Haga Station & Quick Clay of the West Link Project in Gothenburg – Geotechnical and structural design approach

G. Lunardi[1], G. Cassani[2], M. Gatti[3], V. Torricelli[4]
[1]*CEO Rocksoil S.p.A, Milan, Italy*
[2] *Technical Director Rocksoil S.p.A, Milan, Italy*
[3] *Head of Technical Department Rocksoil S.p.A, Milan, Italy*
[4] *Project Manager Rocksoil S.p.A, Milan, Italy*

E-mail: martino.gatti@rocksoil.com

ABSTRACT: Haga Station is part of the West Link Project in Gothenburg, which is one of Sweden's largest and most complex rail projects. Part of the station is located in glaciomarine clays deposits with an estimated depth of 45-50 m, covered by filling consisting of gravel and sand with a thickness between 1 and 5 m and separated from the bedrock by a friction soil layer with thickness ranging between 0.5 to 15 m. The paper presents the characteristics of this sensitive clay by means of several tests, describing in detail the physical and mechanical properties. Specific soil models to be used for calculations by FEM analyses are also presented, both in undrained and drained conditions such as the NGI-ADP model and the Soft Soil Model. The models are calibrated based on experimental data coming from field and laboratory investigations performed in the Haga station area; specific simulations of the laboratory tests by means of the Plaxis "Soil Test" tool are performed too, so to refine the soil models. Finally, the design approach to be used to manage deep excavation in quick clay is defined, outlining the most suitable interventions to be used to avoid negative impact on the surrounding area.

KEYWORDS: Underground station, quick clay, geotechnical analyses, numerical approach, design solutions

1. INTRODUCTION

The railway system in the Gothenburg region has reached maximum capacity. To overcome these problems, a new line, the West Link, is to be built for through traffic, commuter traffic and regional trains.
The West Link, also known as Västlänken, is an 8km-long commuter train connection through central Gothenburg, Sweden, 6.6km of which will be carried in a double-track railway tunnel. The project will also add three new stations at Haga, Korsvägen and Gothenburg central. The complex geological condition requires detailed analysis of the soil parameters.
The paper describes in detail the geotechnical properties of Gothenburg sensitive clays starting from the information obtained by available and new investigations performed in Haga Area, and elaborating them through analyses and interpretation carried out on the rough data.
The correct understanding and knowledge of the clay properties assume an outmost importance and it has the major impact on the design of underground structures in the Westlink Project, which is a railway connection in a tunnel under the centre of Gothenburg, mostly executed in this geotechnical material.
Advanced numerical models have been used in the design of E04 Haga stretch, which includes Rosenlund shaft, Rosenlund canal and Haga station. The complex properties of the clay have been simulated, both in undrained short-term and in drained long-term conditions, using appropriate constitutive soil models, such as NGI-ADP and Soft soil, whose parameters have been calibrated on the base of experimental data and verified through a specific software tool.

2. GEOLOGICAL STRATIGRAPHIC FRAMEWORK

E04 Haga area is characterized by the following stratigraphy:

- Fill material, having a thickness between 1 and 3 m, consisting of clay to a great extent as well as sand, gravel, wood, concrete, cobblestones and pilings
- Glaciomarine clay deposits (Soils and quaternary deposits) having a maximum estimated depth of 40-45 m
- Friction soil layer, having a thickness ranging between 0.5 m (Rosenlund shaft area) and 7 m (Rosenlund canal area), which is usually present between the clay layer and the bedrock, and creates a transition permeable layer between the upper and lower impermeable layers
- Bedrock that, in Gothenburg area, is generally subdivided into granites in the Western part and gneisses in the East, divided by an extensive mylonite zone, running N-S.

Figures 1&2 show the geotechnical profile of E04 Haga Station and the underground structures inserted in the geotechnical layout, being the excavations performed partly in clay and partly in rock.

