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WHITE PAPER

SOIL MODULUS FOR ONSHORE WIND FOUNDATION DESIGN

SUMMARY

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WHITE PAPER

This document is a summary of a more in-depth document of the same title. Paragraph references in the text of this document refer to the original, wider document.

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1. INTRODUCTION

Wind turbine size and power are growing yearly, and loads on wind turbine foundations have been rapidly growing and approximately doubling in the last 8 years. Thus geotechnical design methods used in foundation design should also improve to provide an optimal foundation solution in terms of safety and cost.

The state-of-the-art geotechnical design method is non-linear 3D soil numerical modeling with different mathematical models for different soil types and problems to solve.

The Hardening soil small strain (HSS) mathematical model is one of the best for wind turbine foundations and most soils in terms of accuracy and safety.

Any numerical soil modeling is as accurate as the geotechnical parameters selected and used in the calculations. The geotechnical parameters shall be selected accurately and fit the design problem to design a safe and cost-effective foundation model. The variability of soils often makes geotechnical design and parameter selection challenging.

The Hardening soil small strain model requires many soil parameters for each subsoil layer, and the full list is given in Table 1-1. Many parameters are well-known and easy to measure or derive from the literature, but some are less known and require more testing or research. Table 1-1 has two parts – primary and secondary. Without the primary part, the soil modeling won't be accurate enough, while the secondary part is also needed for modeling; it can be derived from the primary part and literature. Additional notes to the main parameters are given in Table 1-2.

This report presents a comprehensive study on soil modulus for different soil types, ranging from granular (sand, coarse silt, moraine) to cohesive (fine silt, clay) soils. The aim is to provide theoretical and practical guidelines for practitioners to estimate soil modulus for onshore wind turbine foundation design. The focus will be on Young's modulus E and secant modulus for primary loading at 50% of the triaxial failure load E_{50}, secant unloading/reloading modulus E_{ur}, $1D$ constrained (oedometric) modulus E_{oed} and initial tangent modulus at small strain G_{max} or G_0.

The determination of soil modulus is not straightforward, as it depends, among others, on several factors, including state factors (particle density, water content, stress history, cementation) and loading factors (stresses and confinement, strain level, rate effects, number of cycles, drainage). Furthermore, an extensive set of laboratory and in-situ tests would be required to appropriately characterize soil layers across a site.

Multi-layer non-linear soil behavior can be modeled more accurately by non-linear soil material constitutive models such as hardening soil with small strain stiffness compared to linear spring models. Linear spring models cannot accurately take into account stiffness dependency on strain-stress state, backfill effect soil push-out and other important factors, that can lead to less accurate and competitive design, sometimes even wrong design that may lead to problems during wind turbine working life-time.

The study will further investigate existing correlations between measurements from in-situ methods (e.g. CPT, SPT, seismic measurements) and design parameters. Moreover, correlations between soil modulus and other laboratory geotechnical parameters will be investigated. Direct measurements of properties by oedometric or triaxial tests are omitted in this report as they are well-defined in the literature. Direct measurements are usually the best option rather than correlations.

Values of modulus for different soil types are often presented in handbooks without sufficient background information and explanation. This study aims to clarify the differences between the different modulus in the literature and provide recommendations for selecting soil modulus for Finnish soil conditions from laboratory and in-situ tests and literature.

Chapters 2 to 6 explain the theory for different deformation modulus and the factors influencing those. Chapter 7 overviews different site investigation techniques and correlations with different deformation modulus. Chapter 8 is specific to Finland, gives literature values according to NCCI 7, and is most relevant for Finnish soil conditions. And chapter 9 summarizes parameter determinations in the flowchart.

Table 1-1 Parameters for non-linear HSS model.

Parameter	Symbol	Dimension
Primary parameters		
Soil type	-	-
Layer thickness	-	m
Specific weight, no buoyancy	γ_d	kN/m^3
Specific weight, incl. buoyancy	γ_w	kN/m^3
Drained secant modulus together with reference confinement pressure	E_{50}^{ref}	MPa
	p^{ref}	kPa
Stiffness modulus together with reference confinement pressure	E_{ur}^{ref}	MPa
	p^{ref}	kPa
Critical friction angle	φ	deg
Poisson's ratio	ν_{ur}	-
Cohesion	c	kPa
Secondary parameters		
Dilatancy angle	ψ	deg
Power	m	-
Oedometric tangent modulus, together with the reference pressure	E_{oed}^{ref}	MPa
	p^{ref}	kPa
K_0 coefficient	K_0^{NC}	-
(if relevant) Pre-Overburden Pressure	POP	kPa
(if relevant) Over-Consolidation Ratio	OCR	-
Initial Young modulus or initial shear modulus at reference stress	E_0^{ref} or G_0^{ref}	GPa
	p^{ref}	kPa
Threshold shear strain	$\gamma_{0.7}$	-

Table 1-2 Additional notes to parameters.

Symbol	Notes
E_{50}^{ref}	EN 1997-2 specifies the secant modulus E_{50}. It can be measured by drained triaxial test directly or derived by methods explained in the report.
E_{ur}^{ref}	It can be tested by triaxial test directly or derived by methods explained in the report.
E_{oed}^{ref}	Oedometric modulus can be measured by an oedometric test or derived from methods explained in the report.
c	In HSS modeling usually assumed 0 even for cohesive soils for drained conditions
ν	In WT foundation, HSS modeling usually assumed 0.2 even for cohesive soils
ψ	Usually assumed 0 or friction angle minus 30 for granular soils
m	Power exponent is usually assumed 0.4-0.5 and can be measured by triaxial testing at different confinement pressures. Not to mix with modulus number.

2. SOIL MODULUS DETERMINATION FOR FINNISH SOIL CONDITIONS

As discussed in the previous chapters, a stress-strain relationship of soils is non-linear and, hence, soil modulus is not constant, but it depends on several factors (see Chapter 3). Soil modulus, including e.g., Young's modulus E, shear modulus G and constrained modulus M can be determined from both laboratory and in-situ testing. Laboratory testing (e.g. triaxial testing) provides a full description of the stress-strain behavior, and stress or strain increments can be designed to obtain the parameters for the stress/strain range of interest. However, these are not always available, especially in small-sized projects. Furthermore, soil specimens may suffer from sample disturbance and may provide unreliable results. In-situ testing has the advantage of testing the soil in in-situ conditions. However, measurements require calibration from laboratory testing, meaning that the existing correlations potentially underlie all the uncertainties associated with retrieving and preparing soil specimens.

2.1. Coarse-grained soils

In Finland, Swedish Weight Sounding (painokairaus) and dynamic penetration test (puristinheijari) are the most common in-situ testing tools. CPTu has been gaining popularity in recent years, while SPT is quite seldom used. CPTu correlations specific to soft Finnish clays exist for constrained soil modulus M (Di Buò 2020). Correlations between sand and silt will likely be available in the upcoming years (D'Ignazio 2022).

