheet calculation 32.7 kPa

Figure 8.5: Wall shear and moment for DA3 Approach
Next we examine the case of DA1-1 where earth and water pressures are multiplied by safety factors while the soil strength parameters are maintained.

Top triangular pressure height = 0.25 $H_{exc} = 2.25$ m

Apparent Earth Pressure Factor = 1.3 (times active)

<table>
<thead>
<tr>
<th>Eurocode Safety factors</th>
<th>1</th>
<th>1</th>
<th>1</th>
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<table>
<thead>
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<th>SOIL UNIT WEIGHT (kPa)</th>
<th>DRY UNIT WEIGHT (kPa)</th>
<th>WATER UNIT WEIGHT (kPa)</th>
<th>WATER TABLE ELEV. (m)</th>
<th>$\phi$ (deg)</th>
<th>$K_a$</th>
<th>$K_p$</th>
<th>$c'$ (kPa)</th>
<th>$H_{exc}$ (m)</th>
<th>Safety factor on net water pressures</th>
<th>Safety factor on passive Resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>32</td>
<td>0.307</td>
<td>3.295</td>
<td>3</td>
<td>195</td>
<td>22</td>
<td>0.1818</td>
<td>1.35</td>
<td>3</td>
<td>3.000</td>
<td>1.35</td>
</tr>
</tbody>
</table>

Modified for calculation/Strength Reductions

**Figure 8.6:** Calculation of lateral earth and water pressures for DA1-1 Approach
Figure 8.7: Apparent lateral earth pressures for DA1-1 Approach (42.4 kPa pressure verified spreadsheet calculations)

Figure 8.8: Net Factored water pressures for DA1-1 Approach
44.18 kPa pressure = 32.73 kPa x 1.35 , 32.7 kPa from Figure 6.3
Spreadsheet calculation 44.18 kPa

In the following pages, the non-linear solution to the same problem is briefly presented.
Figure 9.1: Wall bending moments and shear forces for Paratie Solution for service case.

Figure 9.2: Wall bending moments and shear forces for Paratie Solution for DA3 case.
Figure 9.3: Net water pressures for Paratie Solution for DA3 case (not yet factored)

Figure 9.4: Wall bending moments and shear forces for Paratie Solution for DA1-1 case.

IMPORTANT For DA1-1:
In Paratie when Water Unfavorable or Earth Unfavorable are greater than 1, wall bending, wall shear, and support reaction results are obtained from an equivalent service analysis approach. In this approach, all surcharge magnitudes are standardized by Earth Unfavorable (1.35 in DA1-1), thus, unfavorable variable loads will be multiplied by 1.5/1.35=1.111 while permanent loads by 1.35/1.35=1. When the analysis is completed the wall moment, wall shear, and support reaction results are multiplied x 1.35. The displacements however are not multiplied.
The tieback STR & GEO capacity calculations will be performed for Case DA1-1:

- \( \gamma_R = 1.1 \) (temporary tieback)
- \( \gamma_{SU} = 1 \) (Shear strength also used for bond values)
- \( \text{FS Geo} = 1.0 \) User specified safety factor in this example, recommended value 1.35 in other conditions.

Fixed body length \( L_{\text{FIX}} = 9 \) m
Fixed body Diameter \( D_{\text{FIX}} = 0.15 \) m
Ultimate Skin friction \( q_{\text{ULT}} = 150 \) kPa

Then the ultimate geotechnical capacity is:

\[
R_{\text{ULT,GEO}} = L_{\text{FIX}} \times \pi \times D_{\text{FIX}} \times q_{\text{ULT}} / (\gamma_R)
\]

\( R_{\text{ULT,GEO}} = 578.33 \) kN per ground anchor

The design geotechnical capacity (for stress check ratios) is calculated as:

\[
R_{\text{DES,GEO}} = L_{\text{FIX}} \times \pi \times D_{\text{FIX}} \times q_{\text{ULT}} / (\gamma_R \times \gamma_{SU} \times \text{FS Geo}) = 578.33 \) kN

The Ultimate Structural capacity can be calculated as:

\[
R_{\text{ULT,STR}} = A_{\text{FIX,STEEL}} \times F_y / (\gamma_M)
\]

Note that \( 1/ \gamma_M = \phi \) in the EC = 0.87

\[
R_{\text{ULT,STR}} = 0.87 \times A_{\text{FIX,STEEL}} \times F_y
\]

\( R_{\text{ULT,STR}} = 0.87 \times 5.94 \text{ cm}^2 \times 1862 \) MPa = 961.8 kN

These results are verified by the software program.
The tieback GEO capacity calculations for Case DA1-2:

\[\gamma_R = 1.1\] (temporary tieback)

\[\gamma_{SU} = 1.4\] (Shear strength also used for bond values)

FS Geo = 1.0 In M2 cases this factor is automatically set to 1.0 in order to produce consistent capacities with available design charts for bond resistance of ground anchors (where an FS=2.0).

Fixed body length \(L_{FIX}\) = 9 m

Fixed body Diameter \(D_{FIX}\) = 0.15m

Ultimate Skin friction \(q_{ULT}\) = 150 kPa

Then the ultimate geotechnical capacity is:

\[R_{ULT,GEO} = L_{FIX} \times \pi \times D_{FIX} \times q_{ULT} / (\gamma_R \times \gamma_{SU} \times \text{FS Geo})\]

\[R_{ULT,GEO} = 578.33\, \text{kN per ground anchor}\]

The design geotechnical capacity (for stress check ratios) is calculated as:

\[R_{DES,GEO} = L_{FIX} \times \pi \times D_{FIX} \times q_{ULT} / (\gamma_R \times \gamma_{SU} \times \text{FS Geo}) = 413.1\, \text{kN}\]
### Figure 9.7: Individual support reactions/capacity for DA1-2

**1. Support Reactions and Loads**
- **Axial:** 1000.2 kN, 500.1 kN/m
- **Lateral Force Flat:** 0 kN, 0 kN/m
- **Moment M:** 0 kNm

**2. Support Structural - Geotechnical Checks**

#### Calculated Axial Support Capacities
- **Stress Check:** 2.421
- **Pall:** 413.1 kN, **Pult:** 578.3 kN

#### 3.1 Geotechnical Capacity
- **Pall:** 413.1 kN, **Pult:** 578.3 kN

#### 3.2 Structural Capacity
- **Pall:** 961.8 kN, **Pult:** 961.8 kN
8. Ground anchor and helical anchor capacity calculations

8.1 Ground Anchor Capacity Calculations

A ground anchor has two forms of capacity, a geotechnical and a structural resistance. The structural resistance of the tendons is defined by EC steel standards while the bonded zone has to be examined for its pull-out capacity (geotechnical check). The new software includes a number of ground anchor (tieback) sections. Hence, a ground anchor section can be reused over and over in many different support levels and in many different design sections (the same approach is also utilized for steel struts, rakers (inclined struts), and concrete slabs). The tieback capacities (ultimate and permissible) can be calculated using the following equations:

a) Ultimate geotechnical capacity used for the geotechnical yielding is:

\[ R_{\text{ULT.GEO}} = L_{\text{FIX}} \times \pi \times D_{\text{FIX}} \times q_{\text{ULT}} / \gamma_R \]

b) The design geotechnical capacity (for stress check ratios) is calculated as:

\[ R_{\text{DES.GEO}} = L_{\text{FIX}} \times \pi \times D_{\text{FIX}} \times q_{\text{ULT}} / (\gamma_R \times \gamma_{SU} \times \gamma_{FS \text{ Geo}}) \]

Where:

- \( q_{\text{ULT}} \) = Ultimate Skin friction (options available)
- \( L_{\text{FIX}} \) = Fixed body length
- \( D_{\text{FIX}} \) = Fixed body diameter (0.09m to 0.15m typically)
- \( \gamma_{R} \) = 1 to 2.0 user specified safety factor.
- \( \gamma_{SU} \) = 1 to 1.2 Resistance factor geotechnical capacity
- \( \gamma_{FS \text{ Geo}} \) = 1.0 to 2.0 user specified safety factor.
- FS Geo= 1.0 in M2 design approach methods.
- \( \gamma_{FS \text{ Geo}} \) = 1.0 to 2.0 user specified safety factor.

Note that \( \gamma_R \) and \( \gamma_{SU} \) are by default 1, but take Eurocode or DM08 specified values when a design approach is used.

c) The ultimate and design Structural capacity can be calculated as:

\[ P_{\text{ULT.STR}} = \phi_{\text{ULT.CODE}} \times (\text{Area of Tendons}) \times F_y \]

\[ \phi_{\text{ULT.CODE}} = \text{Material strength reduction factor typically 0.9} \]

\[ P_{\text{DES.STR}} = \phi_{\text{DES}} \times (\text{Area of Tendons}) \times F_y \]

\[ \phi_{\text{DES}} = \text{Material strength reduction factor 0.6 to 0.9} \]

The ultimate capacity is used to determine the structural yielding of the element while the permissible is used for the stress checks. \( \phi_{\text{ULT.CODE}} \) is always picked up from the structural code that is used. \( \phi_{\text{DES}} \) can be specified by the user or can be set automatically when the some code settings are specified. When Eurocodes are used \( \phi_{\text{ALL}} \) should be the same as \( \phi_{\text{ULT.CODE}} \).

Note that \( \phi = 1 / \gamma_m \)

In the default setting during a Paratie analysis, the new software models a tieback automatically as a yielding element (Wire with yielding properties) with its yielding force determined as the
minimum STR or GEO capacity. A ground anchor can also be modeled as a non-yielding wire element by selecting the appropriate option in the Advanced Tab of the Tieback dialog. However, it is felt that due to legal reasons it is better to include a tieback as yielding element by default.

Figure 10.1 shows the main tieback section dialog. The main parameters of interest are the steel material, the cross sectional area of the steel tendons, and the fixed body diameter (Dfix).

![Figure 10.1: Main tieback section dialog (Elastic-Wire command in Red)](image)

The geotechnical capacity represents the capacity of the soil to resist the tensile forces transferred by the steel strands to the grouted body. The new software subdivides the fixed body into a number of elements were soil resistance is computed. As previously mentioned, the geotechnical tieback capacity is evaluated for every stage. Within the current Paratie engine, it is currently possible to change the yield limit of an ELPL spring from stage to stage. Initially, in the Paratie mode the software uses the capacity at the stage of installation. The capacity is adjusted at each stage and the final support check is performed for the actual capacity for each stage. A number of options exist for defining the geotechnical capacity of a tieback:
a) Soil resistance is computed from frictional and cohesive components. For the frictional component, DeepXcav uses the average of the vertical and lateral at-rest stress times the tangent of the friction angle. For the cohesive component, adhesion factors can be applied. Furthermore, individual densification factors can be applied separately to the frictional and cohesive components to simulate the effect of pressure grouting. End bearing at the start of the grouted body is ignored. These calculations should be considered as a first order estimate. Hence, in this case the ultimate skin friction can be defined as:

\[ t_{\text{ULT}} = F_1 \times 0.5 \times (\sigma'_{V} + \sigma'_{H,K_0}) \times \tan(\phi) + F_2 \times \alpha \times (c' \text{ or } S_u) \]

In an undrained analysis the software will use \( S_u \) and \( \phi=0 \). For a drained analysis the program will use \( \phi \) and \( c' \).

Where:
- \( F_1 \) = Frictional densification factor (default 1)
- \( F_2 \) = Cohesional densification factor (default 1)
- \( \alpha \) = Adhesion factor (default =1), but program also offers a dynamic tri-linear approach for defining this parameter based on \( c' \) or \( S_u \). In this approach:
  - \( \alpha = \text{Value 1} = 0.8 \) if \( c' \) or \( S_u \leq \text{Climit1} \)
  - \( \alpha = \text{Value 2} = 0.5 \) if \( c' \) or \( S_u \geq \text{Climit2} \)
  - \( \alpha = \text{Linear interpolation for } c' \text{ or } S_u \text{ between Climit1 and Climit2.} \)

b) User-defined geotechnical capacity (and structural) defined from the advanced tieback tab.

![Figure 10.2: Advanced tieback dialog tab](image-url)
c) Ultimate specific bond resistance for tieback section.

\[ t_{\text{ULT}} = q_{\text{ULT}} \text{ in Geotechnical tab of tieback section} \]

![Figure 10.3: Geotechnical tieback dialog tab (Wire command in Red)](image)

\[ t_{\text{ULT}} = q_{\text{ULT}} \text{ from Soil type (Bond Tab)} \]

In this case, the skin friction can be determined from the Bustamante design charts (Fig. 10.5.1, 10.5.2) when pressuremeter test data are available.
Figure 10.4: Bond tab in Soil type dialog (> button offers ability to estimate from Pressuremeter tests).

e) When a Eurocode design approach is applied ultimate pull out resistance is calculated from bond values by applying the same safety factor (in combination with all other safety factors) as for the undrained shear strength $S_u$. However, in certain cases like the M2 the program does not apply the User Specified FS_geo in order to produce consistent capacity results. Thus, when Eurocode 7 or NTC settings are applied, the user specified FS_Geo is only used in cases where M1 factors are applied.

When the pullout resistance is calculated from soil cohesion and friction, then the skin friction is calculated directly from the adjusted friction angle and shear strength/cohesion values according the M safety factors.
Figure 10.5.1: Estimation of bond resistance for tiebacks from TA-95 according to Bustamante.

Figure 10.5.2: Estimation of bond resistance for tiebacks from Pressuremeter tests FHWA and French recommendations.
8.2 Helical anchor capacity calculations

A helical anchor/pile consists of one or more helix-shaped bearing plates attached to a central shaft, which is installed by rotating or “torqueing” into the ground. Each helix is attached near the tip, is generally circular in plan, and formed into a helix with a defined pitch. Helical anchors/piles derive their load-carrying capacity through both end bearing on the helix plates and skin friction on the shaft (Figure 10.6.1).