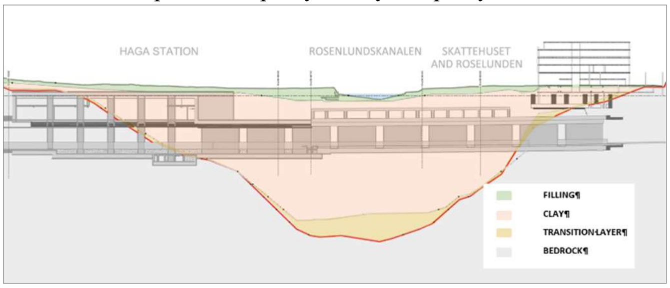

Figure 1 Geotechnical longitudinal profile

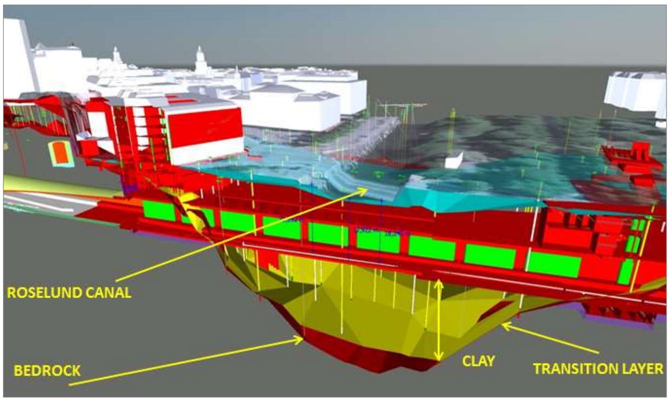

Figure 2 Geotechnical layout of the E04 Haga structures

3. CLAY GEOTECHNICAL PROPERTIES

The properties of clay have been defined on the base of available investigation and a survey campaign carried out during the design stage. Attention has been focused on the aspects with the highest impact on the design issues, such as:

- undrained soil shear strength
- drained soil shear strength

- soil deformability
- pre-consolidation pressure and over-consolidation ratio
- soil index properties (density, water content, Atterberg limits, sensitivity)
- pore pressure values

In particular, undrained and drained soil shear strength together with soil deformability have a crucial importance in the design of underground structures in Gothenburg clay, and need to be carefully assessed to get the necessary parameters for the numerical analyses.
In the following paragraphs, performed tests and investigations are presented, and clay physical and mechanical properties described in detail.

3.1 Field and laboratory investigations

The investigations have been conducted from May 2012 to April 2014 and from June to August 2015 providing base material for geotechnical elaborations. The performed field surveys consist of sampling, probing and field test such as CPT and Vane test. Laboratory tests have been then performed, that include stratigraphy, density, water content, Atterberg limits and sensitivity determination. Supplementary investigations have been carried out during design stage: in Rosenlund area two boreholes have been performed and triaxial and oedometer tests conducted on the samples extracted, that basically confirm the data available from the previous campaigns.
Figure 3 show the position of investigations available in Rosenlund area.

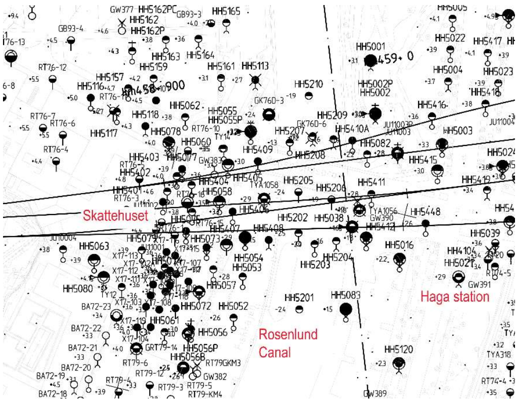

Figure 3 Map of study area (Rosenlund area)

3.2 Undrained shear strength

The undrained shear strength varies with stress direction and can be accordingly classified with respect to the direction of the major stress: typically active when the major stress is vertical, passive when it is horizontal and direct in a horizontal sliding surface The average shear strength is a function of all three. However, C_u calculated from the direct shear test has empirically been found to satisfactory represent the average value and it's applicable in most loading situation (Larsson 1980, Westerberg 2015).
The direct shear test commonly used in Sweden is the Consolidated Undrained Direct Shear apparatus. The test consists of two phases: consolidation (drained) and shearing (undrained or drained depends on the test). The main data coming from the test is the direct undrained shear strength.
Information about the active and passive undrained strength can be estimated from the consolidated undrained triaxial test in compression and extension respectively.
The undrained shear strength coming from the Field Vane test should be corrected according to SGI (1969) with a µ factor as follow:

- $z = 0 - 15\,m \qquad \mu = \left(\frac{0.43}{w_L}\right)^{0.45}$ (1)
- $z = 15 - 20\,m \qquad \mu = \left(\frac{0.43}{w_L}\right)^{0.45} \quad until \ \ \mu = 1$ (2)
- $z > 20\,m \qquad \mu = 1$ (3)

Figure 4 compares undrained shear strength values obtained from Field Vane test (circular points, FV), Consolidated Compression Undrained Triaxial test (coloured triangle points, Tx), Consolidated Extension Undrained Triaxial test (white triangle point, Tx_ext) and Consolidated Undrained Direct Shear test (square points, DS) results. As expected, the active undrained shear strength is slightly higher than the direct strength (square and circular points). Moreover, the results are concentrated in a small band with a significant linear trend with depth from a minimum value of 20 kPa at 4-5 m depth, to almost 70 kPa at 45 m of depth. Different tests (DS and FV) performed in samples extracted from the same borehole at the same depth, show similar undrained strength (see for example the light green squares and circles (CH5022, FV- DS) at 5 and 12m) especially in the first 30 m.