No direct correlations are available for soil modulus from weight-sounding and/or dynamic penetration testing. Nevertheless, the Finnish national Geotechnical Design guidelines based on Eurocode 7 (Annex n.6 in NCCI 7 by Liikennevirasto, 2017) provide reference tables to estimate soil parameters for a wide range of coarse soils (see Table 8-1, Table 8-2, Table 8-3). These include silts, sand, moraine, gravel and crushed rocks. Materials as subdivided according to their classification as loose (or very loose), medium dense or dense.

Table 8-1 Table 1 in Annex 6 in NCCI 7 (Liikennevirasto, 2017) – English translation.

Soil type		Unit weight γ (kN/m^3)		Friction angle φ' (°)	Janbu's tangent modulus parameters		Sounding resistance		
		Dry	Saturated		Modulus number m	Stress exponent β	Cone resistance from dynamic penetration test (puristinheijari) q_c (MPa)	Weight sounding P_k/0.2 m (P_k = half rotations)	Blow count from dynamic penetration test (puristinheijari) L/0.2 m (L = blows)
Coarse silt	Loose	14...16	19...	28	30...100	0.3	< 7	< 40	< 8
	Medium–dense			30	70...150	0.3	7...15	40...100	8...25
	Dense	16...18	21	32	100...300	0.3	>15	>100	> 25
Fine sand $d10 < 0.06$	Loose	15...17	19...	30	50...150	0.5	<10	20...50	5...15
	Medium–dense			33	100...200	0.5	10...20	50...100	15...30
	Dense	16...18	21	36	150...300	0.5	> 20	> 100	> 30
Sand $d10 > 0.06$	Loose	16...18	20...	32	150...300	0.5	<6	10...30	5...12
	Medium–dense			35	200...400	0.5	6...14	30...60	12...25
	Dense	18...20	22	38	300...600	0.5	> 14	> 60	> 25

Table 8-2 Table 2 in Annex 6 in NCCI 7 (Liikennevirasto, 2017) – English translation.

Soil type		Unit weight γ (kN/m^3) Dry	Unit weight γ (kN/m^3) Saturated	Friction angle φ' (°)	Janbu's tangent modulus parameters: Modulus number m	Janbu's tangent modulus parameters: Stress exponent β	Sounding resistance: Cone resistance from dynamic penetration test (puristinheijari) q_c (MPa)	Sounding resistance: Weight sounding P_k/0.2 m (P_k = half rotations)	Sounding resistance: Blow count from dynamic penetration test (puristinheijari) L/0.2 m (L = blows)
Gravel	Loose	17…19	20…	34	300…600	0.5	< 5.5	10…25	5…10
	Medium-dense			37	400…800	0.5	5.5…12	25…50	10…20
	Dense	18…20	22	40	600…1200	0.5	> 12	> 50	> 20
Moraine	Very loose	16…19	20…22	…34	(≤100)* 300…600	0.5	< 10	< 40	< 20
	Loose	17…20	20…22	…36	(100…250)* 600…	0.5	This	40…100	20…60
	Medium-dense	18…21	21…23	…38	800…	0.5	-	> 100	60…140
	Dense	19…23	21…24	…40	1200…	0.5	-	Refusal	> 140

Note to Table 8-2: Values with asterisk* are for cases where the moraine has not been subjected to glacial overburden, i.e "normally consolidated moraine".

Table 8-3 Table 3 in Annex 6 in NCCI 7 (Liikennevirasto, 2017) – English translation.

Grain size	Unit weight γ (kN/m^3)	Modulus number m	Stress exponent β	Friction angle φ' (°)
Crushed rock 0..150/0…300	17…22	500…2000	0.5	38…42
Blasted rock 0…300/0…600	17…22	300…1500	0.5	38…42

The tables contain information on unit weight, friction angle, modulus number, and stress exponent for each soil category. The parameters are meant to be used to model drained conditions.

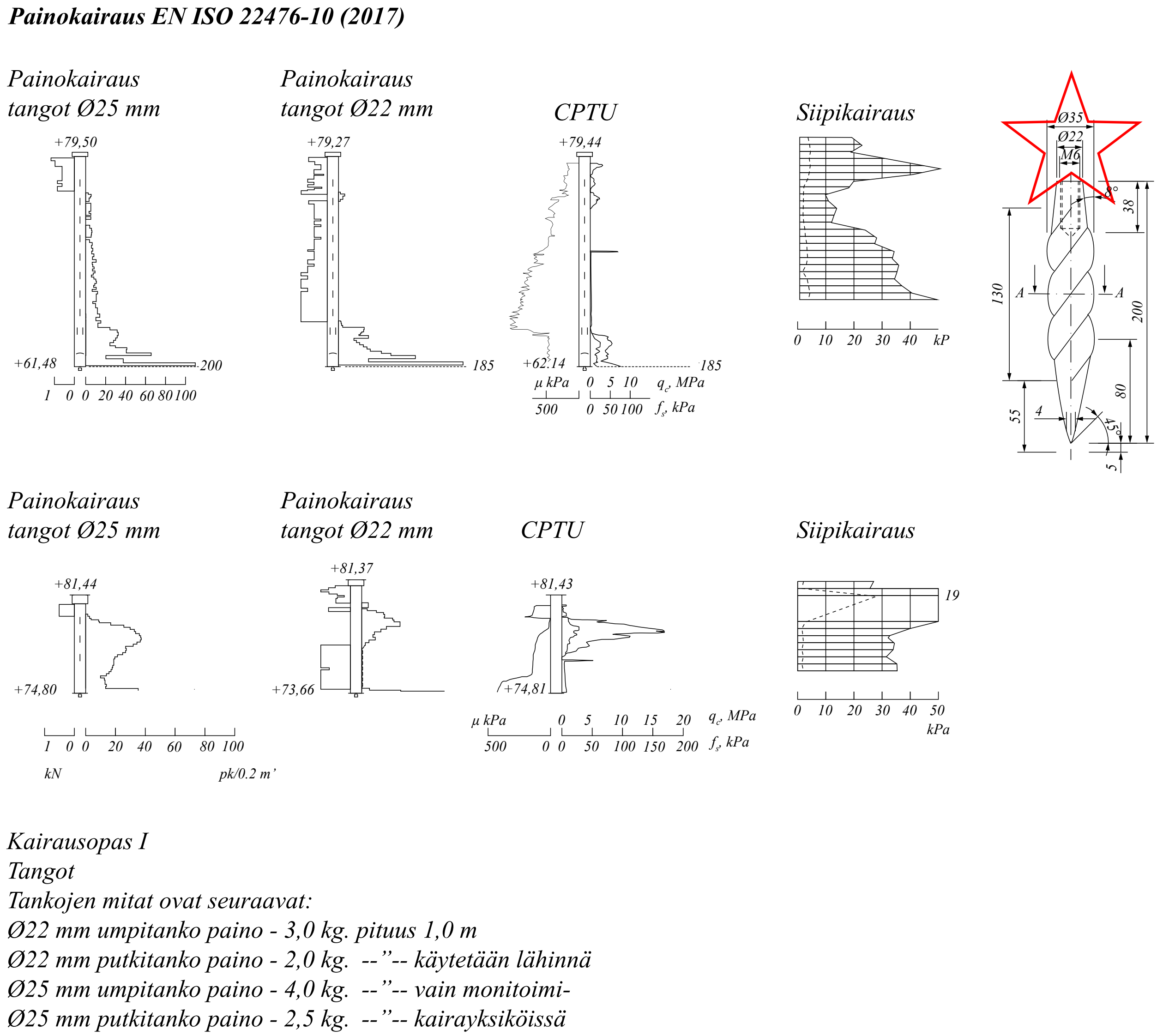
The data in Table 8-1 to Table 8-3 is based on studies done in Finland from the 1960's onwards (e.g. Helenelund 1964, 1966; Tammirinne 1969, Valkeisenmäki 1973). They have been part of established engineering practice in Finland from at least the 1990's onwards, when they were incorporated in official bridge design manuals and later to higher-level guidelines.

