Su from FVT v1.0

User Documentation

GUI Workflow Steps of the tool

Handling of Missing Parameters

System Default Parameter Estimation: Assumptions and Correlations.

Su reduction factors, μ for NC/Slightly OC clays

Theoretical minimum Su

Su at different stress states and recommended application areas

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"Su from FVT" is a desktop based geotechnical application developed using Python, designed to calculate and analyse undrained shear strength, Su from field vane test, FVT results. The system provides automated data extraction, processing, parameter estimation, Su reduction based on different standards, and visualization through a user interactive interface.

Steps of the *Su from FVT* tool

Steps to calculate reduced Su from FVT measurements using the "Su from FVT" tool:

- **Step 1:** Browse "tek" file
- **Step 2:** Extract data from "*tek*" file
- **Step 3.1 - 3.2:** Choose how to handle missing parameters and fill accordingly.
- **Step 4.1 - 4.2:** Select a reduction method and calculate the reduced Su.
- **Step 5.1 - 5.2:** Select what to include on the plots and visualize the plots.
- **Step 6 - 7:** Download the output as a dxf (Reduced Su from FVT) and excel file.

Steps to retrieve calculated reduced Su from SQL server using the "Su from FVT" tool:

- ***Query Step 1:*** *Search for previously calculated data using the test's location id ("X_Y")*
- **Step 5.1 - 5.2:** Select what to include on the plots and visualize the plots.
- **Step 6 - 7:** Download the output as a dxf (contains only the Su^{red} from FVT) and excel file.

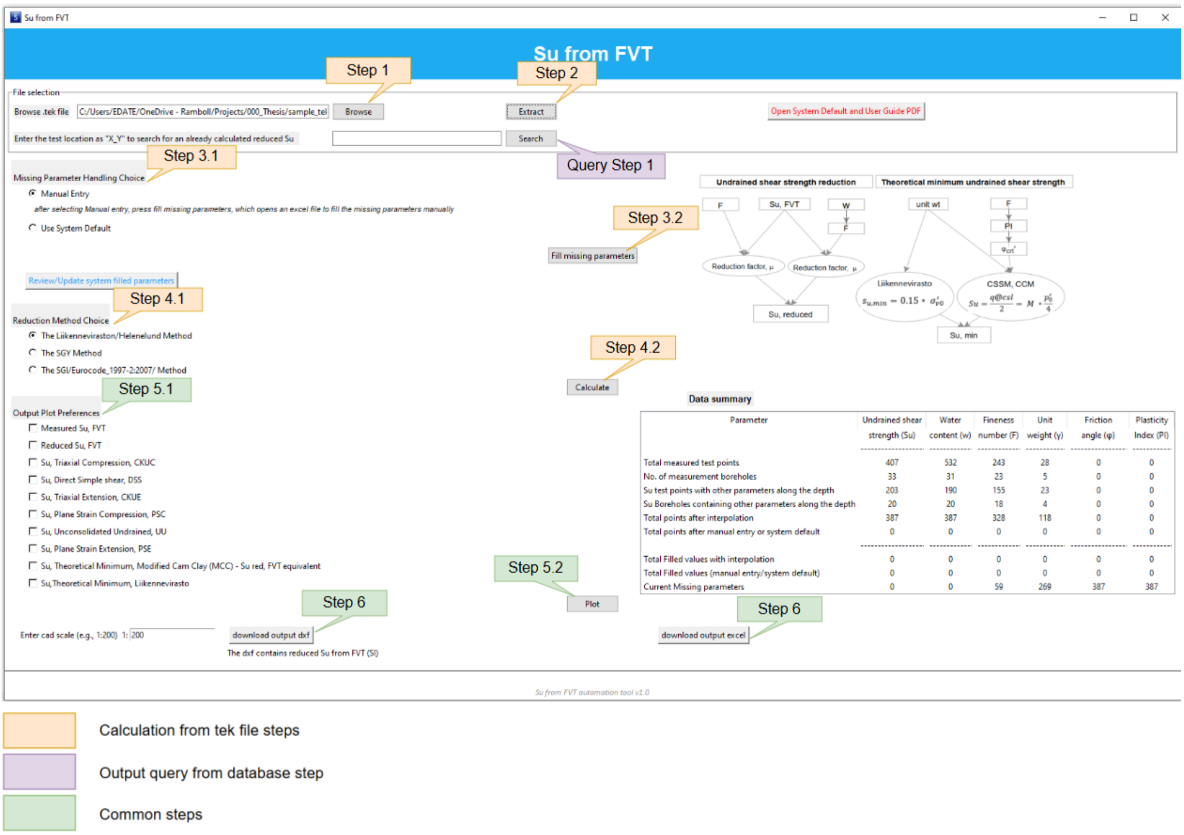


Figure 1. Steps to use the "Su from FVT" tool

An auto updating summary table on the right side of the GUI gives updates of parameter availability, interpolations, missing data and so on to help with rough insight of the data quality.

Integration with Original Soundings:

The measured soundings can be overlayed on top of the reduced Su plots in the downloaded dxf using “piste” option from the “soundings” menu in Civil 3D as shown in Figure 2. The reduced plots can also be used on cross sections by moving the plots with a basepoint.