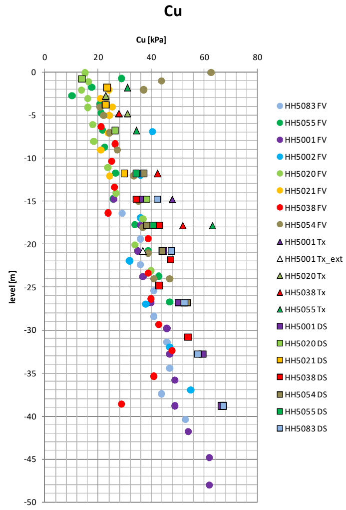

Figure 4 Variation along depth of undrained shear strength

Empirical correlations exist between the tip cone resistance and the undrained shear strength:

$$Cu = \frac{q_c - \sigma_{v0}}{Nk} \qquad \text{(Mayne\&Kemper,1988)} \qquad (4)$$

with q_c tip resistance, σ_{v0} geostatic vertical stress in situ and Nk empirical coefficient depending on the specific instrumentation adopted for the CPTU test (Nk=15 in this case);

$$Cu = \frac{q_t - \sigma_{v0}}{13.4 + 6.65\, w_L} \qquad \text{(Common Swedish practice)} \quad (5)$$

with qt tip resistance, σ_{v0} geostatic vertical stress in situ and wL liquid content.
These empirical relations provide similar results. Deviation smaller than 5 kPa in the first 20 m and less than 10 kPa in the other 30 m.
Figure 5 compares undrained shear strength values obtained from different tests.

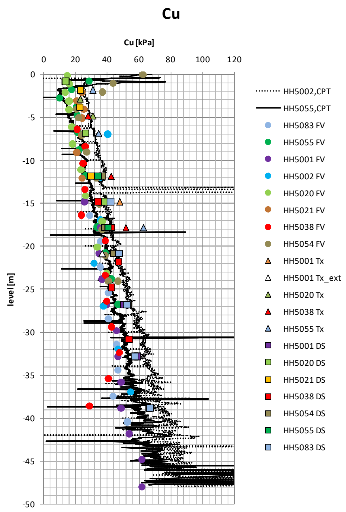

Figure 5 Variation along depth of undrained shear strength

Compared tests are Cone Penetration test (dotted line, CPTU), Field Vane test (circular points, FV), Consolidated Compression Undrained Triaxial test (coloured triangle points, Tx), Consolidated Extension Undrained Triaxial test (white triangle point, Tx_ext) and Consolidated Undrained Direct Shear test (square points, DS). For sake of clarity just one CPTU result is reported and estimated by Mayne and Kemper empirical relation. This curve can be reasonably considered as representative of all CPTU results. Good agreement is observed between different test results. Some high C_u values at 13 m depth and at 45 m depth (out of range of the general linear trend) come from high tip resistance values at these depths and can be caused from the presence of small sand lens.
Moreover, the areal homogeneity of Haga station's clay is confirmed since different samples extracted in different boreholes, at different depths show almost the same Cu/σ'c ratio (undrained active shear strength and consolidation pressure).

3.2.1 Residual undrained shear strength

Sensitive clays are characterized by residual behaviour after the peak condition.

3.3 Drained strength

3.3.1 Friction angle

The undrained triaxial test presented in the Figure 6 were performed in 9 samples (4 boreholes: HH5054 (18m, 24m), HH5020 (6m, 8m), HH5038 (8m, 15m, 21m), HH5055 (5m,10m,21m)). The undrained triaxial tests presented in p'-q space were first consolidated to an estimated in situ stress and the sheared undrained with a deformation rate of 0.01 mm/min. The results from the undrained compression tests indicate a critical friction angle in the range of 27-33° for the compression side, i.e. Mc=1.3-1.4 and about 35° for the extension side, i.e. Me=0.97.
All stress strain curves obtained in the compression tests are showing softening behaviour after reaching the peak. The results reported in the Figure 7 indicate that the peak strength is reached at an axial strain level of about 2-4% for all compression test except for the two samples HH5038 – 8, 15 extracted in the same borehole, which need more deformations (5-7 %) to reach the peak strength, but the mobilized angle at 2-4% of accumulated axial deformation is similar to the values obtained for samples extracted nearby. During the extension test the mobilised friction angle regularly increase its value until 8% of axial strain.