They are often referred to even in projects that are not governed by NCCI7 (i.e. projects not related to traffic infrastructure). As such, their use in Finland can be considered safe in terms of established practice. They do not have the same status outside of Finland but may still be carefully used as background reference material. Additional, locally established references may be required.

The suggested ranges for the different parameters are linked with the test results from Weight Sounding (in Finnish painokairaus) and dynamic testing (in Finnish puristinheijari). Therefore, these tables provide guidance when determining soil parameters when in-situ data or/and grain size information is available. It must be noted that some of the data in the tables overlap. It is therefore recommended to carry out at least grain size distribution analyses along with in-situ testing.

Furthermore, the rod diameter of the weight sounding may influence the measurements. As shown in Figure 8-1, the sounding resistance may be overestimated when a rod diameter of 25 mm is used (as is typical with modern geotechnical crawler rigs). The standard 22 mm diameter results appear to be in line with CPTU measurements for the site considered in Figure 8-1. As the tables in NCCI 7 were produced in the 1960's… 1990's, sounding results refer to the 22 mm diameter. This must be considered when using sounding results to estimate soil parameters from NCCI 7.

Figure 8-1 Comparison of weight sounding (painokairaus) results from 22 mm vs. 25 mm rod diameter (courtesy of FTIA / Panu Tolla, 2021).



In the tables, the symbol m is used to indicate the modulus number from the constrained tangent modulus formulation (Janbu 1998):

$$M = mp^{ref}\left(\frac{\sigma'}{p^{ref}}\right)^{1-\beta}$$

where:

M = tangent constrained modulus (kPa, MPa)

p^{ref} = reference stress = 100 kPa ≈ 1 atm

σ' = intergranular pressure, i.e. effective vertical stress (kPa, MPa)

m = modulus number (dimensionless)

β = stress exponent (dimensionless)

The constrained modulus M can be referred to as E_{oed}. Furthermore, $m \cdot p^{ref} = E_{oed}^{ref}$. This leads to:

$$E_{oed} = E_{oed}^{ref}\left(\frac{\sigma_v'}{p^{ref}}\right)^{1-\beta}$$

Note that here, E_{oed} is given as a function of vertical effective stress σ_v'. Depending on the software used, it may also be given as a function of effective mean stress p'. According to the tables, (1 - β) ≈ 0.5 for coarse soils, while ≈ 0.7 for silty soils.

For the onshore wind turbine gravity foundation, the secant modulus for primary loading E_{50} is a parameter of interest. As discussed in section 3.2.1, E_{50} depends on the confinement stress (i.e., cell pressure σ_3') in the triaxial cell, increasing with increasing σ_3'.

Hence, σ_3' can be used as the reference stress to define the drained secant modulus E_{50}. On the contrary, the constrained modulus E_{oed} increases with increasing vertical stress σ_v' or σ_1' The stresses σ_3' and σ_1' can be linked by means of the lateral earth pressure coefficient at rest K_0 as $\sigma_3' = K_{01}'$.

Figure 8-2 shows the stress dependency of E_{oed} and E_{50}. Given that the reference stresses are different and linked by the K_{0nc}, it is observed that $E_{oed} \approx E_{50}$ for NC soil. This was confirmed experimentally by Schanz (1998) for sands (Figure 8-3).

Figure 8-2 Stress-dependent modulus based on the Hardening Soil model's formulation (adapted from Mansikkamäki 2015).

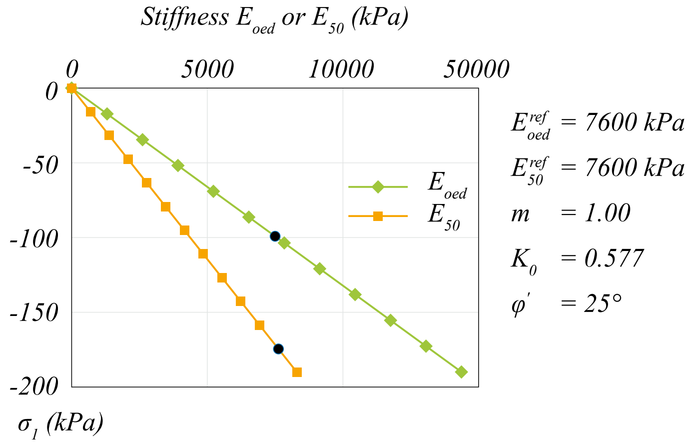
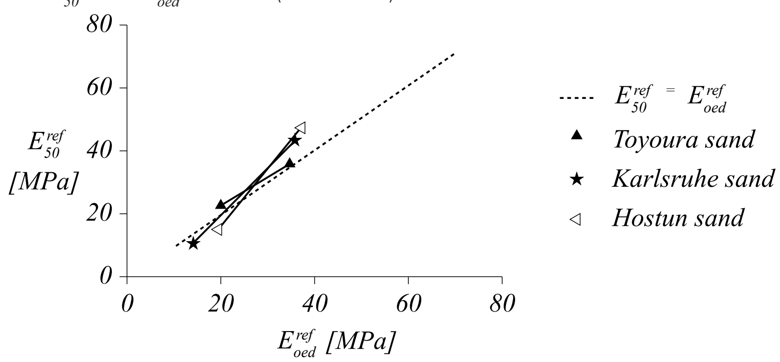


Figure 8-3 Relationship between E_{50}^{ref} and E_{oed}^{ref} for sands (Schanz 1998).