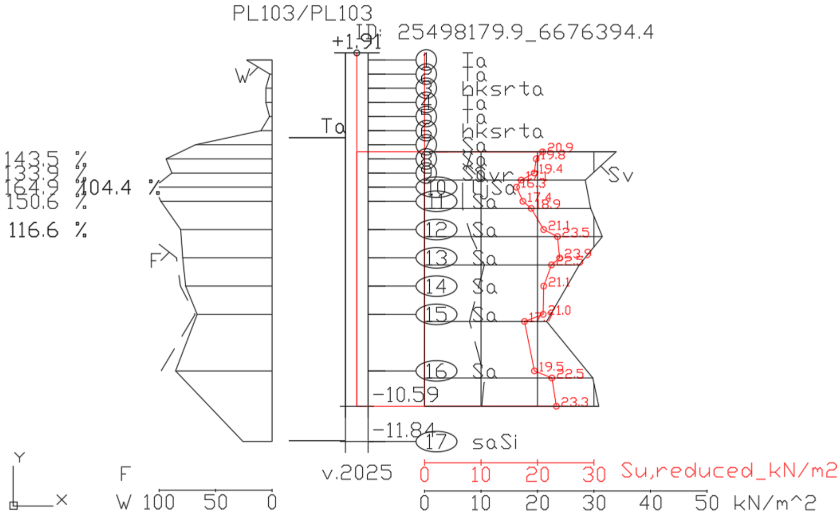


Figure 2. *Sample reduced Su from the system’s dxf output with the original sounding*

Furthermore, the downloaded excel contains a sheet organized as per a typically used soil parameters estimations excel template (Table 1) and can easily be adopted to the template for further processing of the reduced Su and clay layer definition.

Table 1. Sample soil parameter summary table

Maakerros	Vesipitoisuus	Tilavuuspaino	Kitkakulma	Su, redusoitu
	%	kN/m³	kN/m³	kPa
Fill/Täyttö				
Clayey fill/Sata				
Clay 1/Savi 1	91.34	14.83	24.47	24.2
Clay 2/Savi 2	70.94	11.77	25.00	30.2
Clay 3/Savi 3	72.67	11.80	24.98	23.3
Moreeni/Hiekka				

Piste	Syvyys maanpinnasta	Korkeustaso	Maalaji	Humuspitoisuus	W	F	**Su measured**	red factor	**Su, red**	**Su, gaps interpolated**	Unit wt	Friction angle
	m	m		%	%	%	kPa		kPa	kPa	kN/m3	deg
PL108	4.0	-1.8	Sa 1		89.80	153.60	53.40	0.59	31.59	53.40	14.89	24.59
	4.3	-2.1	Sa 1		89.80	153.60		0.59	30.01	50.73	14.89	24.59
	4.8	-2.6	Sa 1		87.90	141.50		0.62	29.86	48.07	14.96	24.75
	5.0	-2.8	Sa 1		87.85	139.20	45.40	0.63	28.47	45.40	14.96	24.75
	5.8	-3.6	Sa 1		87.80	136.90		0.63	27.07	42.75	14.96	24.76
	6.0	-3.8	Sa 1		87.85	140.50	40.10	0.62	25.01	40.10	14.96	24.75
	6.8	-4.6	Sa 2		87.9	144.1		0.6	25.7	41.8	15.0	24.7
	7.0	-4.8	Sa 2		87.8	143.1	43.4	0.6	26.8	43.4	15.0	24.8
	7.3	-5.1	Sa 2		87.7	142.1		0.6	27.7	44.7	15.0	24.8
	8.75	-6.55	Sa 2		59.10	108.10		0.72	33.21	46.07	16.44	25.00
	9.00	-6.80	Sa 2		59.10	108.10	47.40	0.72	34.17	47.40	16.44	25.00
	9.30	-7.10	Sa 2		59.10	108.10	39.29	0.72	28.32	39.29	16.44	25.00

1 Handling of Missing Parameters

To address missing parameter issues the system incorporates the following logics for estimating missing parameters.

- If there is FVT at that location but no sampling is taken, then the system automatically rejects the FVT at that location, as there is no further information about the soil property.
- If any parameter is missing at a specific depth but values are available at least at three depths of the same test location, the missing values are automatically interpolated or projected by default, and a notification will be shown to inform the user that missing values have been filled by linear interpolation.
- If there is at least water content but other parameters are still missing after the interpolation, the user will be presented with two options of filling out the missing parameters:
    - Manual Entry
    - System Default Parameter Estimation

Manual Entry

If the manual option of fill missing values is chosen, a new window will open to display the extracted data with the missing parameters (Fineness number, unit weights, friction angle, and PI) for the user to manually feed the nan values. Here reasonable estimation of the fineness number is the significant part, as it will highly affect the reduced Su. To assist this process, the interpolated and correlated values are presented adjacent to the nan values to be used as a reference.

Since neither friction angle no PI are usually available, the friction angle is set to a default value of 21 and the PI is calculated with correlation to simplify manual data entry, but this default values can still be changed as needed.

System Default Parameter Estimation: Assumptions and Correlations

The system default parameter estimation is based on empirical correlations and formulas with reasonable assumptions for each input parameters. It should be noted that the **correlations developed for the estimation of the Fineness number (F) and Plasticity index (PI) is based on databases containing mainly inorganic clays.** ***Therefore, care should be taken when applying these correlations to organic clays, peat, or other clay soil types.***

1.1 Unit weight, γ

If unit weight data, VG is unavailable for the entire depth or in less than three depths, and the user choses the system's default estimation method, the unit weight is estimated using Eq. (1) based on the measured water content**, w** and these assumptions:

- Specific gravity (SG) **= 2.65,** a typical value for Finnish clays (Ronkainen, 2012)
- Soil is fully saturated **(S= 100%)**

$$\gamma = \frac{(1+\omega)*\gamma_\omega}{\left(\frac{1}{G_s}+\omega\right)} = \frac{(100+\omega(\%))*10}{(37.7+\omega(\%))} \quad (1)$$

- A cutoff range of **10** to **22** is applied to minimize the impact of extremely high or low outliers in the data. These values are the corresponding minimum and maximum water content values from Finnish soil databases FI-CLAY/14/856 (Löfman & Korkiala-Tanttu, 2022), and F-CLAY/7/216 (D'Ignazio et al., 2016). The ranges from these databases are confirmed to fall within the threshold values presented in Suomen maalajien ominaisuuksia (Ronkainen, 2012) While estimating missing parameters, more images similar to Figure 3, for different types of clays, can be found in (Ronkainen, 2012).

