In situ testing of barrette foundations for a high retaining wall in Molasse rock

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in situ testing of barrette foundations for a high retaining wall in molasse rock

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Barrettes are common foundations for high-rise buildings, especially because of their high bearing capacity, for vertical as well as for lateral loads. In this paper, it is shown how these were adopted and tested for anchoring a 27 m high retaining wall in rock on a slope located in the centre of Zurich. The monolithic $10 \times 1 \times 25 \text{ m}$ deep barrettes, excavated and built in layered rock (Upper Freshwater Molasse), are also intended to stabilise the slope. The main contribution to their bearing capacity is given by the contact strength between the reinforced concrete and the rock. Two full-scale pull-out tests, instrumented by distributed fibre-optical measurements, are carried out to investigate the contact strength between rock and reinforced concrete. The results show that dilatancy at the contact caused by the irregular excavated surface has a strong influence on the bearing capacity. Additionally, the distributed fibre-optic instrumentation allowed progressive crack propagation of the reinforced concrete in the barrettes to be detected during the test. Inverse analysis of the rock and concrete tensile properties are obtained within a finite-element calculation, which takes into account the non-linear properties of the reinforced concrete and the rock.

KEYWORDS: bearing capacity; design; diaphragm & in situ walls; excavation; finite-element modelling; full-scale tests

INTRODUCTION

ETH Zurich (Swiss Federal Institute for Technology) is building a new research centre for medical technology research and application. The building is located on a slope in the Zürichberg central city quarter (Fig. 1(a)). The buildings that were previously located on the site have been demolished. The new building now under construction is deeper, and on the north-east side it will have six storeys below ground level. As the structure is built on a slope, only two storeys will be below the surface on the south-west side.

The architectural design of the main building is limited by three main considerations. The construction site is located in a very prominent place surrounded by villas. The requirements of the neighbourhood and the law did not allow the building to be high enough to satisfy all the needs of the new laboratory and research building. The solution is to build deep into the ground, causing a one-sided deep cut in the slope. Additionally, the building owner wanted to avoid a solution with permanent anchors for reasons of maintenance and neighbourhood requirements. The anchors installed to retain the pile wall during excavation should be only temporary. The third important characteristic is the architecture and the requirement that the main building should have a light, flexible construction, allowing more space for teaching rooms and laboratories to be obtained. Therefore, it was important not to conduct any forces exerted by the slope through the walls of the structure, so it was necessary to design an independent retaining structure that was separate from the building. These three requirements together led to a unique slope stabilisation construction consisting of a retaining wall founded on barrette elements. In order to verify the bearing capacity of the foundation, two small-scale pull-out tests of test barrettes (TBs) were conducted. The results of these tests are extensively discussed in this paper.

GEOLOGY

The slope-stabilising wall is built into the Zürichberg hill (Fig. 1(a)). The slope has an inclination between 23° and 28° towards the south-west. The geological cross-section is shown in Fig. 1(b). The soil consists of an approximately 13 m thick moraine layer (USCS: SC-SM, GM), which accumulated through the movement of the ancient Linth glacier (lateral moraine). The upper part of the moraine is slightly less dense, owing to weathering processes. Large boulders can occasionally be found inside the soil silty-sand matrix.

The soil is underlain by the rock of the Upper Freshwater Molasse (moderately weak to moderately strong rock (Eurocode 7 (CEN, 2004)) in which excavation can be carried out by ripping), which consists of layers of sandstone (percentage by mass 43%, RQD 85–100%), siltstone (percentage by mass: 47%, RQD 85–100%) and marl including fine coal layers (percentage by mass: 10%, RQD 25–60%). The top layers of the rock present vertical joint discontinuities which are moderately to highly weathered.

Joints occur mainly in sandstone and siltstone layers; marl shows almost no jointing.

In the coal layers, small snail shells can also be found. The rock is layered in a more or less horizontal direction with a very slight dip. The marl layers represent a potential risk for slope stability, and their slippage needs to be prevented during excavation by suitable design of the retaining structure. This issue is addressed by the wall design described in the paper. A cross-section of the rock layers at the bottom of the pit (excavation trench of TBs) and pictures from a rock core drilling are presented in Figs 1(c) and 1(d). Boreholes instrumented with vibrating wire piezometers showed that groundwater is not present in the slope. However, water from precipitation can potentially infiltrate into the rock fissures,
and flow can take place over the impermeable rock surface. A drainage layer will therefore be installed on the temporary retaining pile wall in order to prevent development of pore-water pressure behind the permanent retaining structure.