Hence, the stress-dependent secant modulus for primary loading E_{50} can be defined as follows:

$$E_{50} = E_{50}^{ref}\left(\frac{\sigma_3'}{p^{ref}}\right)^{1-\beta} = mp^{ref}\left(\frac{\sigma_3'}{p^{ref}}\right)^{1-\beta}$$

Similarly, the stress-dependent unloading/reloading modulus can be defined as follows:

$$E_{ur} = E_{ur}^{ref}\left(\frac{\sigma_3'}{p^{ref}}\right)^{1-\beta}$$

Typically, when E_{50} or E_{ur} cannot be directly determined from experimental curves, it may be relevant for many practical cases to assume:

$$\frac{E_{ur}}{E_{50}} = 2 \text{ to } 6$$

Higher ratios can be assumed for loose sands (3 to 6) or clays (5 to 10), whereas lower for dense sands (2 to 4) or crushed aggregates (≈ 2). As discussed in chapter 5, the E_{ur}/E_{50} ratio for well-compacted aggregates can be lower than 2. This may reflect the effect of compaction or preloading, resulting in an "overconsolidated" soil state. Any subsequent loading may actually be considered a reloading event.

For sand and silt, the formulation by Andersen and Schjetne (2013) described in 6.3 can be used for a preliminary estimate of unloading and reloading constrained modulus. The average of these two modulus can be used to approximate the $E_{oed,ur}$.

Table 8-4 summarizes recommended values of E_{oed} at p^{ref} = 100 kPa for the coarse-grained soils defined in Table 8-1 to Table 8-3. Recommendations on the $E_{oed,ur}^{ref}$ / E_{oed}^{ref} ratio are based on Andersen & Schjetne's (2013) model illustrated in section 6.3.

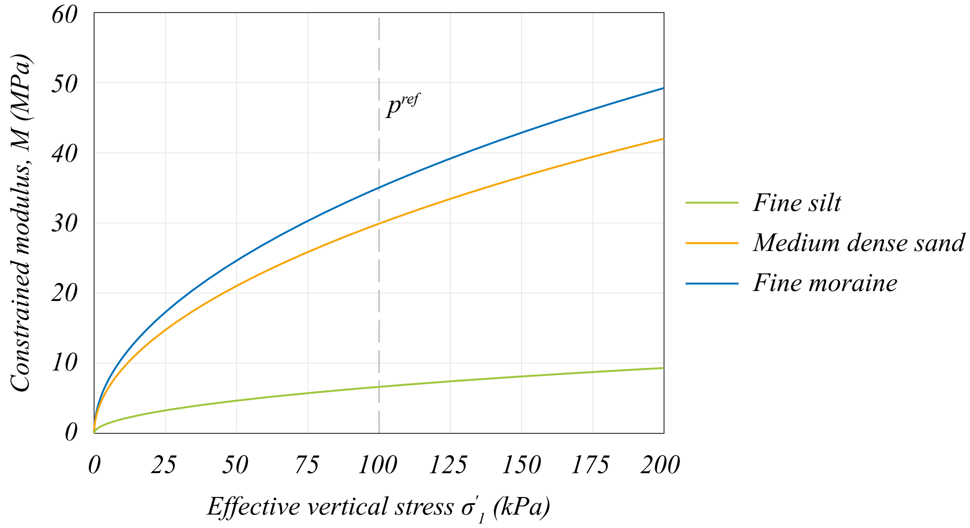
Table 8-4 Range of recommended E_{oed}^{ref} and $E_{oed,ur}^{ref}$ / E_{oed}^{ref} for coarse-grained material based on Liikennevirasto (2017). Note that according to experimental data by Schanz (1998), $E_{50}^{ref} \approx E_{oed}^{ref}$ (see also Fig. 8-3).

Soil type	Density	E_{oed}^{ref} (MPa)*	$E_{oed,ur}^{ref}$ / E_{oed}^{ref} **
Coarse silt	Loose	3–10	8.7–20
	Medium–dense	7–15	6.6–11.1
	Dense	10–30	4.1–8.7
Fine sand *d*10 < 0.06	Loose	5–15	6.6–14
	Medium–dense	10–20	5.4–8.7
	Dense	15–30	4.1–6.6
Sand *d*10 > 0.06	Loose	15–30	4.1–6.6
	Medium–dense	20–40	3.3–5.4
	Dense	30–60	2.5–4.1
Gravel	Loose	30–60	2.5–4.1
	Medium–dense	40–80	2.1–3.3
	Dense	60–120	1.6–2.5
Moraine	Very loose	(≤ 10)*** 30...60	2.5–8.7
	Loose	(10...25)*** 60...	2.5–8.7
	Medium–dense	80...	2.1
	Dense	120...	1.6
Crushed rock 0..150/0...300	–	50–200	1.1–2.8
Blasted rock 0...300/0...600	–	30–150	1.3–4.1

* Calculated at atmospheric pressure p^{ref} = 100 kPa
** $E_{oed,ur}^{ref}$ calculated as the average of unloading and reloading modulus M_u and M_r from section 6.3 at p^{ref} = 100 kPa and assuming maximum stress of $2p^{ref}$ = 200 kPa prior to unloading.
*** For cases where the moraine has not been subjected to glacial overburden, i.e "normally consolidated moraine".

Figure 8-4 illustrates an example of constrained modulus vs. stress σ_1' for different soil types.

Figure 8-4 Constrained modulus M versus vertical effective stress σ_1' for different soil types.



2.2. Fine-grained soils

Field Vane Test is the most popular in-situ tool to test undrained clays and silts in Finland. Field Vane test is used to determine the undrained shear strength s_u. CPTu correlations specific to soft Finnish clays exist for constrained soil modulus M (Di Buò 2020). As most of the clays in Finland are soft, show apparent pre-consolidation due to aging, and especially in coastal areas can be sensitive to extra-sensitive (D'Ignazio 2016), the constrained modulus from CPTu is generally representative of the overconsolidated state. As discussed in section 6, the constrained modulus of sensitive clays is highest before reaching the pre-consolidation pressure and drops dramatically after σ_c' and increases as a function of stress and the modulus number m.

Determining soil modulus in clays would require laboratory testing, e.g., triaxial or direct simple shear (DSS). Constrained modulus can be determined from oedometer test results. In the absence of laboratory testing, correlations based on undrained shear strength, water content, plasticity index, and over-consolidation ratio can be used for preliminary assessment. In situ tests such as weight sounding or dynamic penetration testing have not been calibrated in clays and therefore do not provide any information on parameters.

The constrained modulus of clays Eoed follows the stress-dependent modulus formulation introduced in previous sections:

$$E_{oed} = E_{oed}^{ref}\left(\frac{\sigma_v'}{p^{ref}}\right)^{1-\beta} = mp^{ref}\left(\frac{\sigma_v'}{p^{ref}}\right)^{1-\beta}$$

As discussed by Janbu (1963, 1998), the stress exponent β for most clays tends to 0, giving $(1 - \beta) \approx 1$. For extra-sensitive clays, β can be negative, resulting in $(1 - \beta) > 1$ (e.g. Länsivaara 1999). The modulus number m can be estimated based on, e.g., water content, as illustrated in Figure 6-6.