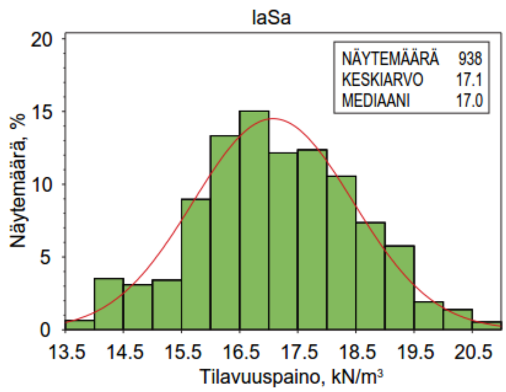
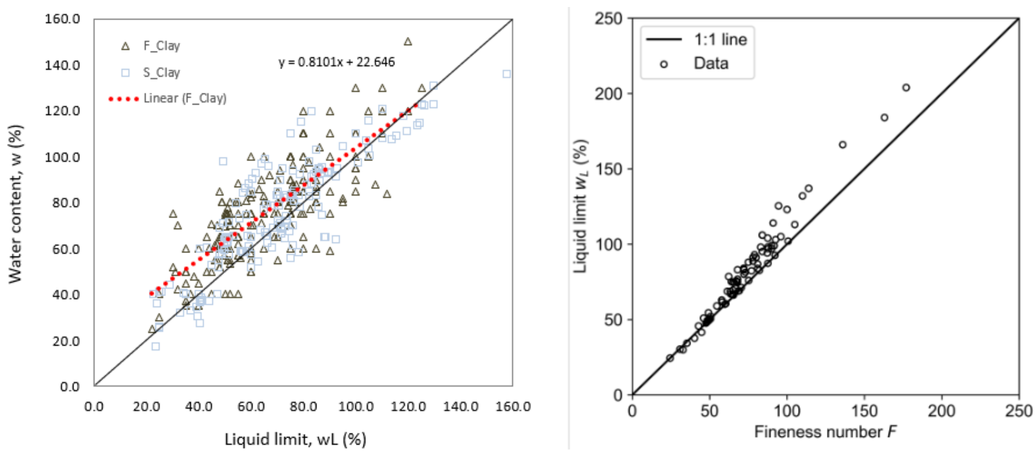


Figure 3. Unit weight range in Finnish soils (Ronkainen, 2012)

1.2 Fineness number/Liquid limit, F

If there is no fineness number (F) in the entire depth, but there is a water content available, then the Fineness number which can fairly be represented by the liquid limit (Löfman & Korkiala-Tanttu, 2022) as shown in Figure 4b, is estimated from an empirical correlation (Eq. (2)) developed based on a Finnish database F-CLAY/10/173 (D'Ignazio et al., 2016) as shown in Figure 4a.

$$wL = 0.7543\ w + 8.6974 \quad (2)$$



(a) (b)

Figure 4. (a) Liquid limit (wL) Vs Water content (w) for F-CLAY/7/216 and S-CLAY/7/168 (b) Fineness number (F) Vs Liquid limit (wL) for FI-CLAY/14/856

- A cutoff range of **22** to **200** is applied considering the minimum and maximum wL or F values in the Finnish databases FI-CLAY/14/856, and F-CLAY/7/216 to minimize the impact of extremely high or low outliers in the data. The ranges from these databases are confirmed to fall within the threshold values presented in Suomen maalajien ominaisuuksia (Ronkainen, 2012). It should be noted that the databases used for these correlations are mainly from inorganic clay soils and thus may not a best prediction for other clay types. For reference while estimating missing parameters, additional images similar to Figure 5 for different clay types, can be found in (Ronkainen, 2012).

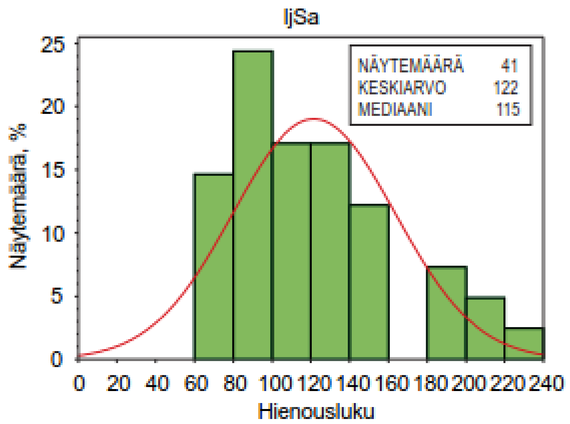


Figure 5. Fineness number range in Finnish soils (Ronkainen, 2012)

1.3 Friction angle, φ'

The system uses correlations between friction angle at critical state and Plasticity index (PI), where the PI is estimated from the LL using Eq. (4). The friction angle of clays at critical state is estimated from PI as shown in Figure 6 using Eq. (3) proposed by Mitchell (1976), cited in (Kulhawy & Mayne, 1990)):

$$sin(\varphi'_{cv}) = 0.8 - 0.094\ ln(PI) \tag{3}$$

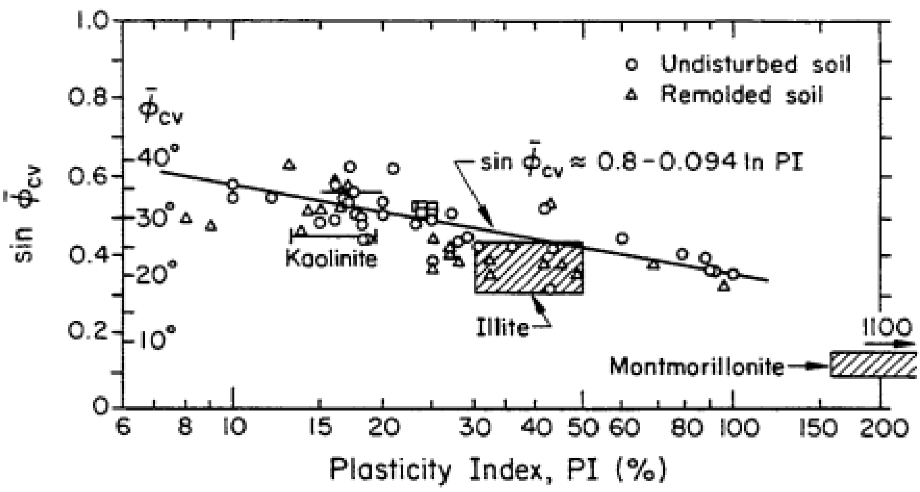


Figure 6. φ'_{CV} for NC Clays Vs PI (source: Mitchell, 1967)