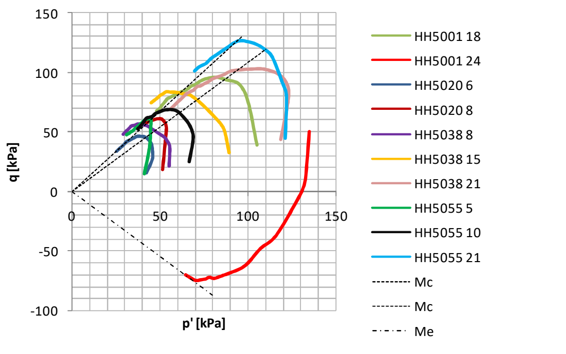

Figure 6 Undrained triaxial test both in compression and extension

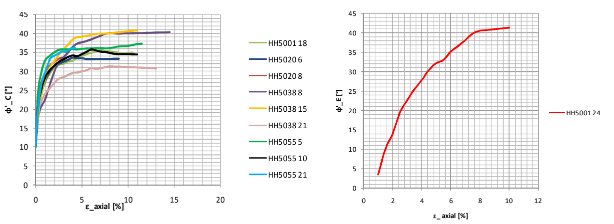

Figure 7 Mobilization of friction angle with axial deformation in triaxial undrained compression (left) and extension (right)

The mechanical strength properties of the clay are listed in Table 1.

Table 1 Summary of mechanical strength properties

CLAY							
Parameters' values variable with depth	Level [m]						
	-1÷-4	-4÷-19	-19÷-21	-21÷-25	-25÷-37	-37÷-40	-40÷-50
Choesion c' [kPa]	0	0	0	0	0	0	0
Friction Angle ϕ'c [°]	30	30	30	30	30	30	30
Angle of dilatancy Ψ [°]	0	0	0	0	0	0	0
K_0 value for normal consolidation K_0^{NC} [-]	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Undrained shear strength, passive shearing $C_{U,\ PASSIVE}$ [kPa]	18	18÷37	37÷39	39÷43	43÷55	55÷59	59÷65
Undrained shear strength, direct shearing $C_{U,\ DIRECT}$ [kPa]	22	22÷42	42÷44	44÷48	48÷56	56÷65	65÷74
Undrained shear strength, active shearing $C_{U,\ ACTIVE}$ [kPa]	28	28÷56	56÷70	70÷78	78÷102	102÷108	108÷120

The low values of parameters, especially of C_u, which governs the behaviour of the structures in undrained conditions, should be remarked since most of the excavation stage is performed in undrained conditions and those properties are connected both to high soil thrusts on the active side and to low resistance on the passive side of retaining structures, that require a robust structural solution and the need of improving the characteristics of the soil.

3.4 Soil deformability

3.4.1 Constrained modulus

The constrained modulus E_{oed} also called ML when dealing with plastic deformations, is estimated from CRS and IL oedometer test directly analysing the stress-strain curves.
The constrained modulus can be also calculated from the CPTU test by means of an empirical relation:

$$E_{oed}\left(\frac{kg}{cm2}\right) = \alpha\, q_c \quad \text{(Mitchell \& Gardner, 1975)} \quad (6)$$

where with q_c is the tip resistance,α is an empirical coefficient related to the soil type, as reported below. Gothenburg clay at this site is a inorganic clay of high plasticity (CH). The value of α chosen is equal to 6.

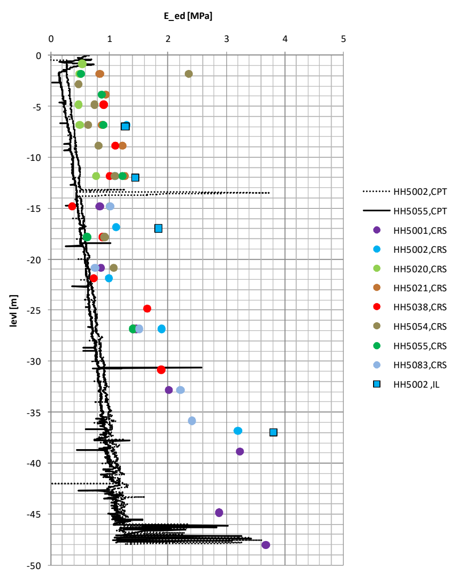

Figure 8 Variation of constrained modulus obtained by laboratory and CPTu tests

Figure 8 compares constrained modulus values obtained from Cone Penetration test (black line, CPTU), Constant Strain Rate Oedometer test (circular points, CRS) and Increment Loading Oedometer test (square points, IL).
For sake of clarity, just one CPTU result is reported and estimated by Mayne and Kemper empirical relation. Looking at the graph, good agreement can be seen between laboratory tests while CPTU

empirical relation underestimate the constrained modulus, especially at greater depth.
The soil stiffness increases regularly with depth (0.5-4 MPa), in particular great increase is observed between 25 and 50m depth (from 1.5MPa to 4 MPa).
Some empirical relations are presented in order to give a wider framework of the clay stiffness.