The ratio $1/m$ represents the modified compression index λ^*, indicating the slope of the normally consolidated oedometer compression line in ε-log϶ plot. Therefore, the oedometer modulus at atmospheric pressure can be written as:

$$E_{oed}^{ref} = \frac{p^{ref}}{\lambda^*}$$

The unloading-reloading modulus is linked to the modified swelling index κ*, indicating the slope of the overconsolidated oedometer compression line in ε-logσ' plot, as

$$E_{ur,oed}^{ref} = \frac{p^{ref}}{\kappa^*}$$

Literature suggests λ^*/κ^* ratio varies between 5 and 10 for Finnish clays. It can be higher for extra-sensitive clays (Mansikkamäki 2015).

For a typical range of $m \approx$ 5–30 in Scandinavian soft (normally consolidated) clays (Janbu 1998; Karlsrud & Hernandez-Martinez 2013; Di Buò 2020), E_{oed}^{ref} = 0.5–3 MPa with $E_{oed,ur}^{ref} / E_{oed}^{ref}$ = 5–15. For overconsolidated clays with pre-consolidation pressure $\sigma_c' >> \sigma_{v0}' + q_F$ (q_F = foundation load), the $E_{oed,ur}^{ref} / E_{oed}^{ref}$ ratio can be lower than for NC clays. This is due to the higher stiffness in the OC region. In that case, $E_{oed,ur}^{ref} / E_{oed}^{ref}$ = 1–3.

For soft clays, E_{50}^{ref} can be as high as $2E_{oed}^{ref}$ (Plaxis 2022). In general, the primary loading of gravity foundations on clay is initially governed by undrained conditions. Hence, the calculation model used shall be able to estimate the undrained modulus $E_{u,50}$ from the drained E_{50} when needed. The $E_{u,50}$ in undrained conditions is higher than the drained E_{50} due to the higher Poisson's ratio $\nu_u \approx$ 0.5.

For cases where the calculation method requires direct input of the undrained modulus, this can be estimated from undrained triaxial tests or, in the absence of laboratory data, from correlations. Figure 8-5 and Figure 8-6 illustrate the relationship between the soil modulus of clays and the undrained shear strength s_u.

Figure 8-5 shows a relationship between the undrained modulus E_u and the undrained shear strength su as a function of the over-consolidation ratio (OCR) and plasticity index I_p. Figure 8-6 illustrates the relation between G_{50}/s_u and the plasticity index I_p of clays.

These relationships can be used for instance, when su from the Field Vane test is known. Alternatively, the G_{max}/ s_u^{DSS} ratio can be estimated according to Andersen (2015) if s_u^{DSS}, roughly corresponding to Field Vane test conditions, OCR, and plasticity index I_p is known (section 4). The G_{max} can then be reduced according to shear stress mobilization via the modulus reduction factor (MRF) presented in section 4.

Figure 8-5 Ratio E_u/s_u as a function of OCR and I_p (Duncan and Buchignani 1976).

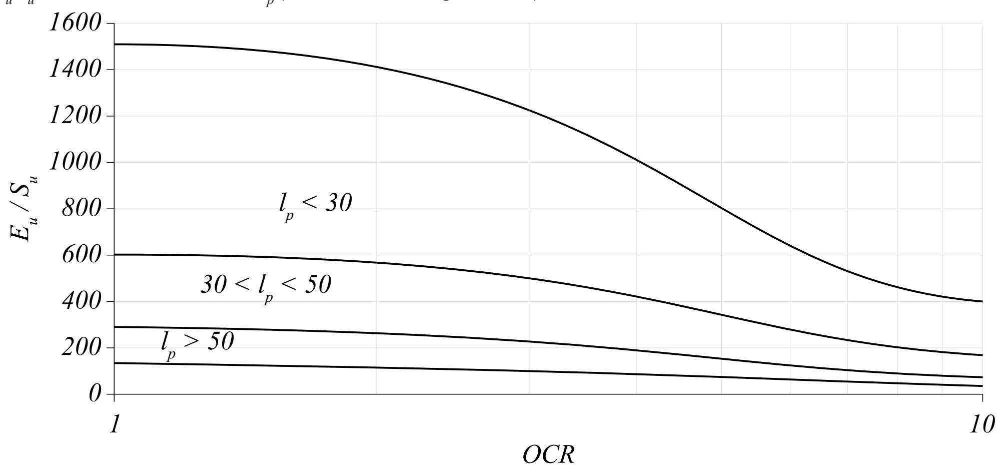
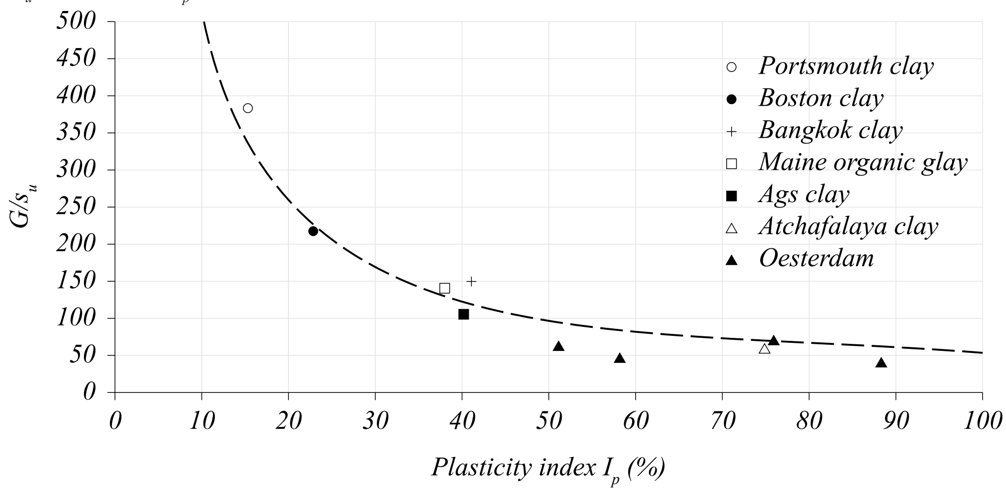


Figure 8-6 Ratio G/s_u as a function of I_p (Termaat et al. 1985).