Meanwhile the PI can be estimated from the wL or Fineness number using a correlation developed from Finnish soil database FI-CLAY/14/856 as shown in Figure 7.

$$PI = 0.8343\ wL - 15.531 \tag{4}$$

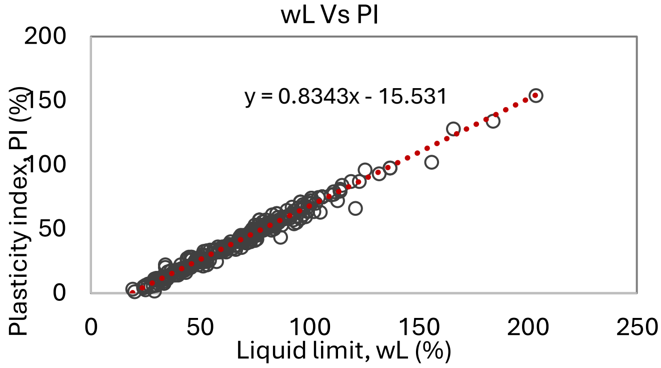


Figure 7. Liquid limit, wL Vs Plasticity index, PI _ FI-CLAY/14/856

- A cutoff range of **18** to **25** is applied to minimize the impact of extremely high or low outliers in the data, by refereeing friction angle boundaries presented on (Mitchell & Soga, 2005) and lowest boundary value by (D'Ignazio et al., 2016). Additionally, typical values of friction angle at critical state for London clay, Weald clay, and kaolin (Table 2) is given by (Schofield and Wroth, 1968), cited on (Bardet, 1997).

Table 2. Critical state parameters and plasticity indices for London clay, Weald clay, and kaolin (Schofield and Wroth, 1968)

Material constatns	London clay	Weald clay	Kaolin
Slope of critical state line (CSL), λ	0.161	0.093	0.260
Void ratio on CSL at 1 kPa, Γ	1.759	1.060	2.767
Slope of critical state in p'–q space, M	0.88	0.95	1.02
Friction angle ϕ' at critical state (deg)	22.6	24.2	25.8
Slope of swelling line, κ	0.062	0.035	0.050
Parameter $\Lambda = (\lambda - \kappa)/\lambda$	0.615	0.624	0.808
Liquid limit LL (%)	78	43	74
Plastic limit PL (%)	26	18	42
Plasticity index PI (%)	52	25	32
Specific gravity G_s	2.75	2.75	2.61
Void ratio at LL	2.145	1.183	1.931
Void ratio at PL	0.715	0.495	1.096

In general, the parameters link to the Su can be summarized as shown in Figure 8.

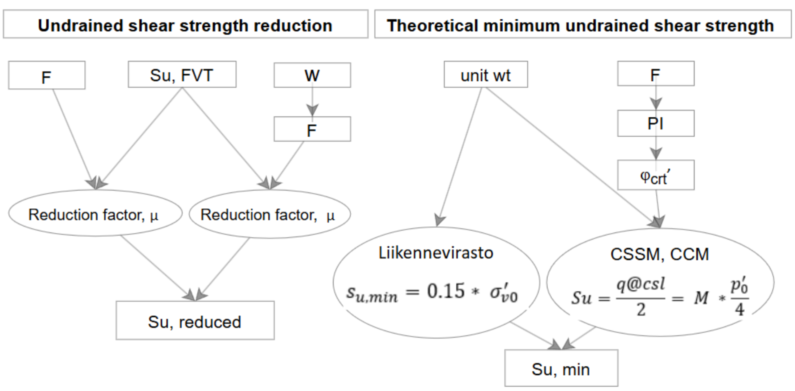


Figure 8. Input parameters link to Su

2 Su reduction factors, μ for NC/Slightly OC clays

The peak undrained shear resistance of the vane test is corrected as per Eq. (5) to determine the undrained shear strength for geotechnical analysis (ASTM, 2018).

$$s_u^{red} = \mu * s_u^{FVT} \tag{5}$$

Where: μ - correction factor

S_u^{FVT} - uncorrected (measured) shear strength from Field Vane

Su^{red} - the reduced undrained shear strength.

There are different approaches to calculate the reduction factor. The approaches can be classified based on either the soil is normally consolidated (slightly over consolidated) or over consolidated.

The method for Su reduction of NC or slightly OC soils is based on the Liquid Limit (W_L), Plasticity Index (I_p) or Fineness number (F). The W_L is the water content at which the soil changes from liquid to plastic state, while the I_P is the water content at which the soil changes from plastic to semi solid state both measured in laboratory with either Casagrande method or fall cone test method.
In Finland, The Liquid limit from fall cone test one point method i.e., the Fineness number, F is calculated using Eq. (6) and typically used to reduce Su values from FVT.

$$F = a * w_i \tag{6}$$

where:

F = Fineness number (%), the Liquid limit using one point fall cone method.