The parameters for soil and rock obtained by extensive soil investigation (GLM Report, 2013) are shown in Table 1. No angle of dilatancy has been considered for the soil and the rock for the foundation design (conservative assumption).

**RETAINING WALL CONCEPT**

The permanent earth-retaining structure consists of a reinforced concrete wall (Figs 2(a) and 2(b)). The task of slope stabilisation is taken over by buttresses founded on barrettes. Reinforced concrete arches transmit the earth loads to the buttress like a multiple arch dam. The cavities between the arches and the rear side of the wall, directly supporting the earth, are adopted as vertical ventilation channels, which bring clean air into the building. In order to link the foundation to the wall, six pre-stressed cables are positioned inside the barrette piles. The cables are then conducted through the buttresses up to the top of the wall (Figs 2(a) and 2(b)). During construction of the wall, the cables are step-wise tensioned and the force in the anchors progressively removed.

The design of the steel cage was rather complex and is not described in this paper. The steel cages (25/C2/C2 1m, weight = 550 kN; see also Figs 3(a) and 3(b)) were built in one piece on site as monoliths on a steel table, which was later

**Table 1. Geotechnical parameters**

<table>
<thead>
<tr>
<th></th>
<th>Loose moraine</th>
<th>Dense moraine</th>
<th>Sandstone, moderately weathered</th>
<th>Sandstone, highly weathered</th>
<th>Siltstone</th>
<th>Marl</th>
<th>Upper Freshwater Molasse</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma$: kN/m$^3$</td>
<td>20</td>
<td>21</td>
<td>25</td>
<td>24-5</td>
<td>25</td>
<td>24</td>
<td>25</td>
</tr>
<tr>
<td>$\phi'$: degrees</td>
<td>32</td>
<td>36</td>
<td>38</td>
<td>34</td>
<td>32</td>
<td>22</td>
<td>28</td>
</tr>
<tr>
<td>$c'$: kN/m$^2$</td>
<td>0-3</td>
<td>0-3</td>
<td>0-3</td>
<td>0-3</td>
<td>0-3</td>
<td>0-3</td>
<td>0-3</td>
</tr>
<tr>
<td>$q_u$: MN/m$^2$</td>
<td>—</td>
<td>—</td>
<td>100</td>
<td>15</td>
<td>50</td>
<td>10</td>
<td>30</td>
</tr>
<tr>
<td>$E_{pl}$: MN/m$^2$</td>
<td>25</td>
<td>50</td>
<td>2700</td>
<td>2300</td>
<td>1400</td>
<td>460</td>
<td>2500</td>
</tr>
<tr>
<td>$E_{ur}$: MN/m$^2$</td>
<td>80</td>
<td>170</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The cohesion values for the rock are given for the direction orthogonal to the layers. The contribution of dilatancy was not considered.
lifted to put them in the upright position. A separate crane transported the cage to the excavated trench.

The design developed for the retaining wall was driven by the concept of clamping the foundation in the rock, while preventing the slippage of the rock layers. To prevent slippage and carry the enormous forces from the slope weight, a foundation structure with a large moment of inertia was realised. Fourteen very large barrette foundations were arranged as single elements, with their longest plan dimension parallel to each other at intervals of 7·15 m. The barrettes were placed with the direction of their largest horizontal inertia moment parallel to the slope (see also Fig. 2(b)). Inside each steel cage, six pre-stressed steel cables are positioned (see Fig. 3(b)). The pre-stressed cables have two main tasks: (a) reducing the wall bending and deformation (including concrete creep); (b) ensuring high enough tensile strength of the barrettes.

The geotechnical design was carried out using the finite-element (FE) code Plaxis (Plaxis3D, 2015). The results were validated against simple mechanical models, which resemble the bearing behaviour of standard retaining structures, such as a cantilever or heavy retaining wall (Figs 4(a) and 4(c)).