Figure 10.6.1: Typical helical anchor detail and geotechnical capacity behavior

According to IBC 2009, the allowable axial design load, \( P_{a} \), of helical piles shall be determined as follows:

\[
P_a = 0.5 \ P_u \quad \text{(IBC 2009 Equation 18-4)}
\]

Where \( P_u \) is the least value of:

1. Sum of the areas of the helical bearing plates times the ultimate bearing capacity of the soil or rock comprising the bearing stratum.
2. Ultimate capacity determined from well-documented correlations with installation torque.
3. Ultimate capacity determined from load tests.
4. Ultimate axial capacity of pile shaft.
5. Ultimate capacity of pile shaft couplings.
6. Sum of the ultimate axial capacity of helical bearing plates affixed to pile.
An explanation and summary of each of the six design criterions required per the IBC for helical pile design have been listed below to better explain the design process and intent of the code. Item 1 is in reference to the Individual Bearing Method. This method requires prior knowledge of the soil properties at the site via a soils report or boring logs. Please note that most soil reports only report the allowable bearing capacity of a soil or stratum. This allowable capacity normally has a safety factor of two or three applied.

**Item 1** is in reference to the Individual Bearing Method. This method requires prior knowledge of the soil properties at the site via a soils report or boring logs. Please note that most soil reports only report the allowable bearing capacity of a soil or stratum. This allowable capacity normally has a safety factor of two or three applied. Applying another factor of safety of two per the IBC would be extremely conservative.

Typical helical plate sizes are 8”, 10”, 12”, 14” and 16” in diameter. The maximum number of helical plates placed on a single pile is normally set at six (6). The central area of the shaft is typically omitted from the effective area of the helical plate when using the Individual Bearing Method.

The total capacity of the anchor can be calculated as:

\[ Q_{ult} = Q_{shaft} + \sum Q_h \]

where:

- \( Q_{ult} \) = Total Multi-helix Anchor/Pile Ultimate Capacity
- \( Q_h \) = Individual Helix Ultimate Capacity
  
  \[ Q_h = A_h (N_c + \gamma D N_q) \leq Q_{h,str} \]
  
  \[ Q_h = A_h (9 c + \gamma D N_q) \leq Q_{h,str} \]

where:

- \( A_h \) = Projected effective area of helix
- \( N_c \) = 9 for ratio of top helix depth to helix dia. > 5 (program assumes value =9)
- \( D \) = Depth of helical plate below ground line
- \( N_q \) = Bearing capacity factor for sand
- \( Q_{h,str} \) = Upper mechanical limit determined by helix strength
- \( Q_{shaft} \) = Geotechnical shaft resistance can be also included. Within the program, the shaft resistance is calculated only within from the starting point of the fixed length of the anchor to the first encountered helical plate. In most cases, the shaft resistance is conservatively ignored.

The software program replaces the above \( \gamma D \) term with the vertical effective stress at each helix.

According to a standard practice (AB Chance corporation presentation), \( N_q \) can be calculated as adapted from G. G. Meyerhof Factors for Driven Piles in his paper *Bearing Capacity and Settlement of Pile Foundations*, 1976

**Equation:**

\[ N_q = 0.5 (12\phi)^{\phi/54} \]

With a few exceptions, the shaft resistance can be calculated in a similar manner as the geotechnical capacity of ground anchors. When the capacity is calculated with side frictional methods, a distinction can be made between a grouted a non grouted shaft (i.e. a grouted shaft has friction of cement grout with soil while a non grouted shaft between steel and soil).
Item 2 is in reference to the Torque Correlation Method. The Torque Correlation Method is an empirical method that distinguishes the relationship between helical pile capacity and installation torque and has been widely used since the 1960's. The process of a helical plate shearing through the soil or weathered bedrock in a circular motion is equivalent to a plate penetrometer test. The method gained notoriety based on the study performed by Hoyt and Clemence (1989). Their study analyzed 91 helical pile load tests at 24 different sites within various soil types ranging from sand, silt and clay soils. They demonstrated the direct correlation of the installation torque of a helical pile to its ultimate capacity in compression or tension. The common denominator discovered from the study was a parameter referred to as the torque correlation factor, $K_t$.

The equation is:

$$P_u = K_t \cdot T$$

Where:

- $P_u$ is the ultimate capacity of the helical pile or anchor [lb (kN)].
- $K_t$ is the empirical torque factor of the central shaft of the pile [ft-1 (m-1)].
- $T$ is the final installation torque [ft-lb (m-kN)].

It’s important to point out that the tests analyzed by Hoyt and Clemence (1989) were in tension. It was shown in sub-sequential studies that the tension capacity of helical piles was 16 to 33 percent less than the measured compression capacity. The difference is attributed to the fact that the lead helical plate is bearing on relatively undisturbed soil in compression applications. In tension applications, the leading and trailing helical plates are bearing on soil affected by the installation of the helical plates. It has become common practice to use the same torque correlation factor for a helical pile of the same size for tension and compression and ignore the slight increase in compression capacity. This creates a more conservative compression capacity for helical piles when compared to the Individual Bearing Method. Also unlike the Individual Bearing Method, the number of helical plates on a pile is completely independent of the pile capacity based on the Torque Correlation Method.

$$Q_{ult} = K_t \cdot T$$

Where:

- $Q_{ult}$ = Ultimate Capacity [lb (kN)]
- $K_t$ = Empirical Torque Factor [ft-1 (m-1)]
  - “Default” Value = 10 (33) for Type “SS”
  - “Default” Value = 8 (26) for 2-7/8” Pipe Shaft
  - “Default” Value = 7 (23) for 3-1/2” Pipe Shaft
  - “Default” Value = 6-7 (20-23) for 4-1/2” Pipe Shaft
- $T$ = Installation Torque [ft-lb (kN-m)]

Figure 10.6.2: Typical recommended torque factors

When helical anchors are used in a non-linear analysis, the elastic behavior of the helical anchor must be first estimated. In most cases, the ultimate geotechnical pullout resistance of a helical anchor is controlled by the bearing at each helix. As a result, the ultimate resistance of a helical anchor is realized at much greater strains than shaft resistance, and may require as much displacement as 10% of the helix diameter. A reasonable, albeit rough assumption, is to use an additional displacement 4% of the largest helix diameter at 65% of the ultimate anchor capacity (Figure 10.6.3). This additional strain/displacement is added to the elastic anchor elongation and must be verified by load tests.
Figure 10.6.3: Elastic behavior of helical anchors – advanced tab
9. Geotechnical Safety Factors
9.1 Wall Embedment Stability (toe stability)

9.1.1 Introduction
Support walls must be embedded sufficiently to prevent toe stability failure. DeepXcav uses classical methods in determining the toe embedment depth for a safety factor of 1.0 and the available safety factor. The following paragraphs describe the available methods for calculating toe stability for cantilever, single support, and multiple level braced walls. The following safety factor definitions are available:

1) Passive Resistance Safety Factor (Conventional Analysis):
\[ F_{S_{\text{passive}}} = \frac{\text{Resisting horizontal forces}}{\text{Driving horizontal forces}} \]  
(Eq. 9.1)

2) Rotational Safety Factor (Conventional Analysis):
\[ F_{S_{\text{rotation}}} = \frac{\text{Resisting moments about a point}}{\text{Driving moments about the same point}} \]  
(Eq. 9.2)

3) Length based (Conventional Analysis):
\[ F_{S_{\text{embed}}} = \frac{\text{Available wall embedment depth}}{\text{Max. Required embedment depth for } F_S = 1 \text{ from Equations 1 & 2 above}} \]  
(Eq. 9.3)

4) Mobilized passive resistance (PARATIE)
\[ F_{S_{\text{pass. mob}}} = \frac{\text{Available soil passive resistance beneath subgrade}}{\text{Mobilized passive soil reaction beneath subgrade}} \]  
(Eq. 9.4)

The mobilized passive resistance is currently calculated with conventional analysis methods (that can include the effects of non-linear ground surface).

5) Zcut method (PARATIE)
The internal Paratie engine gradually reduces the wall length until the wall collapses.

When both the conventional and PARATIE methods are used, the software first analyzes the model with the conventional approach and stores the calculated conventional safety factors (Equations 9.1, 9.2, 9.3). Subsequently, once the PARATIE analysis is successfully performed, the software evaluates the toe safety based on Eq. 9.4. In this way the user has the benefit of evaluating all conventional safety factors while performing a PARATIE analysis at the same time.

Note that in equations 9.1 and 9.2 the water contribution is included as a net driving (i.e. unfavorable) component in the denominator.
9.1.2 Cantilever Walls (conventional analysis)

For cantilever walls, one may use the "free" earth method or the "fixed" earth method as illustrated in the figure below. The safety factor is defined as the available embedment depth over the embedment depth for a safety factor of 1.0 using the given set of pressure assumptions. The fixed earth method uses an iterative search solution to find the minimum depth were stability is satisfied. Note that in some instances, the fixed earth method may fail to converge when very stiff soils with high cohesion values are used. In such a case you might have to use the free earth method to obtain results.

In the free earth method the software finds the depth of wall where the overturning moment is equal to the resisting moment and this depth is then set as the $FS_{rotation} = 1.0$ depth. In this method, the horizontal forces are not balanced and for stable walls the passive resistance is greater than the driving resistance (wall shear is not automatically balanced). The software then calculates a safety factor for resisting to driving moments, and also an embedment safety factor just based on depth ($FS = \text{Depth from subgrade to wall bottom} / \text{depth from subgrade to elevation of zero moment}$). The overall reported safety factor is the minimum of the two values.

![Figure 11.1: Fixed Earth Method for a cantilever wall](image-url)
9.1.3 Walls supported by a single bracing level in conventional analyses.

For walls supported by a single level of bracing the software uses the free earth method as illustrated below. For multiple support walls, moments are taken in the same fashion about the lowest support level. However, only pressures below the lowest support level are considered. The safety factor is defined as:

\[
FS = \frac{\text{Resisting Moment about anchor point}}{\text{Overturning Moment about anchor point}}
\]

![Free Earth Method for a wall supported by a single level of anchors](image)

9.1.4 Walls supported by a multiple bracing levels (conventional analysis)

For walls supported by multiple braces the software can calculate both the horizontal force safety factor and the rotational safety factor (Eq. 9.1 and 9.2):

\[
FS_{\text{rotation}} = \frac{\text{Resisting Moment about lowest brace}}{\text{Overturning Moment about lowest brace}}
\]

Only the force/pressures below the lowest brace are accounted in the above equation.

For the passive safety factor, the software offers two possibilities depending on how the support forces are determined:

a) If the wall is analyzed with a beam analysis “Blum’s method”, then the software finds the virtual net horizontal reaction that acts on the virtual fixity point beneath the subgrade. The passive safety factor is then taken as:

\[
FS_{\text{pas}} = \frac{\text{Available Resistance beneath virtual fixity point}}{\text{Hor. reaction at virtual point + Driving pressures beneath virtual fixity point}}
\]
a) If the support reactions are determined with the tributary area method, then the software determines the apparent horizontal reaction that acts at the subgrade level (i.e. at the excavation elevation). The passive safety factor is then taken as:

\[ F_{p\text{as}} = \frac{\text{Available Resistance beneath subgrade}}{\text{Apparent reaction at subgrade} + \text{driving pressures beneath subgrade}} \]

Driving pressures = Soil Pressure at driving side + Net Water Pressures

9.2 Clough Predictions & Basal Stability Index

Clough (Clough et. al., 1989) developed a method for predicting wall deflections based on a normalized system using a stiffness factor and on a factor of safety for basal stability. Clough defines the system stiffness factor as the product of the modulus of elasticity (E) times the inertia (I) of the wall divided by the unit weight of water (γw) times the average brace spacing. This approach of predicting wall deflections has some obvious limitations when applied to stiff walls because: a) the typical spacing between vertical supports varies little from project to project (average 9ft to 12 ft) with 7ft minimum and 17ft maximum, b) the wall thickness typically varies from 2’ to 3’, c) the effects of prestressing braces are totally ignored, and d) the effect of soil conditions is partially accounted in the basal stability factor that is not directly applicable for the majority of the walls that were keyed into a stiff stratum.

It is clear that the big limitation of the system stiffness approach is the generic assumption that wall deflections are primarily related to deformations occurring between support levels. In individual projects, there may be several length scales affecting the wall deflections depending on the toe fixity of the wall, the depth to bedrock, the wall embedment below the base of the excavation, the width of the excavation, the size of berms, and the initial unsupported excavation depth. Furthermore, the proposed method of Clough et. al. [1989] takes not account of the stiffness profile in the retained soil.

Recently, a much more detailed finite element study has carried out by Jen [1998]. These finite analyses consisted of three main groups of parametric analyses to quantify the effects of excavation geometry, soil profile, and support system. The parameters she studied included: I) Geometry (wall length, excavation width, depth to bedrock), II) Soil Profile (overconsolidation ratio of the clay, cohesionless layer over a clay stratum, presence of clay crust over low OCR clay stratum), and III) Support System (stiffness of support wall, and bracing components).

Jen [1998] found that walls basically undergo three phases of deformation: i) Unsupported cantilever deflections; ii) bulging (subgrade bending); and iii) toe kickout. She concluded that the actual deformation phase was determined by the wall embedment depth. Wall stiffness was more effective in reducing deformations for soft soils but had a smaller effect in stiffer soils. She also found that the depth to bedrock had a significant impact on the surface settlement at a distance from the excavation.
Figure 11.3: Predicted wall displacements versus system stiffness according to Clough et. al.

Table 4.1: Limit equilibrium calculations of base stability (after Terzaghi, 1943).