w_i = Water content corresponding to the penetration of the used cone in mm

a = Coefficient, which depends on the weight of the cone

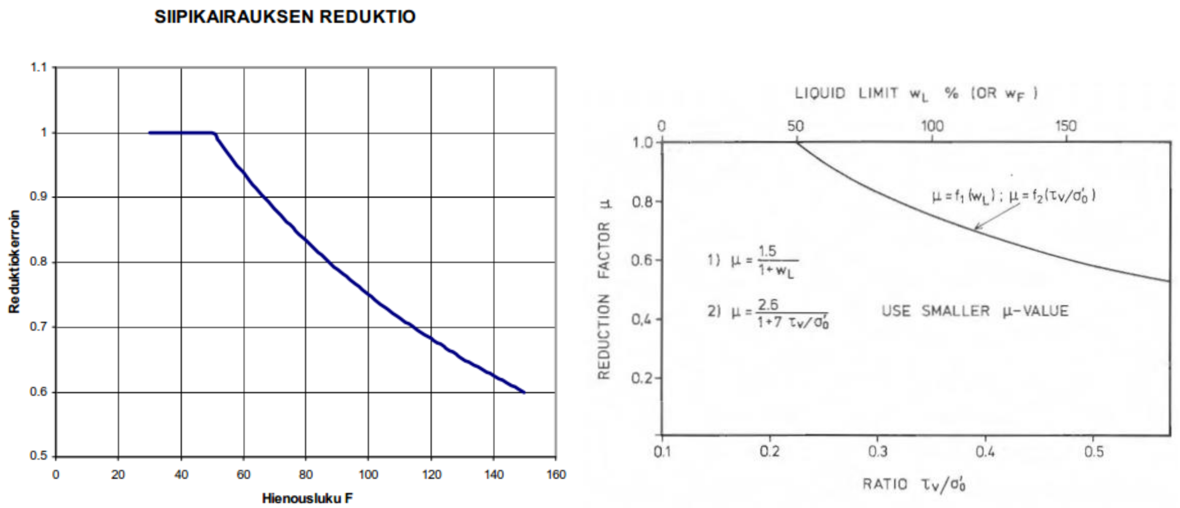
As per Monica et al. The liquid limit from Casagrande method and the Fineness number from one point fall cone test method show close agreement to each other with the liquid limit being slightly higher (Löfman & Korkiala-Tanttu, 2022). In general, for the liquid limit, the fall cone method should be preferred to the Casagrande method. The fall cone method gives more reliable results particularly for low plasticity soil (Eurocode 7 - Part 2, 2007).

The automation tool includes three Su reduction factors based on different sources; the Liikenneviraston/Helenlund method following penkereiden stabiliteetin laskentaohje, Liikenneviraston ohjeita 14/2018 (initially given by K.V. Helenelund), the SGY Method which is based on the Suomen Geoteknillinen Yhdistys (Finnish Geotechnical Society) guideline, and the Eurocode_1997-2:2007_Annex I/ SGI Method from the Swedish Geotechnical Institute's guideline.

2.1 The Liikenneviraston/ Helenelund Method

This method is adopted from Penkereiden stabiliteetin laskentaohje, Liikenneviraston ohjeita 14/2018 which was also initially given by K.V. Helenelund Eq. (7) (Liikennevirasto, 2018), (Kari V. Helenelund., 1977).

$$\mu = \frac{1.5}{1 + w_L} \quad and \quad \mu = 1, for\ w_L < 50 \quad \Big| \quad \mu = \frac{1 \cdot 5}{1 + \frac{F}{100}} \quad and \quad \mu = 1, for\ F < 50 \qquad (7)$$



(a) (b)

Figure 9. Su reduction factors (a) Liikenneviraston (b) Helenelund

2.2 **The SGY Method** (SGY, 1999)

As per SGY, Su based on field vane test for high plasticity soils is over estimated, while Su for lower plasticity soils is underestimated. therefore, correction is needed accordingly. It suggests the undrained shear strength of the soil is not considered to be greater than the shear strength measured from the vane shear test, therefore the limit of correction factor is 1 Eq. (8).

$$\mu = \left(\frac{0.43}{w_L}\right)^{0.45} \geq\ 0.5, \quad and \quad \mu = 1, for\ w_L < 43 \qquad (8)$$

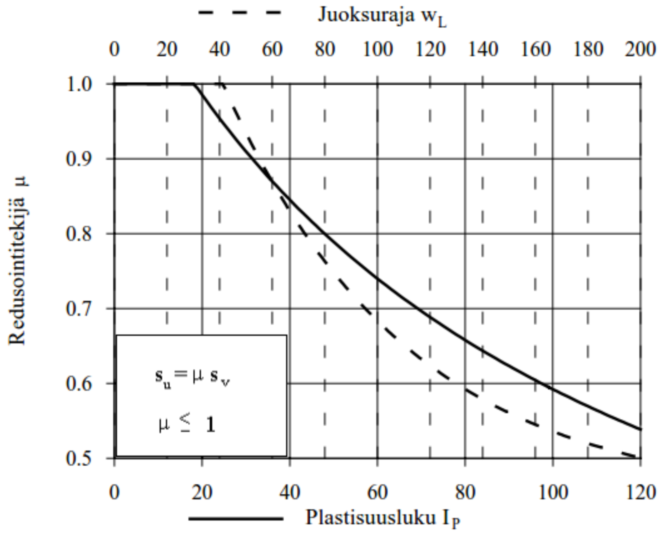


Figure 10. Su reduction factor SGY Method, (SGY, 1999)

2.3 The SGI /Eurocode_1997-2:2007/ Method

The SGI or Eurocode_1997-2:2007_Annex I methods mainly refer Larson's formula for reduction of undrained shear strength from field vane test. (Eurocode 7 - Part 2, 2007) (Larsson et al., 1984). The SGY and SGI/Eurocode methods share similar formula differing mainly in their cutoff values (Eq. (9)).

$$\mu = \left(\frac{0.43}{w_L}\right)^{0.45} \geq 0.5, \quad and \quad \mu = 1.2, for\ w_L < 29 \tag{9}$$