$$E_{u,25} = n * C_u \qquad \text{(Ladd, 1977)} \qquad (7)$$

where
n = 100 (minimum value for CH clay type, with wL=71%, IP=40).
Relation is based on the assumption of normalized behaviour. In this specific case Cu/σ'c seems to be almost constant with depth and a common compression line maybe exists (see previous paragraph), so this relation can be accepted.

$$V_s = A\left(\frac{q_c}{p_A}\right)^{\alpha} \rightarrow G_0 = 2 * E_0 * (1-\nu) \rightarrow E = \frac{E_0}{10} \rightarrow$$

$$E_{ed} = E * \frac{1-\nu}{(1+\nu)(1-2\nu)} \qquad \text{(Rix \& Stokoe, 1991)} \qquad (8)$$

Mainly used for sand.

$$\frac{E_u}{C_u} = 300 \text{ or } \frac{E_u}{C_u} = \frac{15000}{IP} \rightarrow E_{ed} = \frac{2}{3}E_u * \frac{1-\nu}{(1-2\nu)}$$

$$\text{(Duncan \& Buchignani, 1976)} \qquad (9)$$

Based on the ratio Eu/Cu. They suppose the ratio depends on the stress history, stress level and plasticity index (Figure 9).
The previous calculated constrained modulus ML, is the compression stiffness representative of the stress interval acting just after the pre-consolidation pressure. However, as the effective stress increase, the soil response is characterized by continuously increasing of the confined modulus. Thus, it's relevant to know which level of stress will be reach during the construction stage in order to assess the precise soil deformation response. Called M' the modulus number ($M' = \Delta M/\Delta\sigma'$), this parameter shows values in the range of 10-15 and they slightly increase with depth.

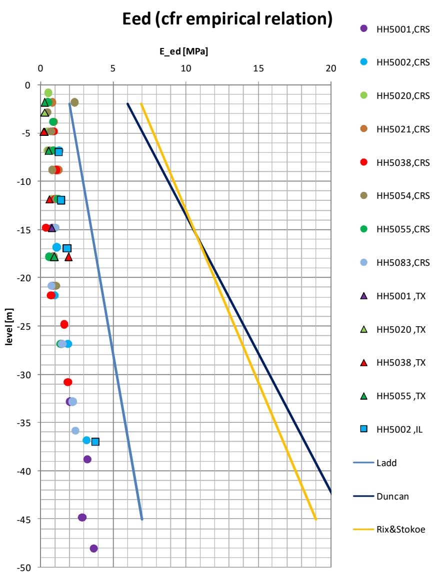

Figure 9 Variation of constrained modulus obtained by laboratory tests and empirical relations with depth

3.4.2 Constrained compression modulus – elastic deformation

Constrained compression modulus regarding elastic deformation can be evaluated through the CRS oedometer test. Swedish practice suggests to multiply M0- values obtained by CRS tests by a factor of 3-5.

3.4.3 Constrained unloading-reloading modulus

Constrained unloading-reloading modulus can be evaluated by means of IL oedometer test. Empirical relations are here presented in order to offer a comparison for the results:

$$M_{UL} = 10 * \sigma'_c * \exp\left(5 * \frac{\sigma'_v}{\sigma'_c}\right) \qquad (10)$$

where:
σ'_c = preconsolidation pressure
σ'_v = effective vertical stress
Karslsrud (2003) carried out oedometer tests in order to observe the unloading and reloading behaviour of clay. The tests concluded that the unloading modulus are dependent on the magnitude of the unloading and also the pre-consolidation pressure. The following formula was derived:

$$M_{UL} = 250 * \sigma'_v * \left(\frac{\sigma'_v}{\sigma'_c - \sigma'_v}\right)^{0.3} \qquad (11)$$

These empirical relations are compared with laboratory result. Eur increase almost linearly with depth from 9 MPa at 8 m to 55 MPa at 50 m depth. Empirical relations tend to overestimate the modulus.