<table>
<thead>
<tr>
<th>Condition</th>
<th>Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D &lt; \sqrt{\frac{2}{2} B}$</td>
<td>$N_c \frac{s_{sub}}{\gamma - \frac{s_{sub}}{D}}$</td>
</tr>
<tr>
<td>$D &gt; \sqrt{\frac{2}{2} B}$</td>
<td>$N_c \frac{s_{uu}}{2 \left(\frac{s_{uu}}{\gamma} - \frac{1}{\sqrt{2 B}}\right)}$</td>
</tr>
</tbody>
</table>

$s_{sub} = \text{undrained strength of basal clay}$  
$s_{uu} = \text{undrained strength of clay above the excavation grade}$  
$B = \text{Breadth of excavation}$  
$D = \text{Depth from excavation grade to firm stratum}$  
$N_c = 5.4$

Since multiple soil types can be included, DeepXcav averages the undrained shear strength of the soil below the subgrade within one excavation depth below subgrade or until a rock layer is encountered. Note that the frictional component of a soil is included by adding to $S_u$ the sum of the vertical effective stress times the tangent of the friction angle on the left and right wall side.
9.3 Ground surface settlement estimation

Boone and Westland reported an interesting approach to estimating ground settlements. This approach associates ground settlements to the basal stability index, a modified system stiffness value, and individual wall displacement components as seen in the following figure. Wall displacements and surface settlements are divided in two major categories:

a) Cantilever wall – generating the sprandel settlement volume trough $A_{vs}$

b) Bending wall movements – generating the concave settlement trough $A_{vc}$

The combination of sprandel and concave settlement troughs results in the combined total settlement profile. Both these areas $A_{vs}$ and $A_{vc}$ are taken as a certain percentage of the corresponding wall movements.

When a non-linear solution is performed (beam on elastic foundations), DeepXcav offers the ability to estimate surface settlements directly from computed wall displacements. In addition, DeepXcav will add a component for toe translation to the concave settlement. This additional volume is estimated as a triangle by extrapolating a line from the maximum displacement above the wall toe to the displacement at the wall base. The following table provides detailed information and recommendations about using this method.

If settlements are used, it is strongly recommended to enable the modifications from the surface settlements tab in the main form (otherwise surface settlements may be greatly overestimated).

![Diagram of lateral and vertical displacement parameters](image)

**Figure 11.4:** Definitions of lateral and vertical displacement parameters: concave on left, sprandel on right (after Boone 2003).
Table 4.2: Summary of displacement estimation equations based on curve fitting of non-linear numerical modeling results.

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Condition</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>maximum unfactored lateral displacement, $\delta_{\text{max}}$</td>
<td>support installation and removal supports removed</td>
<td>$\delta_{\text{max}} = (8.5S_5 + 0.4)FS^{-1.7}$</td>
</tr>
<tr>
<td>construction stage, $\alpha_c$</td>
<td>tiebacks remaining stressed</td>
<td>$\alpha_{CS} = 1$</td>
</tr>
<tr>
<td>preloading, $\alpha_{PL}$</td>
<td>percent of preload maintained</td>
<td>$\alpha_{PL} = \frac{F \cdot L}{(0.45 + S_5)}$</td>
</tr>
<tr>
<td>excavation width, $\alpha_B$</td>
<td>$\alpha_B = 0.75 + H/(4B)$</td>
<td></td>
</tr>
<tr>
<td>strut stiffness, $\alpha_S$</td>
<td>$\alpha_S = 0.3\left(e^{S_5/1000} + e^{S_5/200}\right) + 0.7$</td>
<td></td>
</tr>
<tr>
<td>soil modulus, $\alpha_M$</td>
<td>$\alpha_M = 6.67E_u^{-1/5}$</td>
<td></td>
</tr>
<tr>
<td>max. lateral displacement, $\delta_{\text{max}}$</td>
<td>$\delta_{\text{max}} = \delta_{\text{limax}} \alpha_M \alpha_S \alpha_{CS}$</td>
<td></td>
</tr>
<tr>
<td>ground surface displacements</td>
<td></td>
<td></td>
</tr>
<tr>
<td>maximum lateral displacement at surface, $\delta_{\text{surface}}$</td>
<td>Supports remain in place</td>
<td>$\delta_{\text{surface}} = \frac{(E_u/p_s)}{500 + (E_u/p_s)S_5^{0.7}}$</td>
</tr>
<tr>
<td></td>
<td>Supports removed</td>
<td>$\delta_{\text{surface}} = \frac{(E_u/p_s)}{700}$</td>
</tr>
<tr>
<td>lateral displacement areas</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Area of lateral spandrel displacement, $A_{\text{ht}}$</td>
<td>$\alpha_{PL} = \delta_{\text{surface}}(H + H_p)/2$</td>
<td></td>
</tr>
<tr>
<td>Ratio of spandrel displacement to total displacement area, $A_{\text{ht}}/A_w$</td>
<td>End of excavation stage $A_{\text{ht}} = \frac{(E_u/p_s)}{1,600 + (E_u/p_s)S_5^{0.7}}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>After support removal</td>
<td>$A_{\text{ht}} = \frac{(E_u/p_s)}{300 + (E_u/p_s)}$</td>
</tr>
<tr>
<td>Area of convex displacement, $A_{hc}$</td>
<td>$A_{hc} = 1 - A_{ht}/A_{ht}$</td>
<td></td>
</tr>
<tr>
<td>ratios of vertical and lateral displacement areas</td>
<td>Cantilever walls $A_{vh}/A_{hc} = A_{vh}/A_{hc} = A_{vh}/A_{ht} = 1$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Supports remain in place</td>
<td>$A_{vh}/A_{ht} = A_{vh}/A_{hc} = A_{vh}/A_{ht} = 0.85$</td>
</tr>
<tr>
<td></td>
<td>Supports removed</td>
<td>$A_{vh}/A_{ht} = A_{vh}/A_{hc} = A_{vh}/A_{ht} = 1.1$ (no dilation)</td>
</tr>
<tr>
<td>spandrel portion of settlement trough</td>
<td>Maximum settlement, $\delta_{\text{vert}}$</td>
<td>$\delta_{\text{vert}} = 3A_{vh}/D_s; D_s \approx 1.2H$ to $1.5H$</td>
</tr>
<tr>
<td></td>
<td>Settlement at any point, $\delta_{sv}$</td>
<td>$\delta_{sv} = \delta_{\text{vert}} \frac{3[(D_s - d)/D_s]^2}{D_s \approx 1.2H$ to $1.5H}$</td>
</tr>
<tr>
<td>concave settlement portion of settlement trough</td>
<td>Maximum settlement, $\delta_{\text{vert}}$</td>
<td>$\delta_{\text{vert}} = \frac{A_{vh}}{\left[1 - \Phi(0, d_{\text{max}}/\sigma)\right]^{2/1}}$</td>
</tr>
<tr>
<td></td>
<td>Settlement at any point, $\delta_{sv}$</td>
<td>$\delta_{sv} = e\left(\frac{\delta_{\text{vert}}^2 \cdot i^2}{2\pi}</td>
</tr>
</tbody>
</table><p>ight)^{1/2}$ |
| D_s = twice the distance from the wall top to the position of the load resultant | $\Phi = \frac{\text{area of standard normal distribution function, with random variable } = 0 \text{ (wall position)}}, \text{mean } = d_{\text{max}}, \text{and standard deviation } = i$ | |
| complete settlement profile            |                                              |                                                                          |
| Total settlement at any point          | $\delta_t = \delta_{\text{ex}} + \delta_{sv}$ |                                                                           |</p>
10. **Handling unbalanced water pressures in Paratie**
   As discussed before, automatic handling of water pressures in Paratie when Eurocode is used can be a very tricky subject. The following paragraphs list some available options for reasonably modeling water in Paratie simulations:

**Automatic Generation:**

**Alternative A:** Inclusion of a model factor on wall and support results
In this approach, wall bending/shear and support reactions are multiplied by a model factor to account for the additional net water forces. The software uses the safety factor on unfavorable earth resistance as the “model” factor which in all Eurocode 7 and DM08 approaches is equal to the water unfavorable factor. This philosophical contrast stems from the fact that, in this case, we would have gone through all the effort of readjusting soil properties, actions, etc. only to come at the end of our analysis and essentially use an allowable safety factor. This approach is used.

**Manual Generation (Must be specified externally by user)**

**Alternative B:** Readjustment of water levels (Manual)
From a theoretical standpoint, the most appropriate solution if the groundwater table is beneath the surface would be to simply raise the ground water elevation (but one should be careful not to elevate the ground water elevation above). However, in this approach an automatic groundwater elevation change might not make much sense all together in many cases. For example, in a 15 m excavation beneath the groundwater one would multiply the retained water height by 1.35 resulting in:

$$\text{New water height } H = 1.35 \times 15 \text{ m} = 20.25 \text{m}$$

It is obvious, that for a typical excavation elevating the water table by 5.25m would create unrealistic modeling of the soil stress-state history. For example, if one were to do this in Milan (where there is a canal and the water table is 3 m below the street) then the simulated assumption would be that the water has raised 2.25 m above the street level, essentially flooding the entrance of many buildings.

Thus, a reasonable automatic generation procedure for readjusting water levels can be very challenging and is not implemented within the software.

**Alternative C:** Addition of net water pressures as external pressures
In this approach, the additional net factored water pressures (above the normal water pressures calculated from groundwater flow analysis) are added automatically as an external surcharge. With this alternative additional water pressures should be accounted on every stage.
11. Wall Types - Stiffness and Capacity Calculations

The new software offers a number of different wall options for calculating wall stiffness and wall capacity. The different wall types control also the manner by which soil and water pressures are accounted beneath the excavation subgrade (i.e. bottom of excavation). The following tables summarizes the basic stiffness calculations used for each different wall type.

**Table 5.1: Wall Type and Automatic Stiffness Calculations – Soldier Pile & Sheet Pile Walls**

<table>
<thead>
<tr>
<th>Wall Type</th>
<th>Subtype</th>
<th>Wall Stiffness (Each Element)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soldier Pile Wall and Lagging</td>
<td>With Steel H beam</td>
<td>The strong axis moment of inertia of the steel element I_{xx}</td>
</tr>
<tr>
<td></td>
<td>With Steel Pipe Only</td>
<td>For thread reduction please see table notes.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>[ I_{xx} = \frac{\pi}{64} \left( D^4 - (D - 2 \cdot t_p)^4 \right) ]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>[ S_{xx} = \frac{I_{xx}}{D - 2 \cdot t_p} \cdot \text{Thread Reduction Percentage} ]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>[ \delta_{xx} = \frac{\pi}{8} \left[ D^4 - (D - 2 \cdot t_p)^4 \right] \cdot \text{Thread Reduction} ]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>[ k_{NY} = \frac{I_{xx} \cdot \text{Thread Reduction}}{A_{p}} ]</td>
</tr>
<tr>
<td></td>
<td>With Steel Pipe filled with Concrete</td>
<td>All properties same as above except: [ A_{st} = A_{pipe} + \frac{\pi}{4} \left( D - 2 \cdot t_p \right)^2 \cdot \frac{E_{concrete}}{E_{steel}} ]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>[ k_{NY} = \frac{I_{xx} \cdot \text{Thread Reduction}}{A_{st}} ]</td>
</tr>
<tr>
<td></td>
<td>Reinforced Concrete</td>
<td>[ I_{xx} = \frac{\pi}{64} D^4 ] [ A_{st} = \frac{\pi}{4} D^2 ] (Only on reinforced piles, can also be defined by user)</td>
</tr>
<tr>
<td></td>
<td>Soil mix (only concrete)</td>
<td>[ I_{xx} = \frac{\pi}{64} D^4 ] [ S_{xx} = \frac{\pi}{16} D^3 ] [ A_{st} = \frac{\pi}{4} D^2 ] (Only on reinforced piles, can also be defined by user)</td>
</tr>
<tr>
<td></td>
<td>Steel sheet piles</td>
<td>The moment of inertia of the sheet pile I_{xx}/unit length.</td>
</tr>
</tbody>
</table>

**Notes:** Often with steel pipes smaller elements have to be sequentially connected in order to form the full wall length. In this case, the joints between different segments are typically threaded.
These threads must be fully welded to provide the full pipe strength. Otherwise, the pipe strength (not stiffness) has to be empirically reduced by the thread reduction factor.