REVIEW OF RESISTANCE OF SHAFTS IN ROCK

The bearing capacity of shafts in rock has been extensively studied in the past. In the literature, several approaches for predicting shaft resistance of piles can be found. Well-known methods are based on empirical approaches in which the shaft resistance is generally described by empirical functions related to the uniaxial compressive strength of the material (Pells et al., 1980; Horvath et al., 1983; Rowe & Armitage, 1984; Reese & O’Neill, 1988; Kulhawy & Phoon, 1993). These approaches have also been modified with coefficients for taking into account socket roughness factors (Seidel & Collingwood, 2001). A different approach was followed by Serrano & Olalla (2004, 2006), who developed and validated a prediction method based on the Hoek and Brown failure

The analytical approach followed in this paper is the one described by Kulhawy & Carter (1992). They developed a method based on an elastoplastic solution, which takes into account the effects of cohesion, friction and dilatancy for the bearing capacity of shafts socketed into rock. The predictions resemble quite well the foundation bearing behaviour observed in this study, although, in the present case, a more complex interaction between the rock and the structure was observed, because of the non-linear behaviour of the reinforced concrete. Exhaustive interpretation of the test results was therefore carried out with non-linear FE analysis, which takes into account the observed non-linear behaviour of rock and reinforced concrete. Additionally, the traditional approach described by Fleming (1992) based on Chin hyperbolic functions was adopted to extrapolate the failure load from experimental results.

Generally, bearing capacity tests of barrettes are less common than pile testing, and not many examples can be found in the literature. A good example of a conventional head-down static load test on a barrette foundation can be found in Musarra & Massad (2015). For very high loads, static load tests are nowadays often carried out with O-cells (Osterberg, 1998; Fellenius & Ann, 2010). In the present study, the results and the interpretation of a conventional head-down pull-out test are presented.

IN SITU BEARING CAPACITY TESTS

As explained in the previous sections, the verification of admissible contact forces between the barrette and the rock is crucial to determine the safety of the retaining wall. Two in situ pull-out tests were therefore designed and built with the aim of determining the contact strength between the rock and the barrette. The two tests were carried out on opposite sides of the excavations (Fig. 5(a)). This allowed two different extremes of the geology to be studied and, thanks to the repetition of the test, allowed reliable results to be obtained. The TBs were built inside an excavated trench at a depth of between 9 and 12·5 m (Fig. 5(b)). The excavation for building the TB took place from the level corresponding to the head of the wall barrette foundation. Therefore, the vertical position corresponds to the location at which the highest shear force due to the rotation of the foundation is expected.

The trench was excavated with a Bauer trench cutter BC-40 and bentonite mud conveyor. The bentonite had a density of 1·05 kN/m³ and Marsh funnel time equal to 32 s. Two overlapping trenches of 2·8 m were excavated in order to obtain the desired size of 4·7 m (longest dimension). The same device and techniques were adopted for excavating the barrettes for the foundation of the retaining wall. The TBs were cast on 9 (TB2) and 10 October 2016 (TB1), and the tests took place on 21 and 23 November 2016, respectively.

The trenches were wider at the top, down to a depth of 9 m (4·7 m long and 1·2 m wide), whereas the trench where the TBs were cast had a length of 3·7 m and a width of 1 m (Fig. 6(a)). The TBs had a depth of 3·5 m, a length of 3 m and a width of 1 m. A 35 cm wide separation was realised between the shortest side of the TBs and the rock by adopting sheet piling steel profiles during casting. These were removed after the concrete had cured. Therefore, the TB was only in contact with the rock on the longer side of its horizontal cross-section. This particular geometry of the excavation trench was realised in order to prevent wedging of the wall during the pull-out.

The TBs were pulled out through eight GEWI bars (dia. 63·5 mm), which were anchored at the bottom of the steel cages with steel plates. The barrettes were additionally reinforced with horizontal and vertical steel rebars with a diameter of 20 mm at 25 cm spacing. Each GEWI steel bar was connected on the top to a hydraulic press (eight presses in total) which was able to apply a maximum theoretical load of 2 MN (total maximum load 16 MN) (Fig. 6(b)). The maximum applicable load was limited to 12 MN because of the steel frame adopted.

