Design and construction of a secant pile wall in Glasgow

Conception ét realisation d'une parois en pieux sécants á Glasgow

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ABSTRACT

The White Cart Water is a small river which runs through Glasgow. Heavy rainfall has caused numerous floods in the past. Work Section 16 is a 200 m long stretch in the southern part of Glasgow. For this section, a secant pile wall with an in-situ reinforced concrete wall on top has been proposed to form the flood defence wall. High differential water loads without anchoring possibilities, an existing wall which had to be maintained and the presence of tenement buildings at a short distance from the wall were part of the engineering challenges. Furthermore, the piled wall had to be constructed in difficult soil conditions consisting of overconsolidated clays underlain by sandstone and siltstone. A large soil investigation program was performed to obtain the required soil and rock parameters. For the latter parameter the methods as proposed by Hoek-Brown and Bieniawski were used. Construction of the secant pile wall was executed without problems and distress to the tenement buildings and existing wall.

RÉSUMÉ

Le White Cart Water est un petit fleuve qui traverse Glasgow. De violentes chutes de pluies ont par le passé provoqué des inondations. L'ouvrage 'Section 16' est une étendue de 200 mètres de long dans le sud de Glasgow. Pour cette section, la solution proposée contre les inondations est un mur de soutènement à pieux sécants en béton armée. De grandes poussées hydrauliques sans l'ancrage de mur, une digue existante à rénover aussi bien que la présence à une distance très proche de logements faisaient partie des défis techniques. De plus le mur à pieux devait être construit sur un sol inconvenable composé d'argile sur du sable. Une étude de sous-sol a été accompli pour obtenir les paramètres (géotechniques) requis sur le sous-sol et le rocher. Pour les paramètre de rocher plusieurs méthodes ont été utilisées : celle dite de Hoek Brown et de Bieniawski. La construction du mur à pieux sécants a été réalisé sans problèmes et sans dégâts pour les logements à proximité.

Key words : Secant, Pile Wall, Rock, Sandstone, Siltstone, Plaxis, Hooke-Brown, Bieniawski, Glasgow

1 INTRODUCTION

The White Cart Water has inflicted misery on residents and businesses in the south side of Glasgow over many decades. There have been numerous significant flood events over the last 25 years with several hours of heavy rainfall causing river levels to increase rapidly. Glasgow City Council has promoted a flood prevention scheme for the White Cart Water and its tributary the Auldhouse Burn. The Scottish Ministers approved the flood prevention scheme in 2006. The £53million White Cart Flood Prevention Scheme was a direct response to the flooding of the 1980s and '90s. Works in the south part of Glasgow commenced in January 2009 and are due for completion by October 2011.

VolkerStevin Ltd. were awarded the contract for construction of new or improved flood defences to a number of work sections through Cathcart, Langside, Shawlands, Pollokshaws, Auldhouse and Pollok. The contract includes the design and construction of a new retaining structure between the White Cart Water and tenement buildings on Cartside Street, Langside. The geotechnical design and construction support for this retaining wall, known as Work Section 16 (WS16), was performed by Volker Wessels Stevin Geotechniek.

Figure 1. Existing situation White Cart Water.

The following challenges had to be overcome in the design and construction phases:

- an existing flood defence wall which was in poor condition but had to be maintained;
- tenement buildings with 2 m deep basements at a distance of 6 to 10 m from the retaining structure;
- high differential water loads with no opportunities for anchoring systems, meaning the retaining wall had to be free standing;
- risks of flooding during execution of the works on the retaining wall;
- difficult soil conditions as described below.

2 SOIL INVESTIGATION

The original Soil Investigation (SI) information was not sufficient for detailed design. Therefore, additional SI was scheduled. Field testing was performed in March/April 2009 and consisted of:

- Cone Penetration Tests (CPT) to rockhead;
- boreholes including rock coring to a depth of 25 m below ground level;
- disturbed and undisturbed soil sampling;
- rock coring.

Laboratory testing was performed between April and July 2009 and consisted of:

- determination of unit weight and moisture content;
- grain size analyses on cohesionless samples;
- Atterberg limit tests on clay and silt samples;
- shear box tests on sand samples;
- Consolidated Isotropic Undrained (CIU) Triaxial testing on cohesive samples;
- Unconfined Compression Strength (UCS) and Point Load tests on rock corings.