Table 5.2: Wall Type and Automatic Stiffness Calculations, Secant & Tangent Pile Walls

<table>
<thead>
<tr>
<th>Wall Type</th>
<th>Subtype</th>
<th>Wall Stiffness (Each Element)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Secant Pile Walls</strong></td>
<td>With Steel H beam</td>
<td>The strong axis moment of inertia of the steel element $I_{xx}$ is combined with $E_{	ext{effect}}$ typically taken as 25% of the equivalent rectangular concrete block stiffness.</td>
</tr>
<tr>
<td></td>
<td>D= Beam Depth</td>
<td>$I_{xx} = I_{xx} + (E_{	ext{effect}}/E_{	ext{steel}}) \left( \frac{D^3}{12} - \frac{D^3}{12} \right)$</td>
</tr>
<tr>
<td></td>
<td>With Steel Pipe filled with Concrete</td>
<td>$I_{xx} = \frac{\pi}{4} \left[ D^4 - (D - 2 \cdot t_p)^4 \right]$</td>
</tr>
<tr>
<td></td>
<td>D= Pipe Diameter</td>
<td>$S_{xx} = \frac{\pi}{4} D^4 \cdot \text{Thread Reduction Percentage}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$E_{xx} = \frac{\pi}{3} \left[ D^3 - (D - 2 \cdot t_p)^3 \right] \cdot \text{Thread Reduction}$</td>
</tr>
<tr>
<td></td>
<td>With Steel Pipe filled with Concrete</td>
<td>$I_{xx} = \frac{\pi}{4} \left[ D^4 - (D - 2 \cdot t_p)^4 \right] \cdot \text{Thread Reduction}$</td>
</tr>
<tr>
<td></td>
<td>D= Pipe Diameter</td>
<td>$A_{\text{pipe}} = \frac{\pi}{4} \left( D^2 - (D - 2 \cdot t_p)^2 \right) \cdot \text{Thread Reduction}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\alpha = \frac{I_{xx} \cdot \text{Thread Reduction}}{A_{\text{pipe}}}$</td>
</tr>
<tr>
<td></td>
<td>With Steel Pipe filled with Concrete</td>
<td>All properties same as above except:</td>
</tr>
<tr>
<td></td>
<td>D= Pipe Diameter</td>
<td>$A_{\text{pipe}} = A_{\text{pipe}} + \frac{\pi}{4} \left[ D^2 - (D - 2 \cdot t_p)^2 \right] \cdot \frac{E_{\text{concrete}}}{E_{\text{steel}}}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\alpha = \frac{I_{xx} \cdot \text{Thread Reduction}}{A_{\text{pipe}}}$</td>
</tr>
<tr>
<td></td>
<td>With Steel Pipe filled with Concrete</td>
<td>(Only on reinforced piles, can also be defined by user)</td>
</tr>
<tr>
<td></td>
<td>D= Reinforced Concrete</td>
<td>$I_{xx} = \frac{\pi}{6} D^4 \cdot A_{\text{circular}} = \frac{\pi}{4} D^4$</td>
</tr>
<tr>
<td></td>
<td>D=W</td>
<td>(Only on reinforced piles, can also be defined by user)</td>
</tr>
<tr>
<td></td>
<td>With Steel Pipe filled with Concrete</td>
<td>$I_{xx} = 2 \pi r^2 \cdot \frac{r^2}{6} D^4 - 2 \pi r^2 t_p^2$</td>
</tr>
<tr>
<td></td>
<td>D=W</td>
<td>$S_{xx} = \frac{I_{xx}}{6 \pi r^2}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$A_{\text{circular}} = \frac{\pi}{2} \cdot D^2$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(Only on reinforced piles, can also be defined by user)</td>
</tr>
<tr>
<td><strong>Tangent Pile Walls</strong></td>
<td>Same as secant pile walls</td>
<td>Same as soldier pile walls but Effective width is limited to the flange size.</td>
</tr>
</tbody>
</table>
From version 8.0.9.33, for soldier pile and tangent pile walls, the software program offers the ability to model walls with piles that are offset in respect to the wall axis (as the following figure shows). The program offers the ability to include a stiffness increase that simulates the pile offset. In this case, the wall stiffness increase is accounted by:

$$\text{Moment of inertia increase/pile} = \text{Effectiveness Factor (\%)} \times \text{Area} \times (\text{Offset}/2)^2$$

The increase in moment of inertia is only accounted for the equivalent wall thickness (or wall stiffness) and not for the moment of inertia of each individual pile.

Where:
- **Area** = Steel pile area for walls with steel members, or concrete area for concrete piles
- **Effectiveness factor** = The effectiveness factor (0 to 100%) for the stiffness increase due to the wall offset. It is important to note, that a rigid pile cap connection must be utilized if this factor is included. In essence, the wall stiffness can be increased if there is sufficient friction between adjacent piles. A factor of 100% would imply that the wall is fully braced all along the whole length and especially at the top and bottom end points (a case that is not realistic in most conditions).
Table 5.3: Wall Type and Automatic Stiffness Calculations, Diaphragm and SPTC Walls

<table>
<thead>
<tr>
<th>Wall Type</th>
<th>Subtype</th>
<th>Wall Stiffness (Each Element)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Always Reinforced Concrete</td>
<td></td>
<td>Only Concrete stiffness is considered by default. $I_{xx} = \frac{w^4}{12}$, $S$, $w^4$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>However, note that while this value is automatically calculated the program uses the specified $I_{xx}$ within the Wall Sections dialog.</td>
</tr>
<tr>
<td>SPTC Walls (Soldier Pile and Tremied Concrete)</td>
<td>Only pile stiffness is considered</td>
<td>Only pile stiffness is considered. Usually concrete is ignored as it is very weak, but the software program allows the user to include a percentage of the rectangular block stiffness thus: $I_{eq} = I_{xx} + (\text{Effective%}) \left{ \frac{S \cdot W^2}{12} \cdot \frac{E_{\text{concrete}}}{E_{\text{steel}}} \right}$</td>
</tr>
</tbody>
</table>

**Equivalent Thickness Calculations for Paratie:**

The non-linear module works with an equivalent stiffness approach where the wall thickness and the material elasticity are defined. In this approach the wall is treated within the engine as a continuous slab with an equivalent thickness $T_{eq}$.

The equivalent thickness is calculated by using the $I_{xx}$ defined in Tables 5.1 through 5.3 and dividing it by the wall spacing $S$. Thus:

$$T_{eq} = \sqrt{\frac{12 \cdot I_{xx}}{S}} \cdot \frac{E_{\text{WALL}}}{E_{\text{STANDARD}}}$$

Typically in most cases $\frac{E_{\text{WALL}}}{E_{\text{STANDARD}}} = 1$.

$I_{xx}$ and $S$ are always converted in consistent units. However, since a wall may be composite, the wall thickness may be adjusted internally to account for the standard material that is being used for the wall. The equivalent thickness and stiffness calculations are always reported in the paratie input file.
12. **Seismic Pressure Options**

Earth retaining structures such as braced excavations and anchored bulkheads experience additional forces during seismic events. The true wall behavior is very complex and can rarely be truly simulated for most earth retaining structures. Instead, engineers have long used widely acceptable simplified models and methods that allow seismic effects to be added as external pressure diagrams. These additional seismic pressures can be essentially divided in three parts:

- a) The additional force due to the soil skeleton
- b) The additional hydrodynamic forces.
- c) Inertia effects on the retaining structure

Unyielding walls (i.e. rigid walls that do not move) experience greater forces compared to yielding walls. Hence, permissible wall displacement influences the magnitude of the external forces that a wall might experience during an earthquake. In an elastoplastic analysis (i.e. Paratie engine) an automatic simplified procedure is available that gradually reduces pressures from the theoretical rigid wall limit as the wall displaces. The details of this method are outside the scope of this text and are covered in the Paratie engine theoretical manual.

Further, water in highly permeable soils may be free to move independently from the soil skeleton, thus adding hydrodynamic pressures on a wall. Water above the ground surface will also add hydrodynamic effects. The following sections provide full details for all available methods within the new software. In general the following procedure has to be followed:

- a) Determination of site factors, base acceleration.
- b) Selection of the basic wall behavior (flexible or rigid). For rigid wall behavior the wood method is selected and then we proceed to step e).
- c) For flexible walls, determination of the structure response (R factor).
- d) Selection of method for seismic pressures due to soil.
- e) Option to select seismic pressures to be included for full wall height or just excavation height.
- f) Selection of method for calculating hydrodynamic effects (if any).
- g) Option to include inertia effects due to the wall mass.
12.1 Selection of base acceleration and site effects

The horizontal design acceleration in most building codes is defined by determining the base acceleration (at the bedrock base) and including adjustment factors that account for soil conditions, site topography, structure behavior, and finally structure importance. The horizontal design acceleration is generally defined as:

\[ A_{\text{DES}} = \frac{A_{\text{BASE}} \cdot S_s \cdot S_T \cdot I}{R} \]

Where:
- \( A_{\text{DES}} \) = Maximum horizontal design acceleration
- \( A_{\text{BASE}} \) = Base acceleration determined for a required return period
- \( S_s \) = Soil type amplification factor (typical values 1 to 2)
- \( S_T \) = Topographic site amplification factor (valley, ridge, flat ground) typical values range from 1 to 1.4
- \( I \) = Importance factor (depends on structure usage). 1 for typical structures, see individual building codes for actual values.

Some building codes (i.e. Italian) provide detailed methods for determining various factors automatically (see 12.5).
12.2 Determination of retaining structure response factor R

For flexible walls (i.e. yielding walls) the software offers a number of options for calculating the structure response factor R:

a) When the user site response option is selected, the permissible displacement and responded factor R can be defined according to Eurocode 8 and Simonella (2004).

![Figure 12.2: Seismic response factor R according to Simonella 2004](image)

b) Richards-Elms displacement control based method: Richards and Elms (1979) outlined a procedure for determining the design acceleration based on the acceptable permanent wall displacement. This method is essentially summarized in the following equation:

\[ \delta_{\text{PERM}} = 0.087 \cdot \frac{v_{\text{MAX}}^2}{\alpha_{\text{MAX}}} \]

Or

\[ \alpha_X = \left(0.087 \cdot \frac{v_{\text{MAX}}^2}{\delta_{\text{PERM}}} \right)^{4/4} \]

Where:
- \( \delta_{\text{PERM}} \) = Permanent horizontal design displacement
- \( v_{\text{MAX}} \) = Maximum base velocity
- \( \alpha_{\text{MAX}} \) = Maximum surface ground acceleration (units of length/sec^2) that includes all site and importance effects.
- \( \alpha_X \) = Horizontal design acceleration including (units of length/sec^2)

Kramer (1996) suggests the following preliminary values for \( v_{\text{MAX}} / \alpha_{\text{MAX}} \):

<table>
<thead>
<tr>
<th>Stratigraphy</th>
<th>( v_{\text{MAX}} / \alpha_{\text{MAX}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rocks or rigid profiles</td>
<td>0.05 sec</td>
</tr>
<tr>
<td>Deformable layers over a rigid base</td>
<td>0.15 sec</td>
</tr>
</tbody>
</table>

c) Liao-Whitman displacement control method:
Liao and Whitman provided an alternative way to calculate the design acceleration from the permissible displacement.

\[
\alpha_x = \frac{-\alpha_{\text{max}} \cdot \ln\left(\frac{2 \cdot \text{PERM} \cdot \alpha_{\text{max}}}{3 \cdot \alpha_{\text{MAX}}^2}\right)}{9.4}
\]

**d) Italian building code (NTC 2008) automatic procedure:**

The Italian 2008 building code offers a straightforward manner for estimating the horizontal design seismic acceleration.

\[
a_h = k_h g = \alpha \cdot \beta \cdot a_{\text{MAX}} \quad \text{(NTC 2008 eq. 7.11.9)}
\]

\[
a_{\text{MAX}} = S_t \cdot a_g = S_t \cdot S_v \cdot a_g \quad \text{(NTC 2008 eq. 7.11.10)}
\]

The software program then determines the design acceleration with:

\[
\alpha_x = \frac{I \cdot \alpha_h}{g}
\]

The \( \alpha \) and \( \beta \) parameters are determined with the aid of the following design charts where:

- \( H \) = Excavation height (automatically determined during analysis)
- \( u_s \) = design permanent wall displacement (defined by user)

![Figure 12.3: Site response factor \( \alpha \) according to Italian building code NTC 2008](image)

Figure 12.3: Site response factor \( \alpha \) according to Italian building code NTC 2008
12.3 Seismic Thrust Options

12.3.1 Semirigid pressure method

In the semirigid approach the seismic pressure is calculated as the product of the total vertical stress at the bottom of the wall (or excavation subgrade depending on user selection) times a factor B. The seismic thrust is then included as an external rectangular pressure diagram.

\[ \Delta P = B \cdot (\sigma_{\text{TOTAL}} - u_{\text{TOP}}) \]

Where:
- \( B \) = Multiplication factor (typically 0.75, i.e. Hellenic Seismic Code)
- \( \sigma_{\text{VTOTAL}} \) = Total vertical stress at wall bottom or excavation subgrade
- \( u_{\text{TOP}} \) = Water pressure, if any, at the ground surface (if water is above ground surface)
12.3.2 Mononobe-Okabe

Okabe (1926) and Mononobe and Matsuo (1929) developed the basis of a pseudostatic analysis of seismic earth pressures that has become popularly known at the Mononobe-Okabe (M-O) method. The M-O method is a direct extension of the static coulomb theory that accounts for acceleration where seismic accelerations are applied to a Coulomb active (or passive) wedge. The software program always includes seismic pressures calculated with the M-O method as external loads.

\[ K_{\text{M-O}} = \frac{\cos^2(\alpha - \theta - \beta)}{\cos(\beta) \cos^2(\alpha) \cos(\theta + \alpha + \beta) \left[ 1 + \frac{\sin(\alpha + \phi) \sin(\alpha - \alpha - \beta)}{\cos(\theta + \alpha + \beta) \cos(\alpha - \beta)} \right]} \]

Where \( \alpha = \) Slope angle (positive upwards)
\( \beta = \) Seismic effects = Dry soil
\( \tan^{-1}\left(\frac{\text{dry}}{1-\text{dry}}\right) \) (EC. Eq. E.13)

Impervious soil = \( \tan^{-1}\left(\frac{\text{Pervious}}{1-\text{Pervious}}\right) \) (EC. Eq. E.16)

\( \alpha_x = \) horizontal acceleration (relative to g)
\( \alpha_y = \) vertical acceleration, +upwards (relative to g)
\( \theta = \) Wall angle from vertical (0 radians wall face is vertical)

For a vertical wall the net seismic thrust is then calculated by:

\[ F_{\text{EQ}} = \int (1 - \alpha_y) \cdot \cos(\theta) \cdot \sigma_y' \cdot K_{\text{M-O}} \, dh - \int \sigma_y' \cdot K_{\text{M-O,active}} \, dh \]

The seismic thrust is then redistributed according to the Seed & Whitman (1970) recommendation as an inverse trapezoid with the resultant force acting at 0.6H above the wall bottom (or bottom of excavation depending on the selected height option). This results in the pressure at the top and bottom being equal to:

\[ q_{\text{EQ,Top}} = \frac{1.6 \cdot F_{\text{EQ}}}{B} \]
\[ q_{\text{EQ,Bottom}} = \frac{U_B \cdot F_{\text{EQ}}}{H} \]
12.3.3 Richards Shi method
The Richards-Shi method (1994) provides an extension to the M-O approach that includes frictional soils that have cohesion. For free field conditions the proposed equation is outlined as:

\[
K_{HZ} = \frac{1 + \sin^2(\phi)}{\cos^2(\phi)} + \frac{c}{\gamma Z} + \frac{2}{\gamma Z} \tan(\phi) - \frac{2}{\gamma Z} \left(\frac{\tan(\phi)}{\gamma Z}\right)^2 - \left(\frac{\tau_{XZ}}{\gamma Z}\right)^2
\]

With zero acceleration the above equation becomes:

\[
K_{H} = \frac{1 + \sin^2(\phi)}{\cos^2(\phi)} + \frac{c}{\gamma Z} + \frac{2}{\gamma Z} \tan(\phi) - \frac{2}{\gamma Z} \left(\frac{\tan(\phi)}{\gamma Z}\right)^2
\]

Where:
- \(\sigma_Z = (1 - \alpha_Y) \gamma Z - U_{TOP}\) (program uses total vertical stress)
- \(\sigma_{Z0} = \gamma Z - U_{TOP}\) (program uses total vertical stress)
- \(\tau_{XZ} = -\alpha_X \gamma Z\)

The program then calculates:

\[
K_{NET,H} = K_{HZ} - K_{H} \frac{\sigma_{Z0}}{\sigma_Z}
\]

The net seismic thrust increase is then calculated as:

\[
K_{EQ} = \sum (1 - \alpha_Y) \times K_{NET,H} \cdot \sigma_Z \, dH
\]

The seismic pressures are then redistributed as an inverse trapezoid in a similar manner outlined for the M-O method. In this approach, the soil is inherently treated as impermeable. Inclusion of hydrodynamic pressures will always provide a conservative approach. The software program always includes seismic pressures calculated with the Richards-Shi method as external loads.