3.4.4 Elastic modulus

The elastic modulus is estimated by means of elastic relations (v= 0.25) from oedometer and triaxial test results. It varies between 0.5 – 1.2 MPa in the first 20 m and later increase continuously to 2 MPa at 35 m depth.
In the Table 2 stiffness properties of the clay are reported.

Table 2 Summary of stiffness properties

Parameters' values variable with depth		CLAY Level [m]						
		-1÷-4	-4÷-19	-19÷-21	-21÷-25	-25÷-37	-37÷-40	-40÷-50
Constrained modulus Plastic deformation	M_L [MPa]	0.8	0.8÷1	1÷1.1	1.1÷1.4	1.4÷2.4	2.4÷3	3÷4
Constrained modulus Elastic deformation	M_0 [MPa]	12	12÷24	24÷26	26÷30	30÷38	38÷42	42÷50
Unloading/reloading stiffness	E_{ur} [MPa]	10	10÷30	30÷35	35÷40	40÷60	60÷70	70÷85
K_0 value for normal consolidation	K_0^{NC} [-]	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Preconsolidation pressure	σ'c [kPa]	95	95÷190	190÷210	210÷220	220÷280	280÷330	330÷400
Permeability	k [m/s]	$8*10^{-10}$	$8*10^{-10}$	$8*10^{-10}$	$8*10^{-10}$ ÷$4*10^{-10}$	$4*10^{-10}$	$4*10^{-10}$	$4*10^{-10}$

The low values of soil stiffness imply high deformations reached during the excavation stages, that turn into soil displacements and settlements. It is important to design stiff structures and to support them during the excavations with struts or slabs.
Properties of clay also require an accurate modelling, with a step-by-step numerical model, that is able to simulate the development of the deformations and the associated condition of the soil (e.g., active / passive) so to take into account the interaction between soil and structure and estimate correct actions on the structures.

3.5 Soil index properties

From routine test performed on samples extracted in the area of Rosenlund canal and Haga station, estimated values of density, water content, liquid limit and plastic limit are illustrated below. At each depth a number of one to three samples were analysed in the laboratory. The Figures from 20 to 25 show all of them.

3.5.1 Density

Density ranges from about 1.5 t/m^3 near ground surface to about 2.0 t/m^3 at a depth of about 50 m.

3.5.2 Water content

The water content slightly decreases with depth in the first 20 m (60-75%) and then a pronounced decrease from 70% to 40% in between 20-40m depth.

3.5.3 Atterberg limits

Atterberg limits have approximately the same trend of the water content. Liquid limit values comprised between 65÷80% in the first

20 m depth and the range shrinks and decreases with depth (wL from 70% to 40%), while the plastic limit have values around 30÷40% decreasing until 20% from 20m to 40m depth. Thus, the plasticity index (IP) oscillates around 35÷45.
The liquid limit is always slightly greater than the water content of about 5-10 points, therefore indicating a plastic soil (15 < IP < 40) and a liquid-plastic consistency (very small consistency index $IC = \frac{w_L - w_N}{IP}$, approximately 0 ÷ 0.25).

3.5.4 Soil sensitivity

Sensitivity of soil, evaluated starting from the Field Vane Test results, is an indication of the reduction in shear strength of soil when it is subjected to any disturbance. Indeed, it is defined as the ratio of the undrained shear strength of undisturbed soil to the undrained shear strength of remoulded soil at the same water content. According to Skempton et al. (1952) the clay layer in the studied area is extra sensitive (St in the range of 8 - 16) and medium sensitive (8 < St ≤ 30) according to Rankka et al. (2004).

3.5.5 Hydraulic conductivity

The soil's permeability is estimates by means of CRS test. It varies within the first 50 m in the range 3*10-9 – 3*10-10.

3.5.5 Pre-consolidation stress

Pre-consolidation stress is estimated from the IL test results by means of Casagrande method and from CRS test results by means of Sällfors method. They satisfactory highlight the same linear trend for the pre-consolidation stress increasing with depth from about 100 kPa at 2m depth to 400 kPa at 50m.
In Haga station area, soft clay is often slightly over-consolidated due to loading caused by soil weight itself. The OCR effect can probably not be explained by any preloading and another effect, such as delayed compression (creep) is probably the cause. For this reason, Karlsrud et al. (2006) talk about a sort of "apparent over-consolidation ratio". The estimated over-consolidation ratio (OCR) is decreasing with depth, starting from values 1.2-1.8 in the first 10 m to the range of 1.1-1.2 until 20 m depth. Normal consolidation conditions are expected at greater depths.