Figure 2. Rock cores in laboratory.

As no undisturbed samples could be obtained in the overconsolidated cohesive layers, some of the laboratory testing was performed on remoulded soil samples.

Undisturbed soil samples and rock cores were taken at approximately 1 m intervals. Cores were visually inspected in the laboratory where a detailed testing program was generated.

A remarkable fact was that from the rockhead down to maximum investigated depth almost no weathering had occurred. Furthermore, RQD values were generally high, showing almost intact cores of sandstone and siltstone as can be seen in Figure 2.

3 SOIL CONDITIONS

3.1.1 General

The soil stratification consists of the made ground underlain by stiff to hard overconsolidated clay layers. From approximately 10 m depth rock strata consisting of weak to strong siltstones and sandstones as well as layers of coal were encountered.

Strength and stiffness parameters of made ground and dense sand and gravel layers were directly derived from CPT cone resistance. As the thickness of these layers and thus the influence on calculation results was limited, little effort was made in investigating the variation of these parameters.

3.1.2 Cohesive soils

Undrained shear strength values for cohesive soil are normally taken from laboratory triaxial tests and in situ tests (torvane tests, pocket penetrometer). Laboratory tests on overconsolidated layers tend to produce lower values due to sample expansion after extrusion from the samples tube. For this reason and the fact that few undisturbed samples were obtained by the SI subcontractor, the undrained shear strength was also derived from CPT cone resistance using the following equation:

$$c_u = \frac{q_{net}}{N_{kt}} = \frac{q_t - \sigma_{v0}}{N_{kt}} \quad (1)$$

Where:

c_u = undrained shear strength

q_{net} = net cone resistance

q_t = total cone resistance, corrected for cone shape and pore pressure u_2

u_2 = pore pressure, measured in the shoulder of the cone

σ_{v0} = effective overburden pressure

The value of N_{kt} varies usually between 15 and 20 and depends on the Over Consolidation Ratio (OCR) of the material. The value of OCR was derived using the SHANSEP method by Ladd et al [1]:

$$OCR = \left[\frac{(c_u / \sigma'_v)_{OC}}{(c_u / \sigma'_v)_{NC}}\right]^{1.2} \quad (2)$$

Where:

$(c_u/\sigma'_v)_{OC}$ = shear strength over effective soil pressure ratio for overconsolidated clays

$(c_u/\sigma'_v)_{NC}$ = shear strength over effective soil pressure ratio for normally consolidated clays

$\sigma_{v'}$ = effective overburden pressure

From this procedure it was concluded that the clay layers were highly overconsolidated (OCR values up to 3.0) and that a N_{kt} factor of 20 should be used.

The stiffness parameter $E_{u,50}$ was directly derived from the triaxial testing (partly on remoulded samples) and using the following correlation with strength and plasticity according to Termaat et al [2]:

$$E_{u,50} = 15{,}000 \cdot \left(\frac{c_u}{I_P}\right) \quad (3)$$

Where:

I_p = plasticity Index

4 ROCK PARAMETERS

4.1.1 General

Geotechnical stiffness parameters for the rock mass were derived from Hobbs's [3] modulus ratio graph and using different correlations based on the Rock Mass Rating (RMR) system after Bieniawski [4] as an input parameter. Strength parameters were obtained by different theories as shown below.

4.1.2 Stiffness parameters

Hobbs provided a graph, with linear relations, on which the rock stiffness can be determined based on the Unconfined Compression Strength (UCS). Instead of using only one parameter, the RMR system of Bieniawski is based on several parameters, e.g. UCS, spacing and condition of discontinuities, RQD etc. A comparison of these and other methods have been made for the rock strata in Glasgow.

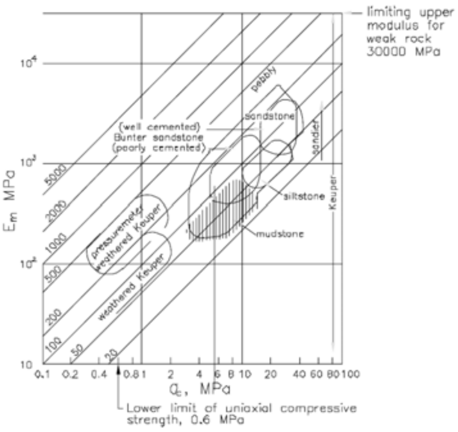

Figure 3. E_m as a function of USC by Hobbs.