12.3.4 User specified external
In this approach the user can directly input the seismic pressures as an external trapezoidal or rectangular pressure diagram.
12.3.5 Wood Automatic method

In this approach the first step is to determine the average lateral thrust of the soil according to the wood approach with:

\[ F_{EQ} = \sum \alpha_W \cdot (1 - \alpha_T) \cdot \gamma_{APPLIED} \cdot dH \]

Where \( \gamma_{APPLIED} \) = Applicable soil unit weight with the following possibilities (also see 12.4):

a) Dry above the water table
b) Total when the soil below the water table is treated as impervious, or
c) Total minus unit weight of water for soil below the water table that is treated as pervious where water can move independently from the soil skeleton.

Then the seismic pressures can be calculated as:

\[ q_{EQ,TOP} = q_{EQ,BOTTOM} = \frac{F_{EQ}}{H} \]

In the limit equilibrium approach the calculated pressures are applied directly on the wall. Therefore with this approach the wall is implicitly assumed to be rigid.

Within the Paratie analysis these pressures are applied as the initial seismic thrust pressures at zero additional seismic strain (i.e. rigid wall behavior when the seismic pressures are initially applied). The initial seismic pressures are then gradually readjusted (typically reduced) as the wall gradually displaces due to the additional seismic load until equilibrium is reached. Details of this automatic procedure are fully outlined in the Paratie engine theory manual.

12.3.6 Wood Manual

This approach behaves in exactly the same manner as the Automatic Wood Method with the only difference being that the zero strain seismic pressures are defined directly by the user.

12.4 Water Behavior during earthquakes

For retaining walls hydrodynamic water pressures are typically calculated with Westergaard’s solution (Westergaard, 1931) and are always applied as external pressures to a wall where the hydrodynamic pressure is calculated as:

\[ p_W = \frac{V}{2} \cdot c_W \cdot \gamma_W \cdot \sqrt{2gH} \]
For retained water above the ground surface external hydrodynamic pressures are included with the previous equation. In case of water above the subgrade (excavation level) on the excavated side, hydrodynamic relief pressures are also included in the excavated side in the direction of the horizontal acceleration.

Depending on soil permeability, water within the soil can move with or independently from the soil skeleton. When soil is treated as pervious hydrodynamic pressures from groundwater will be added (on both retained an excavation side, but towards the same direction). The program offers the following options:

a) All soils are treated as impervious. Water is not free to move independently, hydrodynamic effects for groundwater are not included for any soils.

b) All soils are treated as pervious. Water is free to move independently, hydrodynamic effects for groundwater are included for all soils.

c) Automatic determination of behavior according to Eurocode 8 recommendations. Soils with permeability greater than $5 \times 10^{-4}$ m/sec are automatically treated as pervious while soils with smaller permeability are treated as impermeable.

In case of water flow an alternative approach is to calculate the hydrodynamic pressure based on the actual water pressures (Konstantakos, 2009):

$$p_N = \frac{7}{8} \cdot \alpha_N \cdot \sqrt{Y_{wN}} \cdot u \cdot H$$

**Note:** This equation is an extension of the Westegaard theory that is difficult to verify in real projects. In steady flow conditions, the above equation will produce smaller pressures compared to the traditional Westegaard equation.

### 12.5 Wall Inertia Seismic Effects

Traditionally wall inertia effects (acceleration of the wall mass directly) are not included for flexible retaining structures. The program offers the ability to include these effects.
12.6 Verification Example

Let’s imagine a wall that is 10m deep, with water at the same elevation on both sides of the wall. The example involves the analysis of steel sheet pile wall supported by a single level of tiebacks with the following assumptions:
- Retained ground surface level (uphill side) El. +0
- Maximum excavation level (downhill side) El. -10
- Water level on retained side El. 0
- Water level on excavated side El. 0
- Water density $\gamma_{WATER} = 10kN/m^3$
- Horizontal design acceleration $a_x = 0.25g$
- Vertical design acceleration $a_z = 0.125g$ (upwards)
- Wall friction $\delta = 11$ degrees
- Soil properties: $\gamma_{TOTAL} = 21.55kN/m^3$, $\gamma_{DRY} = 18.55kN/m^3$, $c' = 0$ kPa, $\phi = 32$ deg,
  Permeability $K_x = 0.001$ m/sec
  Exponential soil model: $E_{load} = 15000$ kPa, $E_{reloading} = 45000$ kPa, $a_h = 1$, $a_v = 0$
  $K_{p Base} = 3.225$ (Rankine), $K_{a Base} = 0.307$ (Rankine)

Note that according to Coulomb with wall friction 11 degrees we obtain:
$K_{p Coulomb} = 3.301$, $K_{a Coulomb} = 0.278$

Part A: Calculate the driving seismic thrust from soil and water and the corresponding pressures with the Mononobe-Okabe method assuming that the soil is pervious.

$$\theta := \arctan\left(\frac{\gamma_d \cdot A_x}{(\gamma_t - \gamma_w) \cdot (1 - Ay)}\right)$$
for pervious soil

$$\beta = 0$$
$$\beta = 0 \text{ deg}$$
$$\theta = 24.649 \text{ deg}$$

According to Mononobe Okabe if $B < FR - \Theta$

$$\text{test1} := \phi - \theta$$
$$\text{test1} = 7.351 \text{ deg}$$

$$K_{AE} := \frac{\left(\sin(\psi + \phi - \theta)\right)^2}{\cos(\theta) \cdot (\sin(\psi))^2 \cdot \sin(\psi - \theta - \delta l) \cdot \left[1 + \left(\frac{\sin(\delta l + \phi) \cdot \sin(\phi - \beta - \theta)}{\sin(\psi - \theta - \delta l) \cdot \sin(\psi + \phi)}\right)^{0.5}\right]^2}$$

$$K_{AE} = 0.756$$

In the horizontal direction
$$K_{AE, h} := K_{AE} \cdot \cos\left(\frac{\pi}{2} - \psi + \delta l\right)$$
$$K_{AE, h} = 0.742$$
The effective vertical stress at the wall bottom is:

\[ \sigma'_v = \gamma z - U = 21.55 \times 10 - 10 \times 10 = 115.5 \text{ kN/m}^2 \]

The net seismic thrust due to the soil force is then calculated as:

\[ F_{x, \text{SoilEQ}} = (K_{AE,H} \times (1-ay) - K_{ACoul}) \frac{\sigma'_v H}{2} = 214.4 \text{ kN/m} \]

Then the seismic soil pressure at the wall top and bottom can be calculated as:

\[ q_{\text{EQ,top}} = \frac{1.6 \cdot F_{EQ}}{H} = 34.3 \text{ kPa} \]
\[ q_{\text{EQ,bottom}} = \frac{0.4 \cdot F_{EQ}}{H} = 8.57 \text{ kPa} \]

The hydrodynamic pressure from one wall side at the wall bottom is calculated as:

\[ p_w = \frac{7}{6} \cdot a_r \cdot \gamma_w \cdot \sqrt{2} \frac{H}{H} = 21.875 \text{ kPa} \]

Since water is also found on the excavated side, the hydrodynamic force must be accounted twice, thus:

\[ q_{\text{EQ,top}} = 34.3 \text{ kPa} + 0 \text{ kPa} = 34.3 \text{ kPa} \]
\[ q_{\text{EQ,bottom}} = 8.57 \text{ kPa} + 2 \times 21.875 \text{ kPa} = 52.33 \text{ kPa} \]

The program produces almost the same results:
**Figure 12.5: Verification of MO seismic pressures with software, Part A, Automatic water behavior (pervious in this case).**

**Part B:** Next we will examine if the soil behaves in as impervious. In this case, hydrodynamic pressures must be accounted only in the excavated side:

$$\theta := \text{atan} \left( \frac{\gamma t \cdot A x}{(\gamma t - \gamma w) \cdot (1 - A y)} \right)$$

for impervious soil

$$\beta = 0 \quad \beta = 0 \text{deg} \quad \theta = 28.061 \text{deg}$$

According to Mononobe Okabe if B < FR - THETA

$$\text{test1} := \phi - \theta \quad \text{test1} = 3.939 \text{deg}$$

$$K_{AE} := \frac{(\sin(\psi + \phi - \theta))^2}{\cos(\theta) \cdot (\sin(\psi))^2 \cdot \sin(\psi - \theta - \delta l) \cdot \left[ 1 + \left( \frac{\sin(\delta l + \phi) \cdot \sin(\phi - \beta - \theta)}{\sin(\psi - \theta - \delta l) \cdot \sin(\psi + \beta)} \right) \right]^{0.5}^2}$$

$$K_{AE} = 0.936$$

In the horizontal direction

$$K_{AE,h} := K_{AE} \cdot \cos \left( \frac{\pi}{2} - \psi + \delta l \right) \quad \text{K}_{AE,h} = 0.919$$

The net seismic thrust due to the soil force is then calculated as:

$$F_{x, \text{SoilEQ}} = (K_{AE,h} \times (1 - ay) - K_{AE, Coul}) \cdot \sigma_{V'} H/2 = 303.8 \text{ kN/m}$$

$$q_{\text{EQ, Top Soil}} = \frac{1.6 \cdot F_{EQ}}{H} = 46.61 \text{ kPa}$$

$$q_{\text{EQ, Bottom Soil}} = \frac{0.4 \cdot F_{EQ}}{H} = 12.15 \text{ kPa}$$

$$q_{\text{EQ, Top}} = 48.61 \text{ kPa} + 0 \text{ kPa} = 48.61 \text{ kPa}$$

$$q_{\text{EQ, Bottom}} = 12.15 \text{ kPa} + 21.875 \text{ kPa} = 34.02 \text{ kPa}$$

These results are verified by the program, (note that the maximum pressure is found below the wall top):
Figure 12.6: Verification of MO seismic pressures with software, Part B, impervious water behavior.

Part C: Calculate the seismic thrust assuming the semirigid method with \( B = 0.75 \) and no water on both sides. In this case, the effective vertical stress at the wall bottom is:

\[
\sigma_V' = \gamma z - U = 18.55 \times 10 - 0 = 185.5 \text{ kN/m}^2
\]

Then the seismic thrust, as also verified by the software, is taken as a rectangular diagram equal to:

\[
q_{eq} = 185.5 \times 0.25 \times (1 - 0.125) \times 0.75 = 30.4 \text{ kPa}
\]

Figure 12.7: Verification of MO seismic pressures with software, Part C, no water.

If a rigid behavior is assumed then

\[
q_{eq} = 185.5 \times 0.25 \times (1 - 0.125) = 40.6 \text{ kPa}
\]
13. Verification of free earth method for a 10ft cantilever excavation

The purpose of this exercise is to calculate lateral stresses, toe embedment, and bending moments with the free earth method for a 10 ft cantilever excavation. Pressure calculations and assumptions are:

Left Side El. = 0 FT  Right Side El. = -10 FT  Gen. Water El= -10 FT

Soil $\gamma = 120$ pcf  Friction Angle=30 deg  Water $\gamma = 62.4$ pcf

Active on left side $k_a = 0.333$  Passive on right side $k_p = 3$

<table>
<thead>
<tr>
<th>Soil Weight (kPa)</th>
<th>Water Weight (kPa)</th>
<th>Soil Elevation (ft)</th>
<th>Water Elevation (ft)</th>
<th>$k_a$</th>
<th>$k_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.2</td>
<td>0</td>
<td>-10</td>
<td>-10</td>
<td>0.333</td>
<td>3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Elevation (ft)</th>
<th>Total Vertical Stress (kPa)</th>
<th>Water Vertical Pressure (kPa)</th>
<th>Effective Vertical Stress (kPa)</th>
<th>Lateral Total Stress (kPa)</th>
<th>Net (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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<td>0</td>
<td>0</td>
<td>0</td>
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<td>-20</td>
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<td>-1.218</td>
</tr>
<tr>
<td>-60</td>
<td>6</td>
<td>-2.406</td>
<td>3.504</td>
<td>-1.188</td>
<td>-3.684</td>
</tr>
</tbody>
</table>

Figure 13.1: Cantilever excavation lateral pressure calculations
LATERAL NET PRESSURES ABOVE SUBGRADE

\[ k_{\text{ps}} = 1000 \text{lb} / \text{ft}^2 \quad k_{\text{sfs}} := \frac{k_{\text{ps}}}{2} \]

\[ \sigma_{\text{top}} := 0 \text{ksf} \quad \sigma_{\text{bot}} := -0.4 \text{ksf} \quad L_c = 10 \text{ft} \]