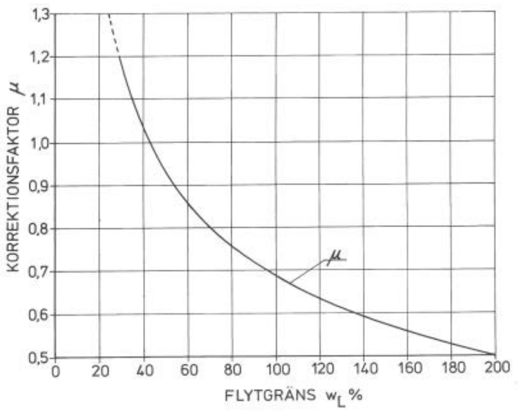


Figure 11. Su reduction factor SGI/Eurocode _1997-2:2007 Method, (Larsson et al., 1984)

3 Theoretical minimum Su

Theoretically, the minimum undrained shear strength mobilized by the soil under undrained loading can be estimated as the strength just before it reaches to a state of continuous deformation without further change in stress (critical state).
The system calculates theoretical minimum undrained shear strength based on two approaches:

- using Cam Clay Model, CCM which is a Critical State Soil Mechanics model (CSSM), and
- using the correlation between minimum Su and effective stress given by Liikennevirasto for a qualitative assessment of FVT (Liikennevirasto, 2018).

3.1 Minimum Su from the modified cam clay model – MCC

$$q = Mp \tag{10}$$

$$v = \Gamma - \lambda \ln p' \tag{11}$$

Where, M - slope of the CSL
V - specific volume, 1 + e
Γ is the specific volume of the soil at a reference stress
λ is the slope of the CSL, which is also the slope of the virgin consolidation curve.
p' - mean effective pressure
q - deviatoric stress

The yield function (Eq. (12)) of MCC model consists of: initial isotropic stress, p_o' which controls the expansion of the yield surface; slope of the CSL, M; the mean effective stress, p; and the deviatoric stress, q(Munda et al., 2014; Rocscience, 2022).

$$f = q^2 - M^2 p'(p_o' - p') = 0 \tag{12}$$

$$\mathrm{M} = \frac{6 \sin \varphi}{3 - \sin \varphi} \tag{13}$$

The coefficient of earth pressure at rest can be calculated using Jaky's formula (Jaky, 1944).

$$k_0 = 1 - \sin \varphi \tag{14}$$

Assumption:

Since oedometer tests to estimate λ and k, are usually not available, It is assumed that there is no volume change moving from the initial stress state to the critical state (from ko line to the CSL) Hence, k = 0 as shown in Figure 12.

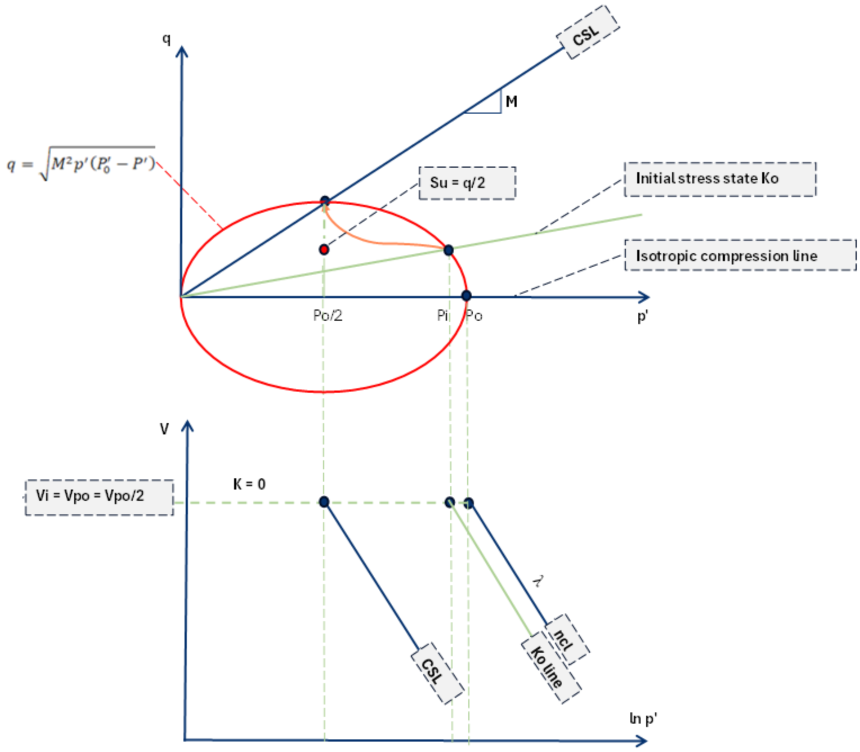


Figure 12. MCC with slope of unloading curve, K = 0

The effective mean stress and deviatoric stress at the CSL considering the earlier assumption will be simplified to Eq. (15) and (16) respectively.

$$p'_{\mathrm{CSL}} = \frac{p_0'}{2} \tag{15}$$

$$q_{CSL} = \sqrt{M^2\, p'_{\mathrm{CSL}}\, (p_0' - p'_{\mathrm{CSL}})} = M * \frac{p_0'}{2} \tag{16}$$

Once the deviatoric stress at CSL is found using Eq. (15), the undrained shear strength, Su can be estimated using Eq. (17).

$$Su = \frac{q@csl}{2} = M * \frac{p_0'}{4} \tag{17}$$

3.2 Minimum Su from Liikennevirasto guideline

The Finnish Transport Agency's guideline 14/2018 on its calculation instructions for the stability of embankments (Liikennevirasto, 2018) considers the minimum undrained shear strength for qualitative assessment using Eq. (18).

$$s_{u,min} = 0.15 * \sigma'_{v0} \tag{18}$$

where: σ'_{vo} = Effective vertical stress, $s_{u,min}$ = Theoretical minimum Su

4 Su at different stress states and recommended application areas

Unlike fundamental material properties such as effective friction angle, specific gravity or mineral composition, undrained shear strength is a conditional measure that is highly dependent on the initial stress state, mode of testing, drainage condition, sample disturbance, rate of loading, amount of confining stress, and other variables (Bardet, 1997; Abed & Korkiala-Tanttu, 2018). Hence, Su may not reflect an inherent property of the soil alone, but rather the soil's response under a specific undrained loading condition. The dependence on the stress state and loading history in addition to poor test execution or sample disturbances makes the accurate estimation of Su more challenging (Ladd et al., 1977).