Hoek-Brown's criterion [5] as shown below is applicable for $\sigma'_{ci} < 100$ Mpa:

$$E_m = \left(1 - \frac{D}{2}\right) \cdot \sqrt{\frac{\sigma_c}{100}} \cdot 10^{\left(\frac{GSI-10}{40}\right)} \tag{4}$$

Where:

σ'_{ci} = Unconfined compression Strength
GSI = Geological Strength Index
D = disturbance factor

Boyd, as described in [6] also found a correlation for low mean effective vertical in-situ stresses ($\sigma'_v < 10$ kPa):

$$E_m = \frac{RMR^4}{1000} \tag{5}$$

Alternatively, the equation of Serafim & Pereira [7] can be used:

$$E_m = \frac{10^{(RMR-10)}}{40} \tag{6}$$

The latter equation is valid for RMR values < 50.

Table 1. Data from core logging and laboratory tests

Rock	UCS [N/mm^2]	RMR [%]	GSI [-]	RQD [%]
Siltstone, med. strong	33	50	30	80
Silstone, weak	12	38	25	56
Sandstone, med. strong	30	45	35	70
Sandstone, weak	12	37	30	36

Table 2. Calculated values of E_m

Rock	Hoek-Brown	Hobbs	Serafim & Pereira	Boyd
Siltstone, med. strong	1840	1650	9800	6250
Siltstone, weak	830	600	5000	2000
Sandstone, med. strong	2330	2290	7500	4100
Sandstone, weak	1100	800	4700	1875

With this analysis a wide range of deformation moduli were found. Moduli based on Rock-Mass-Rating tend to overestimate the value, because Hobbs figure is typically based on UK weak rocks. On the other hand, CIRIA report 181 claims that Hobbs can be used for weak rock with UCS values lower than 0,6 MPa while Hobbs figure is based on UCS-values between 0-100 Mpa.

Because of the strict deformation criterion of the retaining structure Hobbs was adopted in the design of the secant pile wall.

The secant pile wall was designed by using a spring model, Msheet. The deformability parameter of the weak rock E_m, was correlated to a spring parameter k_h by using the following equation by Karlsrud [8]:

$$k_h = 3E_m \cdot \left(\frac{D}{3} + \frac{B}{2}\right) \qquad (7)$$

in which:

k_h = linear 1-dimensional spring
E_m = deformation modulus
D = retained height
B = excavation width (max 2D)

For verification purposes, the correlation was verified by the following method: a comparison was made between a Plaxis 2D and a Msheet model. Soil was modelled with one rock layer with equal strength parameters but different deformability parameters, k_h for Msheet and E_m for Plaxis 2D.

Results from the two calculations were very similar. Plaxis calculated a horizontal deformation at the top of the wall of 32 mm while the linear spring model, Msheet, calculated a deflection of 28 mm. With such close agreement between the results it was decided that the correlation could be used for the project.

4.1.3 Strength parameters

Similarly strength parameters c' and φ' of the rock material can be derived from the Hoek-Brown criterion and the RMR-system of Bieniawski.

Using the Hoek-Brown criterion, low initial stresses can be modelled using the slope-case. In this, the slope height H is equalized as the retaining height, 3m. With this, the RMR system can be compared to Hoek-Brown. Results of the parameters with both models are presented in tables 3 to 5.

Table 3. Bieniawski vs. Hoek-Brown (slopes)

Rock	c' [kN/m^3]		φ' [°]	
	RMR	Hoek-Brown	RMR	Hoek-Brown
Siltstone, med. strong	200-300	82	25-35	56
Silstone, weak	100-200	32	15-25	48
Sandstone, med. strong	200-300	86	25-35	64
Sandstone, weak	100-200	43	15-25	57

Table 4. Bieniawski vs. Hoek-Brown (general)

Rock	c' [kN/m^3]		φ' [°]	
	RMR	Hoek-Brown	RMR	Hoek-Brown
Siltstone, med. strong	200-300	1012	25-35	20
Siltstone, weak	100-200	330	15-25	20
Sandstone, med. strong	200-300	1390	25-35	31
Sandstone, weak	100-200	510	15-25	29

Table 5. Bieniawski vs. Hoek-Brown (tunnels)

Rock	c' [kN/m^3]		φ' [°]	
	RMR	Hoek-Brown	RMR	Hoek-Brown
Siltstone, med. strong	200-300	74	25-35	59
Silstone, weak	100-200	26	15-25	52
Sandstone, med. strong	200-300	72	25-35	66
Sandstone, weak	100-200	32	15-25	60

The large difference in strength parameters between the Hoek-Brown slope/tunnels and general model is caused by the difference in σ'_{3max}. In the "general" failure envelope range σ'_{3max} is equal to $\sigma_{ci}/4$. For the other envelope ranges σ'_{3max} is around 0.05 MPa, which is 100 times lower.