Lateral force above subgrade

\[ F_1 := (\sigma_{\text{top}} + \sigma_{\text{bot}}) \cdot L_c \cdot \frac{1}{2} \quad F_1 = -2 \text{kip} \]

Centroid to force above subgrade

\[ L_c1 := \frac{L_c}{3} \quad L_c1 = 3.333 \text{ ft} \]

LATERAL NET PRESSURES BELOW SUBGRADE

\[ \sigma_{\text{sub}} := \sigma_{\text{bot}} \quad \sigma_{\text{sub}} = -0.4 \text{ksf} \]

At bottom of wall (EI=50)\n\[ \sigma_{\text{bw}} := 5.552 \text{ksf} \]

Wall length below subgrade\n\[ L_{\text{wb}} = 40 \text{ft} \]

Passive pressure slope\n\[ m_p = \frac{\sigma_{\text{bw}} - \sigma_{\text{sub}}}{L_{\text{wb}}} \quad m_p = 0.149 \text{ ksf/ft} \]

Depth to zero passive pressure from subgrade\n\[ E_{L_0} := \frac{-\sigma_{\text{sub}}}{m_p} \quad E_{L_0} = 2.688 \text{ ft} \]

Lateral force from subgrade to ELo\n\[ F_2 := \frac{(\sigma_{\text{sub}} \cdot E_{L_0})}{2} \quad F_2 = -0.538 \text{ kip} \]

Now in order to find toe embedment depth for a safety factor of 1, the total net moment must be zero

Assumed depth from Elo to TOE FS 1 Elevation\n\[ d_1 := 12.075 \text{ ft} \]

Sum moments above ELo\n
\[ M_{\text{top}} := \left[ F_1 \left( L_c1 + E_{L_0} + d_1 \right) \right] + \left[ F_2 \left( d_1 + E_{L_0} \cdot \frac{2}{3} \right) \right] \quad M_{\text{top}} = -43.648 \text{ kip-ft} \]

Lateral net pressure at TOE FS1 Elevation\n\[ \sigma_{\text{FS1}} = d_1 \cdot m_p \quad \sigma_{\text{FS1}} = 1.797 \text{ ksf} \]

NET Resisting lateral force below Elo\n\[ F_3 := \sigma_{\text{FS1}} \cdot \frac{d_1}{2} \quad F_3 = 10.848 \text{ kip} \]

NET Resisting Moment\n\[ M_{\text{BOT}} := F_3 \cdot \frac{d_1}{3} \quad M_{\text{BOT}} = 43.663 \text{ kip-ft} \]

TOTAL NET MOMENT\n\[ M_{\text{NET}} := M_{\text{BOT}} + M_{\text{top}} \quad \text{Which is equal to zero} \]

Elevation at safety factor of 1\n\[ E_{L_{FS1}} := -10 \text{ft} - E_{L_0} - d_1 \quad E_{L_{FS1}} = -24.763 \text{ ft} \]
Figure 13.2: Output for 10ft cantilever excavation.

The same problem in DEEP can be seen in Figure 13.2. DEEP generates the same lateral pressures and predicts FS. 1 embedment is EL. -24.5 ft. Our calculations have showed that the FS. 1 elevation is EL. -24.76 ft. DeepXcav essentially finds the same number. The difference stretches an important aspect of how discretization (i.e. dividing the wall into many elements) can generate slight differences from the actual solution.

The safety factor on embedment length is calculated as:

\[
FS_{\text{embed}} = \frac{40\text{ft}}{10\text{ft} - (-24.5\text{ft})} = 2.758
\]
The embedment safety factor based on the horizontal forces is calculated by dividing the resisting horizontal forces by the driving horizontal forces. DeepXcav calculates a safety factor of 4.136 whereas the following calculation shows $FS=2.213$. The discrepancy is likely due to rounding errors as all the significant digits are carried in internal calculations within DeepXcav.

The effective vertical stress at El. -10ft and at El. -50 ft is:

$$\sigma_{V-10'} = \gamma z - U = 0.120 \text{ kcf} \times 10 \text{ ft} - 0 = 1.2 \text{ ksf}$$

$$\sigma_{V-50'} = \sigma_{V-10'} + (\gamma - \gamma_w)z = 1.2 \text{ ksf} + (0.120 - 0.0624) \text{ kcf} \times 40 \text{ ft} = 3.504 \text{ ksf}$$

However, the water is calculated with the net method where the right side water pressures are subtracted from the left side water pressures. Hence, in this example the net water pressure is equal to zero.

Driving force $= 0.333 \times 1.20 \text{ ksf} \times 10\text{ft}/2 + 0.333 \times (1.2\text{ksf} + 3.504 \text{ ksf}) \times 40 \text{ ft}/2 = 33.35 \text{ kips/ft}$

Resisting force $= 3 \times (0.120 - 0.0624) \text{ kcf} \times 40 \text{ ft} \times 40\text{ft}/2 = 138.24 \text{ k/ft}$

Then the horizontal passive force safety factor is:

$$F_{Spas} = 138.24/33.35 = 4.145 \text{ while the software produces 4.136}$$

Now find the maximum bending moment. In order to achieve this, the point of zero wall shear must be found first.

Shear to Elo $V_{top} = F_1 + F_2 \quad V_{top} = -2.538 \text{kip}$

Depth to zero shear $d_0 = \sqrt{\frac{-2 \times V_{top}}{m_p \times 1\text{ft}}} \quad d_0 = 5.84\text{ft}$

Maximum moment at zero shear

$$M_{max} = \left[ F_1 \cdot (L_{c1} + EL_c + d_0) \right] + \left[ F_2 \left( d_0 + EL_c \frac{2}{3} \right) \right] - V_{top} \frac{d_0}{3} \quad M_{max} = -22.887 \text{kip-ft}$$

DeepXcav generates a bending moment of 22.4 k-ft which verifies the calculated maximum moment above. The slight discrepancy is because of rounding and because when the wall is divided into small elements the exact maximum moment position will likely be missed by the wall node where the maximum moment is reported.
14. **Verification of 20 ft deep single-level-supported excavation**

Using the same general parameters as in section 14.1, we will now verify analyze a problem of a 20 ft excavation that is supported by a single support level at El. -10 ft (10 ft depth from top of wall). The aim of this task will be to find the required toe-embedment for a safety factor of 1.0, the support reaction, and the maximum wall bending moment. Lateral pressures calculations are shown in Figure 1.

<table>
<thead>
<tr>
<th>SOIL UNIT</th>
<th>WATER TABLE ELEV (FT)</th>
<th>WATER WEIGHT (kcf)</th>
<th>WATER ELEV (FT)</th>
<th>K_s</th>
<th>K_p</th>
</tr>
</thead>
<tbody>
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<td>-20</td>
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</table>

**LEFT EXCAVATION SIDE PRESSURES**

<table>
<thead>
<tr>
<th>ELEV (FT)</th>
<th>TOTAL VERTICAL STRESS (ksf)</th>
<th>WATER PRESSURE (ksf)</th>
<th>EFFECTIVE VERTICAL STRESS (ksf)</th>
<th>LATLAL SOIL STRESS (ksf)</th>
<th>TOTAL LATERAL STRESS (ksf)</th>
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</thead>
<tbody>
<tr>
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<td>-3.684</td>
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</table>

**RIGHT SIDE PRESSURES**

<table>
<thead>
<tr>
<th>ELEV (FT)</th>
<th>TOTAL VERTICAL STRESS (ksf)</th>
<th>WATER PRESSURE (ksf)</th>
<th>EFFECTIVE VERTICAL STRESS (ksf)</th>
<th>LATLAL SOIL STRESS (ksf)</th>
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</thead>
<tbody>
<tr>
<td>-20</td>
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<td>0</td>
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</tr>
<tr>
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<td>8</td>
<td>-2.406</td>
<td>3.504</td>
<td>-1.166</td>
<td>-3.684</td>
</tr>
</tbody>
</table>

**LATERAL STRESS (KSF)**

Figure 14.1: Lateral pressure calculations for 20 ft single level supported excavation.
**LATERAL NET PRESSURES ABOVE SUPPORT**

\[ \text{kips} := 1000 \text{lb} \quad \text{ksf} := \frac{1 \text{kips}}{\text{ft}^2} \quad \sigma_{\text{top}} := 0 \text{ksf} \quad \sigma_{\text{bot}} := -4 \text{ksf} \quad L_c := 10 \text{ft} \]

Lateral force above support 
\[ F_1 = (\sigma_{\text{top}} + \sigma_{\text{bot}}) \cdot L_c \cdot \frac{h}{2} \quad F_1 = -2 \text{kips} \]

Centroid to force above support 
\[ L_{c1} := \frac{L_c}{3} \quad L_{c1} = 3.333 \text{ ft} \]

Moments above support 
\[ M_1 = L_{c1} \cdot F_1 \]
\[ M_1 = -6.667 \text{kips ft} \]

**LATERAL NET PRESSURES BELOW SUPPORT AND ABOVE SUBGRADE EL-20ft**

\[ \sigma_{\text{sub}} := -1.216 \text{ksf} \quad L_2 := 10 \text{ ft} \quad \text{DEPTH FROM SUPPORT TO SUBGRADE} \]

Rectangular portion force 
\[ F_{\text{rect}} := \sigma_{\text{bot}} \cdot L_2 \cdot 1 \text{ ft} \quad F_{\text{rect}} = -4 \text{kips} \]

Moment about support 
\[ M_{2\text{RECT}} := -F_{\text{rect}} \cdot \frac{L_2}{2} \quad M_{2\text{RECT}} = 20 \text{kips ft} \]

Triangular portion of force 
\[ F_{\text{tri}} := (\sigma_{\text{sub}} - \sigma_{\text{bot}}) \cdot \frac{L_2 \cdot 1 \text{ ft}}{2} \quad F_{\text{tri}} = -4.08 \text{kips} \]

Moment about support 
\[ M_{2\text{TRI}} := -F_{\text{tri}} \cdot L_2 \cdot \frac{L_2}{3} \quad M_{2\text{TRI}} = 27.2 \text{kips ft} \]

\[ M_2 = M_{2\text{TRI}} + M_{2\text{RECT}} \quad M_2 = 47.2 \text{kips ft} \]

**LATERAL NET PRESSURES BELOW SUBGRADE EL-20ft**

At bottom of wall (EL-50) 
\[ \sigma_{bw} := 3.243 \text{ksf} \quad \text{Wall length below subgrade} \quad L_{wb} := 30 \text{ft} \]

Passive pressure slope 
\[ m_p := \frac{(\sigma_{bw} - \sigma_{sub})}{L_{wb}} \quad m_p = 0.149 \frac{\text{ksf}}{\text{ft}} \]

Depth to zero passive pressure from subgrade 
\[ EL_o := \left( \frac{-\sigma_{sub}}{m_p} \right) \quad EL_o = 8.172 \text{ ft} \]

Lateral force from subgrade to ELo 
\[ F_3 := \left( \frac{(\sigma_{sub} \cdot EL_o) + 1 \text{ ft}}{2} \right) \quad F_3 = -4.969 \text{kips} \]

Moment about support 
\[ M_3 := -\left( \frac{L_2 + \frac{EL_o}{3}}{2} \right) F_3 \quad M_3 = 63.221 \text{kips ft} \]

Net moment above ELo 
\[ M_{\text{NET1}} := M_1 + M_2 + M_3 \quad M_{\text{NET1}} = 103.754 \text{kips ft} \]

To find toe embedment depth for a safety factor of 1, the total net moment must be zero

Assume depth to FS1 below ELo 
\[ d_1 = 7.733 \text{ ft} \]

Pressure at d1 
\[ \sigma_{FS1} := m_p \cdot d_1 \quad \sigma_{FS1} = 1.151 \text{ksf} \quad F_4 := \sigma_{FS1} \cdot d_1 \cdot \frac{1 \text{ ft}}{2} \quad F_4 = 4.448 \text{kips} \]

Moment about support 
\[ M_4 := -\left( \frac{L_2 + EL_o + d_1 \cdot \frac{2}{3}}{2} \right) F_4 \quad M_4 = -103.764 \text{kips ft} \]

**NET MOMENT** 
\[ M_{\text{NET}} := M_{\text{NET1}} + M_4 \quad M_{\text{NET}} = -0.01 \text{kips ft} \]

Elevation at FS1 
\[ EL_{FS1} := -20 \text{ ft} - EL_o - d_1 \quad EL_{FS1} = 31.904 \text{ ft} \]
Figure 14.2: DeepXcav output for 20 ft deep excavation with single support at 10 ft depth.

As we can see in Figure 2, DEEP predicts that the FS=1.0 elevation is at EL. -35.75 ft while the previous calculations show EL. -35.9 ft. Therefore, our calculations essentially confirm the numbers found by DeepXcav.

The wall embedment safety factor based on length is calculated as:

$$FS_{\text{embed.length}} = \frac{30 \text{ ft}}{(-20 \text{ ft} - (-35.75 \text{ ft}))} = 1.904$$

Now in order to find the support force the sum of all forces above EL is 1 must be zero

$$\Sigma F_x = F_1 + F_{\text{inh}} + F_{\text{rect}} + F_3 + F_4 \quad \Sigma F_x = -10.6 \text{ kips}$$

Therefore the support reaction must be

$$R_{\text{SUPPORT}} = -\Sigma F_x \quad R_{\text{SUPPORT}} = 10.6 \text{ kips}$$

DeepXcav finds the same number for the support reaction which is calculated at

$$R_{\text{support}} = 10.48 \text{ kips (per foot of wall length)}.$$  

Using a commercial structural analysis software program we can input the parameters and calculate the moment easily while we can also verify the calculated support reactions above. As it can be seen in Figure 14.3, the calculated maximum moment is 45.9 k-ft/ft while DeepXcav predicts a maximum bending moment of 44.6 k-ft/ft. Thus, DeepXcav predicts the maximum bending moment accurately.
Figure 14.3: Calculated Moments with a structural analysis program for 20 ft excavation.