With thorough detail, (Mayne, 1985) investigated the difference in undrained shear strengths of clay soils under anisotropicaly consolidated undrained, CAU (field conditions) versus isotropicaly consolidated undrained, CIU (laboratory conditions) stress states. he indicated that the anisotropic undrained strength in triaxial compression (CAUC) is typically around 87% of the isotropic strength (CIUC) which is due to the over representation of the confining pressure in the case of the isotropical consolidation. Similarly, for isotropic extension, a correction factor of 0.60 (60%) is suggested to account for stress anisotropy. (Mayne, 2007) also mentioned that Su^{DSS} can simply be estimated as 20% of the preconsolidation stress which is based on reduced shear vane data back calculated from failures of slopes and footings in soft clays.

The conventional practice of projects involving stability of slopes doesn't consider the effect of rotation of the principal stress along the potential slip surface (failure surface). Yet the direction of the major principal stress is almost vertical at the top of the slope, which is called the active side, and it is nearly horizontal around the toe, which is called the passive side, and at an angle in between due to the rotation of stress field along the failure surface (Lo K. Y., 1965). For normally consolidated clays, passive strength is typically the lowest, DSS lies in the intermediate range and active strength is the highest as shown in Figure 13.

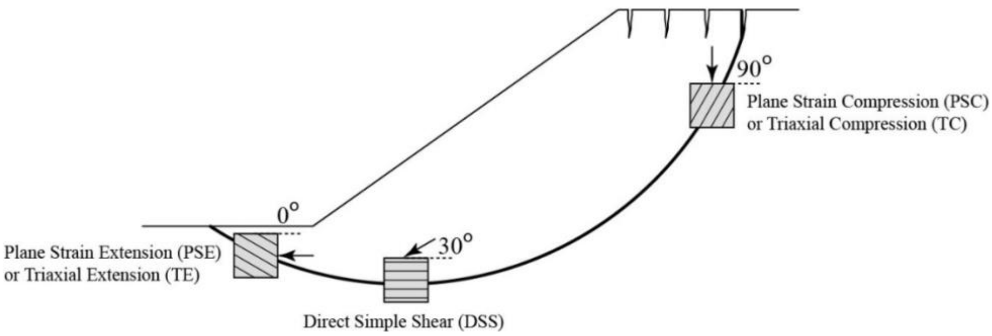


Figure 13. Stress states and direction of major principal stress along failure slip surface, (Abed & Korkiala-Tanttu, 2018)

The correlation of undrained shear strength for different stress paths is established in the program based on a review of literatures (Bjerrum L. & Aitchison G. D., 1973; Ladd et al., 1977; Kulhawy & Mayne, 1990; Mayne et al., 2009; Ching & Phoon, 2013; D'Ignazio et al., 2021). Hence, the program logic follows the correlations summarized in Table 3.

*Table 3. Su of NC Clays Normalized with σ'_{vo} and Su^{FVT} for Different Tests, (Mayne et al., 2009)**

Types of Tests	**S** (Su/σ'_{vo})	**Su/Su^{FVT}**
Triaxial Compression (CKUC)	0.33	**1.57**
Field Vane Shear Test (FVT)	0.21	**1.00**
Direct Simple Shear (DSS)	0.2	**0.95**
Triaxial Extension (CKUE)	0.16	**0.76**
Plane Strain Compression (PSC)	0.34	**1.62**
Unconsolidated Undrained (UU)	0.28	**1.31**
Plane Strain Extension (PSE)	0.19	**0.90**

* After (Ladd et al., 1977) and (Bjerrum L. & Aitchison G. D., 1973), NC Boston blue clay

Where:

CKUC Anisotropixcally (K_0) consolidated undrained triaxial compression
Sample is first consolidated under K_0 stress ($\sigma'_1 = K_0 * \sigma'_3$), then axially compressed without allowing drainage.

CKUE Anisotropixcally (K_0) consolidated undrained triaxial extension
Sample is first consolidated under K_0 stress ($\sigma'_1 = K_0 * \sigma'_3$), then axial stress is lowered without allowing drainage.

CIUC Isotropically consolidated undrained compression
Sample is first consolidated under equal all-round stress (σ'_3), then axially compressed to failure without allowing drainage.

CIUE Isotropically consolidated undrained extension
Sample is first consolidated under equal all-round stress (σ'_3), then lateral stress is increased (σ'_3) without allowing drainage.

FVT Field Vane Test
In-situ method for determining Su by inserting a four-bladed vane into the soil and rotating it at a controlled rate until failure occurs.

DSS Direct Simple Shear
Cylindrical sample is first consolidated under an initial vertical load, then it is sheared horizontally at constant vertical stress.

PSC Plane Strain Compression
A laterally confined (lateral strain in one horizontal direction is prevented) sample is compressed vertically. Because one direction is restrained, the soil mobilizes higher shear strength than CKUC.

PSE Plane Strain Extension
The specimen is extended axially (horizontal pressure increased or vertical pressure reduced) under plane-strain constraint.

UU Unconsolidated Undrained
The specimen is axially loaded to failure without prior consolidation step, giving a quick total-stress.

Geotechnical engineers are ideally expected to choose test conditions which can replicate the natural (in-situ) condition as close as possible (Lambe, 1997), with possible adjustments when the actual condition is different from the test condition (Taylor, 1948). Figure 14 illustrates how loading, unloading and increment of pore water pressure affects the stability of slopes by showing how average effective stress in soil changes during slope failure.