A retaining structure with a retaining height of 3m has a much lower σ'_3 compared to the previously mentioned envelopes based on Hoek-Brown. With this, it seems that all failure envelope ranges are not applicable for retaining structures with relatively small retaining heights and therefore parameter H

Comparison of Hoek-Browns 'general failure envelope' and Bieniawski's RMR method shows a major difference in cohesion values. Internal friction angles are in the same order of magnitude. Caused by a lack of further test results, the Bieniawski method was adopted for the strength parameters. Further investigation is proposed to explain these large variations.

5 GENERAL DESIGN

5.1 Introduction

As stated in section 1, several engineering challenges had to be dealt with. Figure 4 shows a typical cross section where important engineering outlines become clear. The minimum distance between the new wall and the tenement buildings is 6 m and the retained height is 4.4 m in the worst case. The lack of opportunities for anchoring governed the decision for a wall with a high bending stiffness.

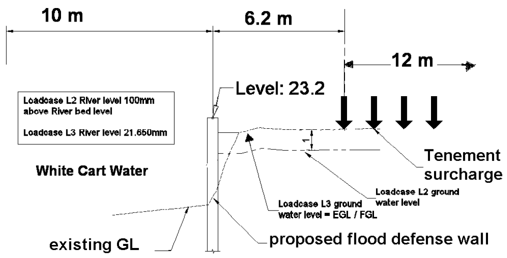

Figure 4. Typical cross section.

5.2 Wall type

Due to the difficult ground conditions, the application of sheet piles, as used elsewhere in the project, was problematic. Alternative options were therefore investigated. After due consideration a secant pile wall with an in-situ reinforced concrete wall above was chosen as the most appropriate and economic solution for providing the necessary water cut-off and retaining function.

At first, a secant pile wall, consisting of all concrete piles (so called 'hard piles') was considered. However, the potential risk of flooding and consequentially non-continuous work processes during construction asked for another solution. Pile hardening after waiting for several days for a flood to abate would make it impossible to drill the secondary piles into the initial piles.

For this reason, a so-called 'hard in soft' alternative has been designed. With this option, continuation of construction work is not dependant of the hardening speed of the concrete. In this construction type, cement bentonite material is used for the primary piles. Strength and stiffness of this material is less in relation to concrete, therefore drilling the infill pile after hardening can be easily done. Soft (primary) piles are drilled prior to construction of the hard (secondary) piles. Primary piles were executed as Continuous Flight Auger (CFA) piles, secondary piles were rotary bores piles.

Furthermore, durability considerations led to the final design: the so called 'hard in firm' solution, where primary piles consisted of low strength, slow hardening concrete. A special concrete mixture for this solution was designed by piling contractor Skanska Cementation.

Pile reinforcement was, according to common practice, placed in the secondary piles only and consisted of steel helical bars. The final secant pile wall configuration is shown in Figure 5.

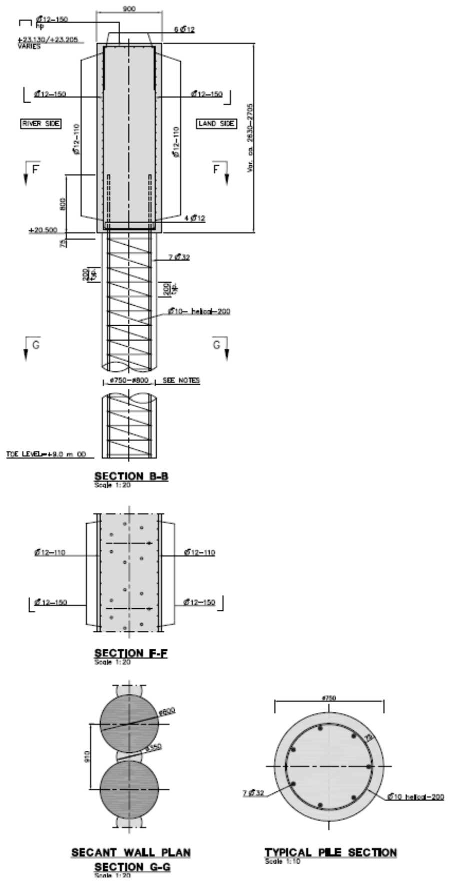

Figure 5: Final pile wall configuration.