3. STEP-BY-STEP PROCEDURE FOR THE DETERMINATION OF E_{50} AND E_{ur}

3.1. General remarks and procedure flow

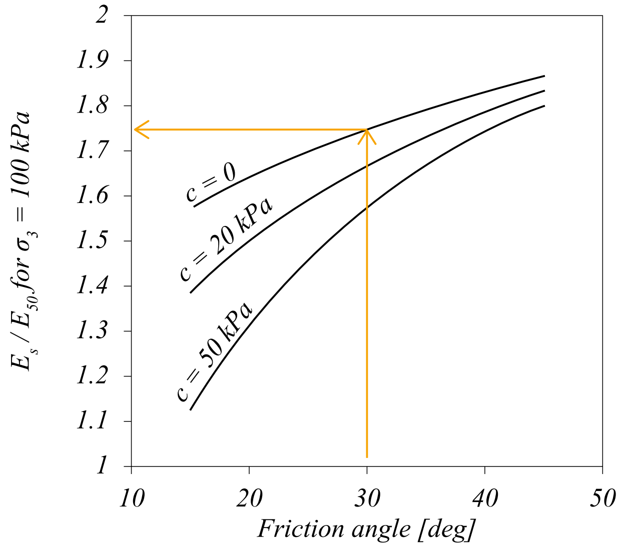
As discussed in previous chapters, the determination of soil modulus can follow different paths based on the available soil data for a specific project. This chapter summarizes a step-by-step procedure to establish the drained triaxial secant Young's modulus at 50% of the maximum deviator stress E_{50} and the unloading/reloading modulus E_{ur} for both coarse-grained and fine-grained soils, based on the information presented in this report.

Triaxial testing on coarse-grained material is generally carried out as drained. Therefore, determining E_{50} from the q-ε_a stress-strain curve is straightforward. On the other hand, undrained triaxial testing is more common in fine-grained material. In this case, it is recommended to estimate G_{50} from the shear stress $\tau = q/2$ vs shear strain $\gamma = 1.5\varepsilon_a$ curve and estimate the drained $E_{50} = 2G_{50}(1+\nu)$.

In general, the primary loading of gravity foundations on clay is initially governed by undrained conditions. Hence, the calculation should also be able to estimate the undrained modulus $E_{u,50}$ from the drained E_{50}. The $E_{u,50}$ in undrained conditions is higher than the drained E_{50} due to the higher Poisson's ratio $\nu_u \approx 0.5$. For cases where the calculation method requires direct input of the undrained modulus, this can be estimated directly from, e.g., Figure 8-5 from OCR and I_p or, alternatively, estimate G_{50} from s_u and I_p from Figure 8-6 and calculate $E_{u,50} = 2G_{50}(1+\nu_u) \approx 3G_{50}$.

In geotechnical interpretative reports, Young's modulus is often given without information on the strain level at which it is estimated. Especially when E (or E') for coarse-grained soils is determined from CPT/CPTu, the correlations by Robertson and Cabal (2015) presented in chapter 7.3 are used. Young's modulus can then be considered as the modulus at 0.1% strain. Based on this, Obrzud and Truty (2018) proposed a method to estimate E_{50} from E' (or E_s, static), as a function of cohesion and friction angle of the soil, as shown in Figure 9-1.

Figure 9-1 Relationship between the E_{50} and E at 0.1% strain as a function of cohesion and friction angle (Obrzud and Truty 2018).



3.2. Determination of E_{50} and E_{ur}

Figures 9-2 and Figure 9-3 illustrate the steps to determine E_{50} and E_{ur} based on the available soil data. Note that while there are "decision gates" based on which types of tests are available, engineering judgment should be applied. Different methods may often give conflicting results, and it's up to the designer to choose which estimate of modulus is the most reliable. In general, the fewer correlations and conversions are needed, the less uncertainty there is.

When using modulus reducing factor, MRF covered in Chapter 4 and mentioned in Figures 9-2 and 9-3 geo enginner shall check or understand if soils characteristic to the region confirms to stated G_{max}/G relations. Evidence suggests that for specific soils (for example, highly consolidated low plasticity clays with very high G_{max}), given relations are not valid and may give too high E_{50} values.

It should still be noted that while directly testing good quality samples can be considered the ideal method of determining soil modulus, **well-established correlations from in situ measurements should be preferred over testing of poor-quality samples.**

For fine-grained soils such as clays, good quality samples can be achieved, e.g., by using piston samplers, large diameter tube samplers, or by taking block samples (see, e.g., standard ISO 22475-1). Core drilling and other "violent" sampling methods will likely disturb the soil structure enough that laboratory testing of such samples will show a much lower modulus than what could be determined from good quality samples or in situ testing. Furthermore, good quality samples must be properly handled and stored to avoid sample disturbance.