3.5.6 Pore pressures

Pore pressure measurements have been conducted at several depths (up to 45 m depth) with a piezometer (BAT system), a closed system, suitable for monitoring pore pressures in low permeable soils.
Pore pressure can be assumed to be hydrostatic, varying linearly with depth from 20 kPa (1m depth) to 470 kPa at 45 m depth.

4. NUMERICAL MODELLING AND CALIBRATION OF PARAMETERS

Specific soil models for the clay have been used for calculations by FEM analyses (Plaxis), both in undrained and drained conditions such as the NGI-ADP model and the Soft Soil Model.
Both models have been used since there is not a model able to properly reproduce both soil deformation and resistance.
Each model has been evaluated by a direct comparison between experimental and the data obtained by Plaxis "Soil Test" tool simulation of the same test.

4.1 NGI-ADP constitutive model

The NGI-ADP model was developed to properly capture the asymmetric behavior of soil when subjected to undrained extension or compression stresses, as the clay of Gothenburg. For this reason, this model can be successfully used for undrained ULS analyses.
All the parameters, evaluated from laboratory and field tests and chosen to fit the selected set of experimental data, have been applied as input in the numerical analyses. The failure shear strains are compared to a reference set of parameters reported by Kullingsjö (2007) used in a similar context for a typical excavation calculation. After calibration, model has been verified through Plaxis "Soil Test"simulation, based on available experimental tests (undrained triaxial tests, in compression and in extension and later direct shear tests). Generally, the simulated behavior in the plane q-εyy matches the experimental data with a satisfying level of accuracy, as shown in the following figures where the orange line refers to experimental data, and the blue line refers to simulated data using the NGI – ADP model (Figure 10).

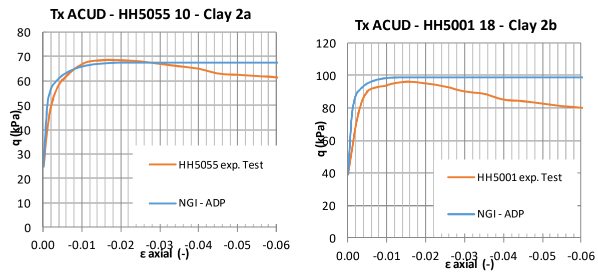

Figure 10 Test HH5055 (-6.9 m) and HH5001 (-15 m)– Orange line: experimental curve, blue line: NGI-ADP simulated curve

In order to verify the set of parameters used in the analysis in direct shear conditions, simulations of undrained direct shear tests are performed (blue line in the Figure 11) and then the results are compared with experimental tests (red line in the Figure 11).

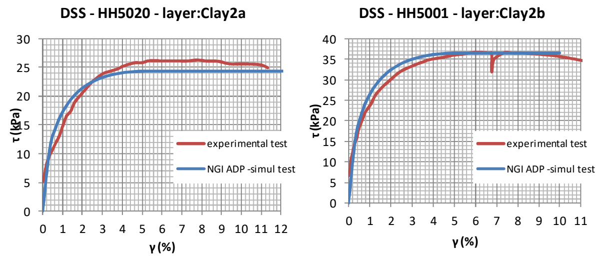

Figure 11 DSS Test HH5020 (-6 m) and HH5001 (-15 m)– Red line: experimental curve, blue line: NGI-ADP simulated curve

4.2 Soft soil constitutive model

The SS model seems to be suitable to model the deformation behaviour of Gothenburg clays. The effective angle of internal friction and the effective cohesion are established a-priori based on available data. Other parameters are set to obtain the best fitting of available data starting from the values estimated within the selected experimental set of data.
Compared to NGI-ADP, Soft Soil model doesn't consider the anisotropic soil behavior in undrained conditions, so it is mainly used to check the deformability of the structure in fully coupled flow deformation analyses.
All the parameters, evaluated from laboratory and field tests and chosen to fit the selected set of experimental data, have been applied as input in the numerical analyses. After calibration, model has been verified through Plaxis "Soil Test"simulation, based on available experimental tests (IL and later on CRS test and undrained triaxial tests).
Figure 12 reports the simulations in the lin-lin and lin-log plane. Simulations exhibit a behaviour very similar to experimental data in terms of stiffness in loading condition, as it is clear from the slope of the stress-strain curves and from the value of the tangent oedometer modulus measured between the pre-consolidation pressure and the limit pressures.
As an example, in Figure 13 tables are reported with the clay parameters, for two layers, used in Plaxis analyses for NGI-ADP and Soft soil constitutive models. Definition of used symbols can be found in Plaxis 2D manual.