Following are calculations for the safety factor against rotation. The safety factor is calculated by the ratio of resisting to driving moments about the support level. DeepXcav calculates a safety factor of 1.915 whereas the following calculations show a safety factor of 1.915. Thus DeepXcav captures the correct safety factor accurately.
The hand calculations confirm the calculated rotational safety factor software calculates a rotational safety factor of 1.915.
15. Verification of 30 ft excavation with two support levels

This section verifies a 30 ft excavation with two support levels, one at 10 ft depth, and the second at 20 ft from the top of the wall. All other project parameters remain the same as in the previous two sections (14.1 and 14.2). Figure 15.1 shows pressure calculations for the 30 ft excavation. DeepXcav calculates the same lateral pressures as calculations in Figure 15.1.

![Left Excavation Side Pressures](image)

<table>
<thead>
<tr>
<th>ELEV. (FT)</th>
<th>TOTAL VERTICAL STRESS (ksf)</th>
<th>WATER PRESSURE (ksf)</th>
<th>EFFECTIVE VERTICAL STRESS (ksf)</th>
<th>LATERAL SOIL STRESS (ksf)</th>
<th>TOTAL LATERAL STRESS (ksf)</th>
<th>NET (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>-10</td>
<td>1.2</td>
<td>-0.624</td>
<td>1.776</td>
<td>-0.592</td>
<td>-1.216</td>
<td>-0.4</td>
</tr>
<tr>
<td>-20</td>
<td>2.4</td>
<td>-1.248</td>
<td>2.352</td>
<td>-0.784</td>
<td>-2.032</td>
<td>-1.216</td>
</tr>
<tr>
<td>-30</td>
<td>3.6</td>
<td>-1.248</td>
<td>2.352</td>
<td>-0.784</td>
<td>-2.032</td>
<td>0</td>
</tr>
<tr>
<td>-43.22</td>
<td>5.186</td>
<td>-2.073</td>
<td>3.113</td>
<td>-1.038</td>
<td>-3.111</td>
<td>1.586</td>
</tr>
<tr>
<td>-50</td>
<td>6</td>
<td>-2.496</td>
<td>3.504</td>
<td>-1.168</td>
<td>-3.664</td>
<td>2.4</td>
</tr>
</tbody>
</table>

![Lateral Stress Graph](image)

Figure 15.1: Lateral pressure calculations for a 30 ft excavation
Figure 15.2: Lateral pressures and bending moments for a 30ft deep excavation by DeepXcav.

Figure 15.3: 30ft Excavation modeled with a structural analysis software
The excavation has been modeled with an independent structural software program with a base support at El -43.22 where the net pressure is equal to zero. Results from the independent software program are shown in Figure 15.3. The software program results in wall moments of 72.5 k-ft/ft at El. -20 ft and 65.0 k-ft maximum positive on the right side. DeepXcav produces a moment of 72.6 k-ft/ft at El. -20 and 65.0 k-ft/ft on the right wall side. Support reactions in DEEP are -1.24 k/ft at the first support level and 32.01 k/ft at the second support level (El. -20 ft). Therefore, DeepXcav within acceptable accuracy levels calculates moments and support reactions.

![Figure 15.4: Wall embedment safety factors, wall moments, and support reactions by DEEP for a 30 ft excavation example.](image)

Now calculate the toe-embedment safety factors.

Reaction at pin support at El -43.22 ft \( F_B := 9.00 \text{kip} \)

Note that the pressure at El -43.22 is zero. Now calculate the net passive resistance to the bottom of the wall.

\[ \sigma_{BOT} := 0.944 \text{ksf} \]

Therefore, the next passive resisting force below El. -43.22 is

\[ R_{NET} := (50 \text{ft} - 43.22 \text{ft}) \cdot \frac{\sigma_{BOT} \cdot 1 \text{ft}}{2} \]

Passive force safety factor \( F_{SPAS} := \frac{R_{NET}}{F_B} \)

\[ F_{SPAS} = 0.356 \]

DeepXcav calculates 0.36 which verifies the hand calculated safety factor.
Now calculate rotational safety factor about lowest support. In this method, driving and resisting moments below the lowest support are calculated. The safety factor is then calculated at the ratio of resisting to driving moments. Note that moments above the lowest support are ignored.

Soil pressure at El- 20 \( \sigma_{DRs20} := 0.592 \text{ksf} \)
Soil pressure at bottom of left wall side El- 50ft \( \sigma_{DRsb} := 1.168 \text{ksf} \)

From the rectangular portion of the driving soil pressures
\[
F_{DRrectS} := 30\text{ft} \cdot 1\text{ft} \left( \sigma_{DRs20} \right) \quad F_{DRrectS} = 17.76 \text{kip}
\]
\[
M_{DRrectS} := F_{DRrectS} \cdot \frac{10\text{ft} \cdot \frac{1}{2}}{2} \quad M_{DRrectS} = 266.4 \text{kip ft}
\]

From the triangular portion of the driving pressures
\[
F_{DRtriS} := \left( \sigma_{DRsb} - \sigma_{DRs20} \right) \cdot 30\text{ft} \cdot \frac{10\text{ft} \cdot \frac{1}{2}}{2} \quad F_{DRtriS} = 8.64 \text{kip}
\]
\[
M_{DRtriS} := F_{DRtriS} \cdot \frac{30\text{ft} \cdot \frac{2}{3}}{2} \quad M_{DRtriS} = 172.8 \text{kip ft}
\]

And total driving moment due to soil pressures on left side \( M_{DRs} := M_{DRrectS} + M_{DRtriS} \quad M_{DRs} = 439.2 \text{kip ft} \)

Now calculate the net driving moment due to water below El. -20ft
\( u_{20} := 0.624 \text{ksf} \quad u_{30} := 1.248 \text{ksf} \)

Moment for water from El. -20 to El. -30ft
\[
M_{1w} := \left[ \frac{10\text{ft} \cdot u_{20} \cdot 10\text{ft}}{2} + \left( u_{30} - u_{20} \right) \cdot \frac{10\text{ft} \cdot \frac{1}{2}}{2} \cdot \frac{10\text{ft} \cdot \frac{2}{3}}{3} \right] \cdot 1\text{ft} \quad M_{1w} = 52 \text{kip ft}
\]

Below El. -30 the net water pressure has a rectangular distribution
\( u_{\text{net}} := u_{30} \quad u_{\text{net}} = 1.248 \text{ksf} \) and the moment contribution is
\[
M_{2w} := u_{\text{net}} \cdot 20\text{ft} \left( 10\text{ft} + \frac{20\text{ft}}{2} \right) \cdot 1\text{ft} \quad M_{2w} = 499.2 \text{kip ft}
\]

The driving moment is then: \( M_{DR} := M_{DRs} + M_{1w} + M_{2w} \)

Resisting moment comes from a triangular pressure distribution pressure at bottom 3.456 ksf only due to soil contribution as water moment is added as a net moment on the driving side
\[
F_{R} := 3.456 \text{ksf} \cdot 20\text{ft} \cdot \frac{1\text{ft}}{2} \quad M_{R} := F_{R} \left( 10\text{ft} + \frac{20\text{ft}}{3} \right) \quad M_{R} = 806.4 \text{kip ft}
\]

Now we can calculate the rotational safety factor \( \text{FS}_{\text{ROT}} := \frac{M_{R}}{M_{DR}} \quad \text{FS}_{\text{ROT}} = 0.814 \)

As Figure 15.4 shows, DeepXcav calculates the same safety factor as calculated above.
A. APPENDIX: Verification of Passive Pressure Coefficient Calculations

1. Calculate $K_p$ according to various equations, define basic parameters

Soil friction angle $\phi := 40\text{deg}$
Slope angle $\alpha := 15\text{deg}$ Note that positive slope angle is upwards
Wall friction $\delta l := 10\text{deg}$
Wall inclination $\theta := 0\text{deg}$ Note vertical face angle theta is 0

Seismic accelerations
$A_x := 0.16$ $A_y := 0$
$\beta := \tan^{-1}\left(\frac{A_x}{1 - A_y}\right)$ $\beta = 0.159$ $\beta = 9.09\text{deg}$

2. Calculate $K_p$ according to Coulomb, DAS pg. 430, Principles of Geotechnical Engineering

$$K_{P1} := \frac{\cos(\phi + \theta - \beta)}{(\cos(\theta))^2(\cos(\beta))^2 \cos(\delta l - \theta + \beta)\left[1 - \left(\frac{\sin(\delta l + \phi) \cdot \sin(\phi + \alpha - \beta)}{\cos(\delta l - \theta + \beta) \cdot \cos(\alpha - \theta)}\right)^{0.5}\right]^2}$$

$K_p := K_{P1}(1 - A_y)$ $K_p = 15.976$

$K_{PH} := K_p \cdot \cos(\delta l - \theta)$ $K_{PH} = 15.734$

3. Calculate $K_p$ according to Lancellotta 2007, note equation does not account for wall inclination

$$\Theta_2 := \sin\left(\frac{\sin(\phi)}{\sin(\delta l)}\right) + \sin\left(\frac{\sin(\alpha - \beta)}{\sin(\phi)}\right) + \delta l + (\alpha - \beta) + 2\beta$$ $\Theta_2 = 1.029$

$$\gamma_1 := \left[(1 - A_y)^2 + A_x^2\right]^{0.5}$$ $\gamma_1 = 1.013$ $\Theta_2 = 58.981\text{deg}$

$$K_{PE} := \cos(\delta l) \cdot \left[\frac{\cos(\delta l) + \left[(\sin(\phi))^2 - (\sin(\delta l))^2\right]^{0.5}}{\cos(\alpha - \beta) - \left[(\sin(\phi))^2 - (\sin(\alpha - \beta))^2\right]^{0.5}}\right]^{\Theta_2 \cdot \tan(\phi)}$$ $K_{PE} = 10.401$

$$K_{PH,\text{Lancellotta}} := K_{PE} \cdot \gamma_1 \cdot \cos(\alpha - \beta)$$ $K_{PH,\text{Lancellotta}} = 10.477$

$$K_{P,\text{Lancellotta}} := \frac{K_{PH,\text{Lancellotta}}}{\cos(\delta l)}$$ $K_{P,\text{Lancellotta}} = 10.639$

In the following page, the results by the software program are presented.
The soil Kp estimation form can be raised in two ways a) From the Soil Data dialog when the Paratie analysis is selected, and b) by writing Kp Estimate in the text command and pressing enter.
DeepXcav theory manual: Developed by Ce.A.S. srl, Italy and Deep Excavation LLC, U.S.A.

![Graph showing Kp vs. Soil Friction Angle]

- Friction Angle, Wall Friction, and Slope Angle B:
  - $\phi = 40^\circ$
  - $\beta = 15^\circ$
  - $\delta = 10^\circ$

- Passive Coefficient Values:
  - $Kp$:
    - Casquet-Kierke: 12.213
    - Casquet-Tables: 12.528
    - Lancellotta: 10.539
    - Coulomb: 16.895
  - $Kph$:
    - Accept Value of $Kfh$: 12.027
    - Accept Value of $Kph$: 12.337
    - Accept Value of $Kph$: 10.379
    - Accept Value of $Kph$: 16.638

Kfh = Passive Coefficient in Horizontal Direction. Program uses $Kph$.

Exit (Without Accepting New Values)
B. APPENDIX: Sample Paratie Input File Generated by New Software Program

**
* PARATIE ANALYSIS FOR DESIGN SECTION: EC7, 2007: DA-1, Combination 2: A2 + M2 + R1
*1: Define General Calculation Settings
delta 0.2
unit m kN
option param itemax 200

*2. ADD GENERAL WALLS & DIMENSIONS
wall Leftwall 0 182 200

*3.1 DEFINE SURFACE FOR LEFT WALL
soil 0_L Leftwall 182 200 1
soil 0_R Leftwall 182 200 2 180

*4: DEFINE SOIL LAYER ELEVATIONS & STRENGTHS
* BORING Boring 1
  *DATA FOR LAYER: 1, SOIL TYPE= 1, F
    Ldata L1 200
    weight 19 10 10
    Resistance 3 32 0.307 3.255
    atrest 0.47 0.5 1
    Moduli 15000 3 0 110 0.5
    permeabil 0.03048
Endl

*5.1: DEFINE STRUCTURAL MATERIALS
*START GENERAL MATERIALS
  * GENERAL CONCRETE MATERIALS - CONVERTED TO CONSISTENT UNITS WITH FORCE/LENGTH^2
    *Concrete material: 0 Name= 3 ksi Concrete, E= 21541.9MPa
      material CONC_0_3 21541900
    *Concrete material: 1 Name= 4 ksi Concrete, E= 24874.5MPa
      material CONC_1_4 24874500
    *Concrete material: 2 Name= 5 ksi Concrete, E= 27810.5MPa
      material CONC_2_5 27810500
  * GENERAL STEEL MEMBER MATERIALS - CONVERTED TO CONSISTENT UNITS WITH FORCE/LENGTH^2
    *Steel material: 0 Name= Fe510, E= 206000MPa
      material STEEL_0_206000000
    *Steel material: 1 Name= A50, E= 200100MPa
      material STEEL_1_200100000
  * GENERAL REBAR MATERIALS - CONVERTED TO CONSISTENT UNITS WITH FORCE/LENGTH^2, USED FOR ANCHORS
    *Rebar material: 0 Name= Grade 60, E= 200100MPa
      material REB_0_Gr 200100000
    *Rebar material: 1 Name= Grade 75, E= 200100MPa
      material REB_1_Gr 200100000
    *Rebar material: 2 Name= Grade 80, E= 200100MPa
      material REB_2_Gr 200100000
    *Rebar material: 3 Name= Grade 150, E= 200100MPa
      material REB_3_Gr 200100000
    *Rebar material: 4 Name= Strands 270 ksi, E= 200100MPa
      material REB_4_St 200100000
  * USER DEFINED MATERIALS - CONVERTED TO CONSISTENT UNITS WITH FORCE/LENGTH^2, USED FOR ANCHORS
    *User material: 0 Name= User mat 0, E= 1MPa
      material USER_0_U 1000
  * END GENERAL MATERIALS

* 5.2 Define a very stiff material for rigid supports
mate stiffness 100000000000

* 6.1 LEFT WALL STRUCTURAL PROPERTIES
  *Calculate equivalent Steel Sheet Pile Ixx. x Wall Spacing
DeepXcav theory manual:  Developed by Ce.A.S. srl, Italy and Deep Excavation LLC, U.S.A.