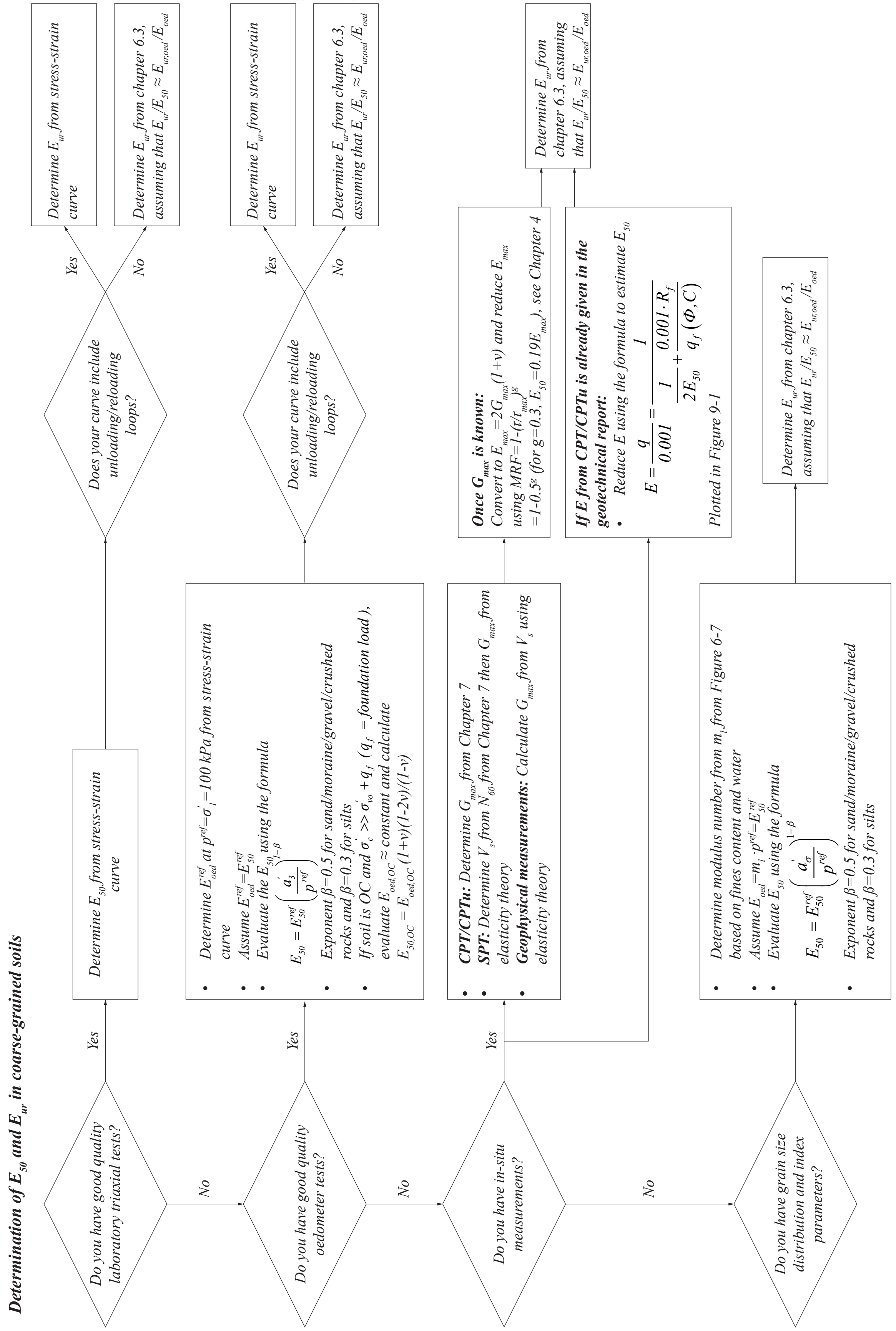


Figure 9-2 Flow-chart for determination of drained E_{50} and E_{ur} for coarse-grained material.

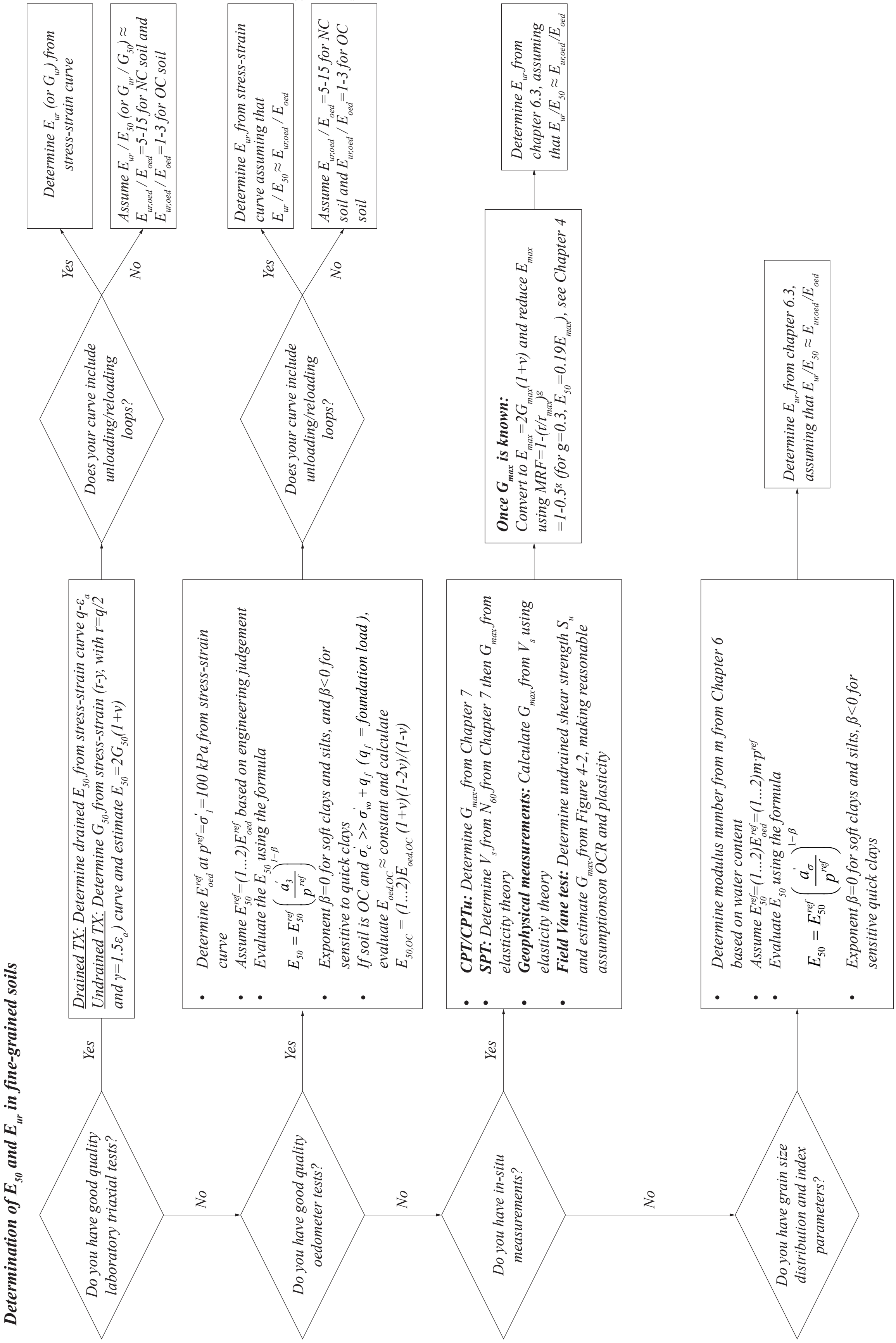


Figure 9-3 Flow-chart for determination of drained E_{50} and E_{ur} for fine-grained material.

3.3. Applying determined modulus values to software with HS formulation

Before design calculations can be made, the stress-dependency of soil stiffness should be modeled in the design software accordingly. Any modulus value determined from laboratory or in situ data is associated with a stress level where it has been determined (i.e., the in situ stress state or the stress state to which the soil sample has been consolidated). The determined modulus value must be converted to a reference modulus value at a reference stress level.

A typical conversion that often needs to be made is to convert measured (or otherwise determined) modulus E at a given stress state to the reference value E'_{ref} at the reference stress p^{ref} = 100 kPa.

We have the general relationship (here, E is presented as a function of vertical stress, as is typically done in relation to oedometer testing):

$$E = E\left(\sigma'_v\right) = E^{ref}\left(\frac{\sigma'_v}{p^{ref}}\right)^{1-\beta}$$

<–>

$$E^{ref}\ \frac{E}{\left(\frac{\sigma'_v}{p^{ref}}\right)^{1-\beta}}$$

Depending on the calculation software, this relationship may also be given as a function of effective mean stress p' or minor principal stress σ'_3. The user should be aware of which relationship is used in the given context.

One significant case is software that uses the Hardening Soil formulation for stress-dependent soil stiffness (E_{50} or E_{ur}, see e.g. Obrzud & Truty 2018):

$$E = E\left(\sigma'_3\right) = E^{ref}\left(\frac{\sigma^*_3 + c\cdot\cot\left(\varphi\right)}{\sigma^{ref} + c\cdot\cot\left(\varphi\right)}\right)^m$$

where :
E is either E_{50} or E_{ur}
E_{ref} is the corresponding reference modulus at the given reference stress
σ^*_3 is $max(\sigma'_3$; 10 kPa)
σ^{ref} is the minor stress where the stiffness E is determined.
m is the stress exponent (equivalent to $1\text{-}\beta$ in the general formulation – **not** the tangent modulus method modulus number!)