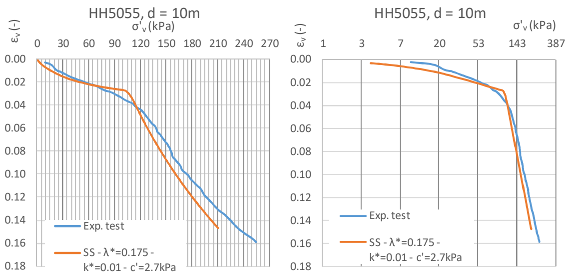

Figure 12 Simulated CRS curves compared with measured values for test HH5055, depth 10m. lin-lin and lin-log plane

ID Layer	[-]	Clay2b	Clay3
Level	[m]	- 12 ÷ -19	-19 ÷ -21
Gur/CuA	[-]	250	250
γf,C	[%]	2.0	2.0
γf,E	[%]	6.5	6.0
γf,DSS	[%]	5.5	5.0
Cu,A,ref	[kPa]	44.0	58.00
Cu,Ctx/Cu,A	[-]	0.99	0.99
yref	[m]	-12.00	-19.00
Cu,A,inc	[kPa/m]	2.00	2.00
Cu,P/Cu,A	[-]	0.57	0.56
Cu,DSS/Cu,A	[-]	0.73	0.72
τ0/Cu,A	[-]	0.72	0.73
OCR	[-]	1.08	1.06
k0,OC	[-]	0.52	0.52
K0,NC	[-]	0.50	0.50
v' (*)	[-]	0.30	0.30
γsat, Clay	[kN/m3]	16.30	16.30

ID Layer	Clay2b	Clay3
Level	- 12 ÷ -19	-19 ÷ -21
c (kPa)	3.7	4.16
φ' (-)	30	30
ψ (-)	0	0
κ* (-)	0.011	0.012
λ* (-)	0.24	0.28
vur (-)	0.15	0.15
K0NC (-)	0.5	0.5
OCR (-)	1.08	1.06
γsat (kN/m3)	16.30	16.30

Figure 13 Clay parameters for Plaxis NGI-ADP model (left table) and soft soil model (right table)

5. DESIGN CONCEPTS

5.1 Main geotechnical issues

As highlighted in Chapter 3, the design of underground structures in Gothenburg area, presents a number of geotechnical issues:

- High horizontal soil thrust, both in undrained and in drained conditions
- High soil deformability
- Stability of the embedded part of retaining structures
- Stability of bottom of excavations against heave
- Variation of pore pressures in clay and hydrogeological conditions in upper and lower aquifers
- Settlements, in short- and long- term of the structures
- Settlements of existing structures (buildings), both in short- and long-term conditions
- Interaction between the part of the station in clay and in rock

5.2 Design solutions

Design solutions can be defined in order to deal with the described geotechnical issues, which result in a design concept where the chosen systems are connected between each other to address the different requirements:

- Reinforced concrete Diaphragm walls can be used for the retaining structures, which are able to deal with high soil thrust and have the stiffness necessary to limit the deformations in horizontal direction as well as vertical settlements
- The horizontal support to the embedded part of the retaining structures and the stability against bottom heave in short-term can be obtained by creating a plug (jet-grouting, DSM – Deep soil mixing / Lime cement columns) or by using a system of concrete Cross-walls, that connect the walls on the opposite sides of the shaft
- The preservation of the water regime, especially within the transition layer between clay and bedrock, which is connected to the settlements of the buildings can be managed by preliminary sealing works: considering the low permeability of the clay, that doesn't need sealing, jet-grouting can be used to seal the material, mostly granular, in the transition layer, and low-pressure injections can be performed in the rock, to avoid water inflows through possible cracks
- The bearing capacity required to avoid absolute and differential settlements between the part of the station founded on clay and the one on the rock, can be obtained by vertical elements, such as Cross-walls or piles, extended down to the bedrock. Since the rock surface has variable slopes, it is necessary to guarantee their contact with the rock and the transfer of the resulting loads; this can be obtained, for example, by using steel-core piles drilled into the rock.

6. CONCLUSION

In this paper geotechnical properties of Gothenburg sensitive clays, have been described, with reference to performed investigations, tests and elaborations. It has then been presented how these data have been used for the definition of NGI-ADP and Soft soil constitutive models, that have been used in FEM numerical analyses. A summary of the main geotechnical issues related to the design in sensitive clays and of the main design solution has also been presented. Compared to to common approach to similar works, peculiar characteristics of the clay required deeper geotechnical investigation and analyses than in most underground stations' projects, as well as the use of sophisticated numerical modelling and calibration methods. Specific construction choices and details have also been introduced to cope with the difficult and demanding geotechnical conditions in Gothenburg's area.

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