6 CALCULATIONS

6.1 General

Design calculations were performed using the Dutch program MSheet, which is an elastoplastic spring model. By using the theory of Karlsrud, the stiffness parameter E_m was correlated to horizontal spring parameter k_h.

6.2 Finite Element Method

Additional Finite Element Model (FEM) calculations were undertaken using Plaxis 2D to establish the influence of the new retaining wall on the existing tenement buildings. The FEM modelling was also used to provide a better understanding of the soil-structure behaviour under different load conditions.

6.3 Evaluation of calculated deflections

Figure 6 presents a detailed overview of the vertical displacements of the tenement buildings. The maximum vertical displacement is approx. 5mm at basement level, just beside the wall.

The following parameters can be derived:

- the maximum rotation in the hogging zone of the shallow foundation (basement) is equal to 1:3,000.
- the angular distortion in the hogging zone is in the order of 0.5 x 10^{-3} and horizontal strain is lower than $0.3x10^{-3}$.

For evaluation of the movement of the tenement buildings, the design chart by Boscardin [9] has been used, see Figure 6. This procedure distinguishes the so called hogging and sagging zone, as demonstrated in Figure 7.

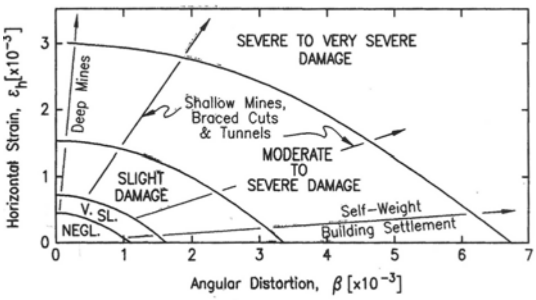

Figure 6. Design chart Boscardin (1989).

According to the design chart by Boscardin, this gives a negligible risk of damage to the tenement building. As the deformation is not critical, no further analyses were performed.

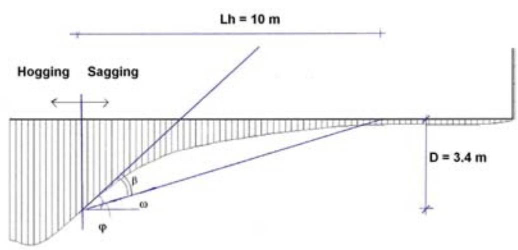

Figure 7. Hogging and sagging zone in deformation profile

7 EXECUTION OF WORKS

Piling works commenced in September 2010 and were finished in November 2010.

Figure 8. Execution of piling works.

Figure 8 shows piling works in progress. During pile execution, the White Cart Water was partly obstructed by a temporary working platform for the piling rigs. In this picture the distance to the tenement building becomes clear.

During piling works, the deformations of the tenement buildings were closely monitored as soil disturbance could occur especially with the execution of the secondary (CFA) piles. However, during construction works no deflections at all were measured, nor did any damage to the buildings occur.

8 CONCLUSIONS

For the design of a flood defence wall in Glasgow several methods were used to derive soil and rock parameters.

Absolute caution is required when determining rock parameters. Large variations in strength and stiffness parameters are possible due to different procedures. For this project, Hobbs equation was chosen for stiffness parameters in rock. Further investigation proved that the correlation by Karlsrud was applicable for the derivation of spring parameters as used in the Msheet model.

When deriving strength parameters using the Hoek-Brown method, great uncertainty is introduced on the choice of the so called general, slope or tunnel failure envelopes. For a retaining wall, the general envelope seems the best option, but still large difference was found in comparison to Bieniawski. The authors are currently undertaking further study of this matter.

Execution of the pile wall in difficult soil conditions, including several metres into intact sand- and claystone proved non problematic. The work was executed without deformation to the existing wall or tenement buildings.

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