Note that Obrzud & Truty (2018) are not quite clear in their notation between effective and total stresses, but effective stresses are implied as modulus depends on effective stresses.

The Hardening Soil formulation allows the reference stress value can be chosen freely. Therefore, if E_{50} has been determined, for example, from a triaxial test, the cell pressure σ'_3 can be input as the reference stress σ^{ref}, and the reference modulus is directly the modulus value determined from the test.

If the E_{50} value is determined based on in situ testing or unconsolidated triaxial testing, the reference stress should correspond to the stress at the depth where the test is done or where the sample is taken. The vertical stress at a given depth can generally (assuming horizontal soil layers) be calculated as:

$$\sigma_v' = \sigma_v - u = \sum (h_i \cdot \gamma_i) - u$$

where:
h_i and γ_i are the thickness and total unit weight of a given soil layer above the given depth
u is the pore pressure at the given depth.

For conversion between σ_v' and σ_3', the following Equation may be used (while assuming that the minor principal stress is the horizontal stress:

$$\sigma_3' = \sigma_h' = k0 \cdot \sigma_v'$$

As an alternative to using the test stress as the reference stress, one can express the E value at the "standard level" of 100 kPa. The conversion to reference stress can then be made by:

$$E^{ref} = \frac{E}{\left(\frac{\sigma_3^* + c \cdot \cot(\varphi)}{\sigma^{ref} + c \cdot \cot(\varphi)} \right)^m}$$

where:
E is the determined modulus from the test
σ_3^* is the effective minor principal stress for the test
σ^{ref} is 100 kPa

For more detailed discussion of the Hardening Soil stiffness formulation, see e.g. Obrzud & Truty (2018) Chapter 2.

Example, test result to Hardening Soil parameters:

For a sand (φ' = 30°, c' = 0.1 kPa, m = 0.5, $K0$ = 0.5), E_{50} has been determined to be E_{50} = 50 MPa at an in situ stress state σ_v' = 70 kPa. Give the corresponding Hardening Soil input parameters σ^{ref} and E_{50}^{ref}:

a) As a function of σ_3', with test stress level as the reference stress
b) As a function of σ_3', with reference stress 100 kPa
c) As a function of p', with test stress level as the reference stress
d) As a function of p', with reference stress 100 kPa

a) As a function of σ_3', with test stress level as the reference stress

$$\sigma^{ref} = \sigma_3^* = \sigma_h' = K0 \cdot \sigma_v' = 0.5 \cdot 70\ kPa = 35\ kPa$$

$$E_{50}^{ref} = E_{50} = 50\ MPa$$

b) As a function of σ_3', with reference stress 100 kPa

$$\sigma^{ref} = 100\ kPa$$

$$E_{50}^{ref} \frac{E_{50}}{\left(\frac{\sigma_3^* + c \cdot \cot(\varphi)}{\sigma^{ref} + c \cdot \cot(\varphi)} \right)^m} = \frac{50\ MPa}{\left(\frac{35\ kPa + 0.1\ kPa \cdot \cot(30°)}{100\ kPa + 0.1\ kPa \cdot \cot(30°)} \right)^m} = 84.4\ MPa$$

c) As a function of p', with test stress level as the reference stress

$$\sigma^{ref} = p' = \frac{\sigma'_{v}\left(1+2+K0\right)}{3} = \frac{70kPa\left(1+2+0.5\right)}{3} = 46.7\ kPa$$

$$E_{50}^{ref} = E_{50} = 50\ MPa$$

d) As a function of p', with reference stress 100 kPa

$$\sigma^{ref} = 100\ kPa$$

$$E_{50}^{ref} \frac{E_{50}}{\left(\frac{p^{*} + c\cdot cot(\varphi)}{\sigma^{ref} + c\cdot cot(\varphi)}\right)^{m}} = \frac{50\ MPa}{\left(\frac{46.7\ kPa + 0.1\ kPa\cdot cot(30°)}{100\ kPa + 0.1\ kPa\cdot cot(30°)}\right)^{m}} = 73.1\ MPa$$

4. SUMMARY AND CONCLUSIONS

The report has presented a comprehensive study on soil modulus for a wide range of soil types for the design and analysis of onshore wind turbine foundations. As reported, soil stress-strain behavior is non-linear. Consequently, the determination of a soil modulus is not a straightforward process. Soil modulus is not constant and is affected by several factors, including state factors (particle density, water content, stress history, cementation) and loading factors (stresses and confinement, strain level, rate effects, number of cycles, drainage, intermolecular and surface forces).

An appropriate soil modulus characterization would require an extensive set of laboratory and in-situ tests. In laboratory tests can be more directly measured modulus values needed for soil modeling, but quite often, laboratory samples can be disturbed and do not precisely represent soil conditions in-situ. However, in-situ tests usually do not measure directly needed modulus, but other parameters such as cone resistance and then using correlation can be obtained needed values. The use of correlation makes in-situ less accurate, but in-situ tests usually better represent the actual soil state in nature (soil is not disturbed). Triaxial and oedometer tests provide a good background to determine drained primary loading secant modulus (E_{50}), secant unloading/reloading (E_{ur}) and tangent constrained (E_{oed}) modulus. Bender element or resonant column tests are ideal for studying the small-strain tangent shear modulus G_{max}. However, these tests are too often limited or unavailable, especially in small-sized projects. In-situ tests can be used to estimate soil modulus based on literature correlations in the absence of site-specific laboratory tests. Among these, the cone penetration test (CPT or CPTu) is considered the most reliable. Tests such as standard penetration test (SPT) are characterized by larger uncertainty.

Weight-sounding (painokairaus) or dynamic cone penetration testing (puristinheijari), which are widely used in Finland do not provide any direct information on soil parameters. Nevertheless, national geotechnical design guidelines (NCCI 7 by Liikennevirasto, 2017) provide guidance for selecting strength and stiffness parameters of coarse-grained soils based on the results of such tests. The NCCI 7 data is based on studies done in Finland from the 1960's onwards, which have been part of the established engineering practice from at least the 1990's onwards when they were incorporated in official bridge design manuals and later to higher level guidelines. They are often referred to even in projects that are not governed by NCCI 7 (i.e. projects not related to traffic infrastructure). As such, their use in Finland can be considered safe in terms of established practice. They do not have the same status outside of Finland but may still be carefully used as background reference material. Additional, locally established references may be required.

The report further summarized correlations to establish soil modulus for different soil types in the absence of laboratory data for both coarse-grained and fine-grained soils. These are based on basic soil index properties. These correlations shall be only used for a preliminary estimate of foundation performance and validated by means of laboratory and/or in-situ tests during subsequent design phases.

A final chapter illustrates a step-by-step procedure to determine E_{50} and E_{ur} based on the available soil investigation data for both coarse-grained and fine-grained soils. The aim is to provide a tool to guide geotechnical designers through the contents of this report when establishing design parameters.

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