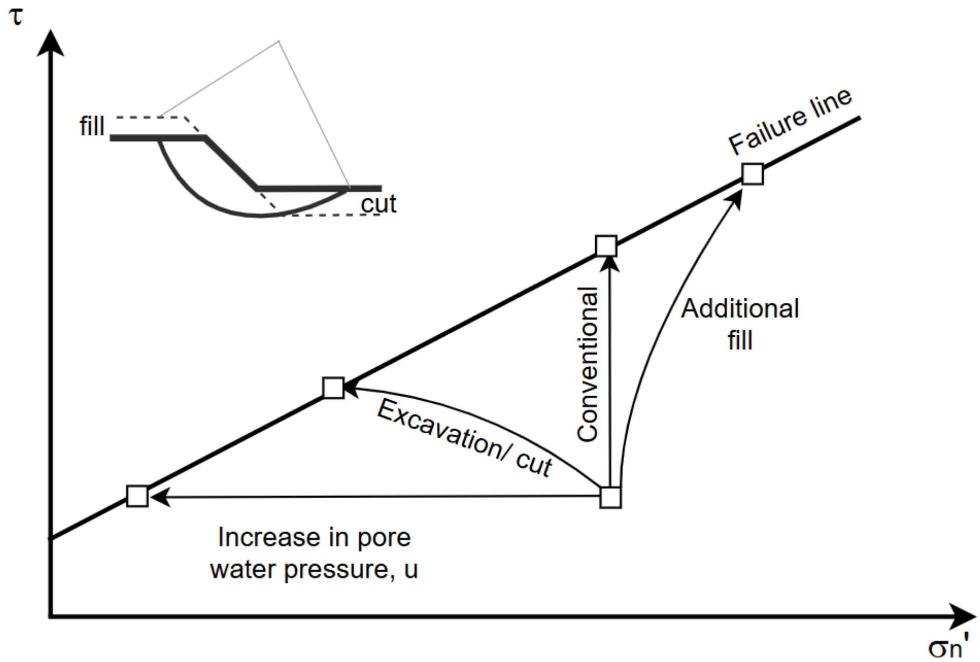


Figure 14. Stress paths to failure of an earth slope, after (Lambe, 1997)

Excavation/cut reduces the normal stress and reduces the soil's shear strength while additional fill/embankment would increase the normal stress which in return increases the shear strength. Hence understanding this allows engineers to choose a a test condition based on the field average effective stress path (compression, extension, pure shear) for that location avoiding both over and under estimation (Lambe, 1997). However, this effect of anisotropy on stability conditions is more significant for flatter slops compared to steep slopes (Lo K. Y., 1965).

The geotechnical manual by the New York state department of transportation summarizes the relevance of laboratory strength tests for actual field conditions as shown in Figure 7 (NYSDOT, 2013).

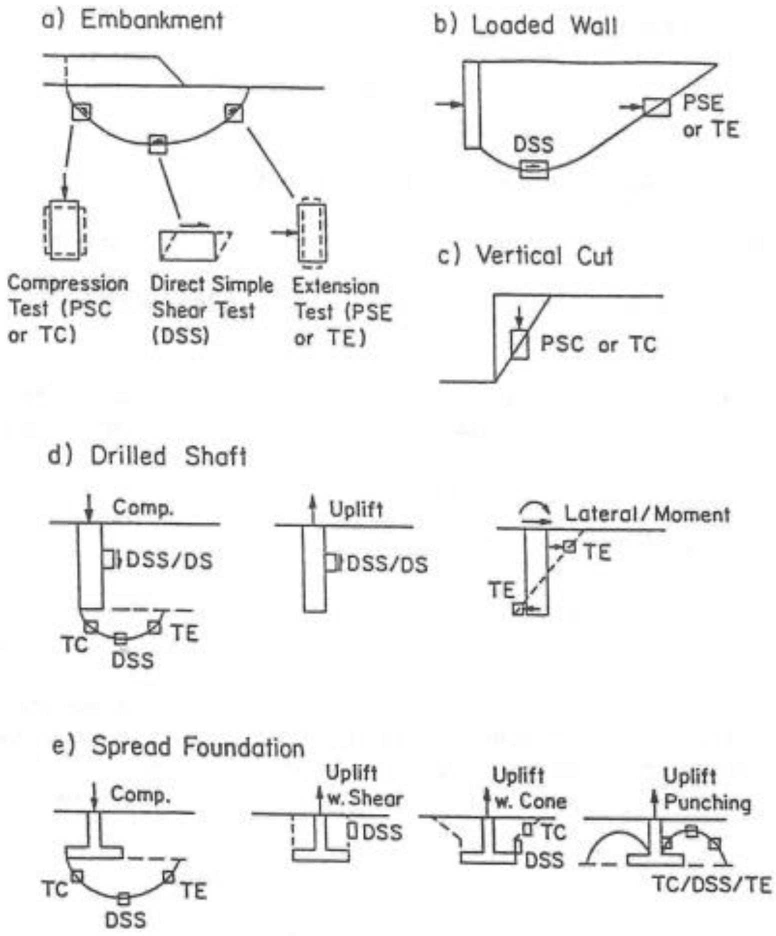


Figure 15. Laboratory strength test representation of field conditions (NYSDOT, 2013)

Recommended test types which can fairly represent the actual field condition for specific types of applications are summarized in Table 4.

Table 4. Summary of recommended test types for specific applications

Test types	Typical application	Remark
Direct Simple Shear, DSS Field vane test, FVT Or average of CKUC & CKUE, if DSS & FVT are not available.	Stability problems	(Mayne, 2007; Abed & Korkiala-Tanttu, 2018)
Triaxial Compression, CKUC	Bearing capacity estimation of deep foundations	Where the compression load dominates.
Unconsolidated Undrained, UU	Short term calculations, construction stage	Fast loading, no drainage & consolidation
Triaxial Extension, CKUE	Unloading conditions, slope toes.	
Plane Strain Extension, PSE Plane Strain Compression, PSC	Long features	(NYSDOT, 2013)

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