* Ewall= 206000 MPa, Stiffness Ixx= 82795.6 cm4/m x 1 m = 82795.6 cm4
* Iequivlant= Ewall x Ixx x ConvEl / (Estandard x ConvEL x Wall Spacing) =>
* Iequivlant= 206000 MPa x 82795.6 cm4/m x 1 m = 82795.6 cm4 x 1E-08 / (206000 x x 1 x 1)= 0.00083  (m^4/m)
* Now calculate Equivalent Wall Thickness from Ixx/Length
* Wall thick= (12 x Ixx/L)^{(1/3)} = (12 x 0.00083)^{(1/3)} = 0.21498 (m)

BEAM Leftwall_BEAM Leftwall 182 200 STEEL_0_0.214979 00 00

*7.1: GENERATE SUPPORTS FOR LEFT WALL
*Convert Tieback to Wire: Stiffness a/L= (Area/ConvA) / [Spacing x (Free Length + Fixed Length x Stiffness Ratio / 100) =>
*Stiffness a/L= (5.94cm2/10000cm2 /m2) /[2m x (7m + 50 x 9 m/100)] = 2.58261E-05

WIRE_0 Leftwall 197 REB_4_St 2.58261E-05 200 30 0 0

*8.1: ADD WALL LOADS & PRESCRIBED CONDITIONS FOR LEFT WALL

NODE 0 Leftwall 195
* END OF NODE ADDITION

* 9.A 1st wall compute external wall surcharges. Stage 0
* 9.A 1st wall compute external wall surcharges. Stage 1
* Stage: 3, examine surcharge load 0 1st point at Elev. 200, x= 0, qx = 5, qz= 0
* 2nd point at Elev. 195, x= 0, qx = 0, qz= 0
* Auto Procedure: Excavation on the right, load is pushing to the right. Load is treated as unfavorable variable load LF=1.3

**** END determination of load factors for strip surcharge load 0

******************************************************************************

*10: GENERATE ALL STEP/STAGES
******************************************************************************

*START DATA FOR STAGE: 0 Name: Stage 0

step 0 : Stage 0

*10.a: DESCRIBE Kp, Ka Changes for this stage due to Defined Wall Friction, Slope or Strength Code Changes

* LAYER 1 Stage 0

*Standard Soil Code Used: EC7, 2007, Case:DA-1, Comb. 2: A2 + M2 + R1
*FS_FR = 1.25, FS_c= 1.25, FS_DriveEarth= 1, FS_pStab= 0.9, FS_RES= 1
*FS_LoadVar = 1.3, FS_LoadPerm= 1, FS_Seismic= 0, FS_ArchPerm= 1.1, FS_ArchTemp= 1.1
*KaH= KaBase x FS_DriveEarth x [Rankine_Kah(deg FR= 26.56, DFR= 0, Asur= 0)] / [Rankine_Kah(deg FR= 32, DFR= 0, Asur= 0)] =>
*KaDH = 0.307 x 1 x 0.382/0.307 = 0.382
*KpDH= [KpHBase /FS_Resist] x [Rankine_Kph(deg FR= 26.56, DFR= 0, Asur= 0)] / [Rankine_Kph(deg FR= 32, DFR= 0, Asur= 0)] =>
*KpDH = [3.255 /1] x 2.618/3.255 = 2.618

* END LAYER 1 Stage : 0
* If Section 10.b is not specified then parameters are same as in previous stage.
*END 10.a

*10b: START GENERATE SOIL PROPERTY CHANGE COMMANDS FOR STAGE
* These changes might be associated with the use of a Strength reduction code such as EUR 7
* or with the user changing from drained to undrained in this stage etc.
change L1 u-frict 26.56 Leftwall
change L1 d-frict 26.56 Leftwall
change L1 u-cohe 2.4 Leftwall
change L1 d-cohe 2.4 Leftwall
change L1 u-ka 0.381719868280688 Leftwall
change L1 d-ka 0.381719868280688 Leftwall
change L1 u-kp 2.61784906429131 Leftwall
change L1 d-kp 2.61784906429131 Leftwall

*10a: END GENERATING CHANGES FOR STAGE.

* DATA FOR LEFT WALL
setwall Leftwall

*10.1 Generate left wall water elevations for stage 0
geom 200 200
water 195 0 -3830

*11: ADD LEFT WALL SUPPORTS

*13.1: ADD LEFT WALL SURCHARGES NOT FROM LOADS DIRECTLY LOADING THE WALL
*13.2.1: ADD LEFT WALL SURCHARGES CALCULATED OUTSIDE FROM PARATIE ENGINE

*13.3: ADD WALL SURCHARGES THAT ARE DIRECTLY ON THE LEFT Wall
*13.3: END ADDING WALL SURCHARGES ON LEFT WALL
* END DATA FOR LEFT WALL

*19.1 EXAMINE IF SUPPORTS ARE REMOVED FOR LEFT WALL
* 19: END SUPPORT REMOVAL

*20: ADD LATERAL LINE LOADS PLACED DIRECTLY ON WALL
ENDSTEP
*END DATA FOR STAGE 0 NAME: Stage 0
******************************************************************

******************************************************************

*START DATA FOR STAGE: 1 Name: Stage 1
step 1 : Stage 1

*10.a: DESCRIBE Kp, Ka Changes for this stage due to Defined Wall Friction, Slope or Strength Code Changes
* LAYER 1 Stage 1
*Standard Soil Code Used: EC7, 2007, Case:DA-1, Comb. 2: A2 + M2 + R1
*FS_Fr = 1.25, FS_c" = 1.25, FS_DriveEarth= 1, FS_gStab= 0.9, FS_RES= 1
*FS_LoadVar = 1.3, FS_LoadPerm= 1, FS_Seismic= 0, FS_ArchPerm= 1.1, FS_ArchTemp= 1.1
* KaH= KaBase x FS_DriveEarth x [Rankine_Kah(deg Fr= 26.56, DFR= 0, Asur= 0)] / [Rankine_Kah(deg Fr= 32, DFR= 0, Asur= 0)]=>
* KaH= 0.307 x 1 x 326/0.307 = 326
* KpH= [KpBase /FS_Resist] x [Rankine_Kph(deg Fr= 26.56, DFR= 0, Asur= 0)] / [Rankine_Kph(deg Fr= 32, DFR= 0, Asur= 0)]=>
* KpH= [3.255 /1] x 2.618/3.255 = 2.618
* KaH= KaBase x FS_Drive x [Rankine_Kah(deg Fr= 26.56, DFR= 0, Asur= 0)] / [Rankine_Kah(deg Fr= 32, DFR= 0, Asur= 0)]=>
* KaH= 0.307 x 1 x 326/0.307 = 326
* KpH= [KpBase /FS_Resist] x [Rankine_Kph(deg Fr= 26.56, DFR= 0, Asur= 0)] / [Rankine_Kph(deg Fr= 32, DFR= 0, Asur= 0)]=>
* KpH= [3.255 /1] x 2.618/3.255 = 2.618
* c'Uphill= c_base / (FS_c x FS_DriveEarth) = 3/(1.25 x 1) = 2.4
* c'Down= c_base / (FS_c x FS_Res) = 3/(1.25 x 1) = 2.4
* END LAYER 1 Stage : 1
* If Section 10.b is not specified then parameters are same as in previous stage.
*END 10.a

* DATA FOR LEFT WALL
setwall Leftwall

*10.1 Generate left wall water elevations for stage 1
gem 200 196.5
water 195 0 -3830

*11: ADD LEFT WALL SUPPORTS

*13.1: ADD LEFT WALL SURCHARGES NOT FROM LOADS DIRECTLY LOADING THE WALL
*13.2.1: ADD LEFT WALL SURCHARGES CALCULATED OUTSIDE FROM PARATIE ENGINE

*13.3: ADD WALL SURCHARGES THAT ARE DIRECTLY ON THE LEFT Wall
*13.3: END ADDING WALL SURCHARGES ON LEFT WALL
* END DATA FOR LEFT WALL

*19.1 EXAMINE IF SUPPORTS ARE REMOVED FOR LEFT WALL
* 19: END SUPPORT REMOVAL

*20: ADD LATERAL LINE LOADS PLACED DIRECTLY ON WALL
ENDSTEP
*END DATA FOR STAGE 1 NAME: Stage 1
******************************************************************

******************************************************************

*START DATA FOR STAGE: 2 Name: Stage 2
step 2 : Stage 2
10.a: DESCRIBE Kp, Ka Changes for this stage due to Defined Wall Friction, Slope or Strength Code Changes
* LAYER 1 Stage 2
* Standard Soil Code Used: EC7, 2007, Case:DA-1, Comb. 2: A2 + M2 + R1
* FS_FR = 1.25, FS_c' = 1.25, FS_DriveEarth = 1, FS_gStab = 0.9, FS_RES = 1
* FS_LoadVar = 1.3, FS_LoadPerm = 1, FS_Seismic = 0, FS_ArchPerm = 1.1, FS_ArchTemp = 1.1
* KpUH = KaBase * FS_DriveEarth * [Rankine_Ka(d) = 26.56, DFR = 0, Asur = 0] / [Rankine_Ka(d) = 32, DFR = 0, Asur = 0]
* KaDH = 0.307 x 1 x 0.382/0.307 = 0.382
* KpDH = [KpHBase / FS_Resist] x [Rankine_Kp(d) = 26.56, DFR = 0, Asur = 0] / [Rankine_Kp(d) = 32, DFR = 0, Asur = 0]
* KpDH = 3.255 / 1 x 2.618/3.255 = 2.618
* KpDH = 0.307 x 3.255/0.307 = 3.255
* KpDH = 0.307 x 3.255/0.307 = 3.255
* KpDH = 3.255 / 1 x 2.618/3.255 = 2.618
* DATA FOR LEFT WALL
setwall Leftwall

10.1 Generate left wall water elevations for stage 2
geom 200 196.5
water 195 0 -3830
* 11: ADD LEFT WALL SUPPORTS
ADD SPL_0

13.1: ADD LEFT WALL SURCHARGES NOT FROM LOADS DIRECTLY LOADING THE WALL
* 13.2.1: ADD LEFT WALL SURCHARGES CALCULATED OUTSIDE FROM PARATIE ENGINE

13.3: ADD WALL SURCHARGES THAT ARE DIRECTLY ON THE LEFT WALL
* 13.3: END ADDING WALL SURCHARGES ON LEFT WALL
* END DATA FOR LEFT WALL

19.1 EXAMINE IF SUPPORTS ARE REMOVED FOR LEFT WALL
* 19: END SUPPORT REMOVAL

20: ADD LATERAL LINE LOADS PLACED DIRECTLY ON WALL
ENDSTEP
* END DATA FOR STAGE 2 NAME: Stage 2

* START DATA FOR STAGE: 3 Name: Stage 3
step 3 : Stage 3

10.a: DESCRIBE Kp, Ka Changes for this stage due to Defined Wall Friction, Slope or Strength Code Changes
* LAYER 1 Stage 3
* Standard Soil Code Used: EC7, 2007, Case:DA-1, Comb. 2: A2 + M2 + R1
* FS_FR = 1.25, FS_c' = 1.25, FS_DriveEarth = 1, FS_gStab = 0.9, FS_RES = 1
* FS_LoadVar = 1.3, FS_LoadPerm = 1, FS_Seismic = 0, FS_ArchPerm = 1.1, FS_ArchTemp = 1.1
* KpUH = KaBase * FS_DriveEarth * [Rankine_Ka(d) = 26.56, DFR = 0, Asur = 0] / [Rankine_Ka(d) = 32, DFR = 0, Asur = 0]
* KaDH = 0.307 x 1 x 0.382/0.307 = 0.382
* KpDH = [KpHBase / FS_Resist] x [Rankine_Kp(d) = 26.56, DFR = 0, Asur = 0] / [Rankine_Kp(d) = 32, DFR = 0, Asur = 0]
* KpDH = 3.255 / 1 x 2.618/3.255 = 2.618
* DATA FOR LEFT WALL
setwall Leftwall

10.1 Generate left wall water elevations for stage 3
geom 200 191
water 195 4

* 11: ADD LEFT WALL SUPPORTS

13.1: ADD LEFT WALL SURCHARGES NOT FROM LOADS DIRECTLY LOADING THE WALL
* 13.2.1: ADD LEFT WALL SURCHARGES CALCULATED OUTSIDE FROM PARATIE ENGINE
*13.3: ADD WALL SURCHARGES THAT ARE DIRECTLY ON THE LEFT WALL
* Stage: 3, examine surcharge load 0 1st point at Elev. 200, x= 0, qx = 5, qz= 0
* 2nd point at Elev. 195, x= 0, qx = 0, qz= 0
* Auto Procedure: Excavation on the right, load is pushing to the right. Load is treated as unfavorable variable load LF=1.3
***** END determination of load factors for strip surcharge load 0

dload step Leftwall 195 0 200 6.5
*13.3: END ADDING WALL SURCHARGES ON LEFT WALL
* END DATA FOR LEFT WALL

*19.1 EXAMINE IF SUPPORTS ARE REMOVED FOR LEFT WALL
* 19: END SUPPORT REMOVAL

*20: ADD LATERAL LINE LOADS PLACED DIRECTLY ON WALL

ENDSTEP
*END DATA FOR STAGE 3 NAME: Stage 3
******************************************************************************

set country english
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