Most of the bentonite mud had been removed before the test was carried out, in order to observe the roughness of the excavated surface and the quality of the rock, leaving only approximately 2·5 m of the bentonite suspension above the upper side of the barrette. The reason for not removing all the bentonite was that rock blocks and debris were falling into the excavation, and there was concern about damaging...
Fig. 4. Simple mechanical model for validating the FE calculation: (a) cantilever beam model; (b) mechanical model for obtaining required necessary rock–concrete contact strength for the stability; (c) heavy wall model and equilibrium of the external forces

Fig. 5. (a) Positions of the two tests in the excavation pit and (b) vertical cross-section of the TB

Instrumentation

The installed instrumentation is listed below.

(a) Eight load cells placed on the GEWI steel profiles (precision 1 kN) for measuring the total load.
(b) One linear variable differential transformer (LVDT) on a GEWI profile for measuring the strains in the steel.
(c) Eight LVDTs for measuring the horizontal deformation of the guide walls.
(d) Two LVDTs on two Tremie pipes, fixed on the reference beam, for measuring the uplift of the TB (see also Fig. 7(e)).
(e) Two surveyor’s levels (one of them fully automatic), precision 1/100 mm. These devices allowed measurements of the movements of the steel frame, the concrete basements and the reference beam displacement.
(f) Two fibre-optic cable loops inside the TB (BRUsens BSST-V9: armoured fibre-optic strain sensing cables with central metal tube protecting one tight-buffered optical fibre, structured polyamide outer sheath, allowing a typical measurement strain range up to 1%).
(g) Oil pressure sensor inside each hydraulic press.

A closer look at a part of the LVDTs adopted for measuring the uplift is given in Fig. 7(e). The most difficult challenge
for the instrumentation was to measure the vertical movement of the TB. Since the expected displacements were very low, the precision of the measurement had to be on the order of 1/100 mm. On top, the uplift had to be measured at 9 m depth directly on the surface of the TB. Therefore, it was decided to put two heavy and rigid tremie pipes on top of the TBs and to measure their vertical movement during the test. This simple solution allowed a highly stable measuring framework. As the tests were carried out at night, the temperature was fairly constant and had no influence on steel tube elongation. Additionally, in order to verify the vertical displacements measured on the tremie pipes, the displacements of the upper part of the GEWI steel profiles were measured geodetically. These are of course affected by the steel elongation induced by the tensile load. After the steel strains induced by the load are subtracted in the measurements, both observations provided exactly the same displacement of the barrette.

The fibre-optic cables used were interrogated by Brillouin optical time-domain analysis (BOTDA; for the related literature, see Horiguchi et al. (1989), Niklès et al. (1997) and Niklès (2007)) and swept wavelength interferometry (SWI) measuring technology (for the related literature, see Froggatt & Moore (1998) and Gifford et al. (2007)). Both technologies adopt the principle that the temperature or strain in a fibre-optic cable can be obtained from the interaction between laser light and the glass fibre. The BOTDA measuring technique requires the application of two light sources of different types – a pumping pulse (the pump) and continuous-wave light (the probe) – to the cable ends. The interrogator (Ditest from Omnisens) measures the backward Brillouin frequency shift generated by the single pulse light when interacting with the continuous wave light, which varies depending on the changing strain and temperature in the cable. The SWI technology allows measurements only from one end of the cable. The interrogator (OBR4600 from Luna) uses SWI to measure the Rayleigh backscatter as a function of length in the optical fibre. Changes in strain or temperature cause time and spectral shifts in the local Rayleigh backscattering. The OBR interrogator measures these shifts and scales them to give a distributed temperature or strain measurement.

The main macroscopic difference between these two technologies is the resolution and sensor length: SWI (OBR4600 device) provides a resolution of 5 mm and a maximum sensor length of 70 m. With BOTDA (Ditest device), the resolution drops to 1 m, but the sensor can be up to 30 km long. The commercial cables used here were developed by Iten (2011) in the framework of a dissertation at ETH Zurich, Switzerland. Technical details of implementation of the SWI technology for geotechnical problems can be found in Hauswirth (2015).

Test results
The TB was loaded step-wise up to a maximum load of 12 MN. The measured displacement for each loading step is plotted in Fig. 8(a). The waiting periods after each loading step were determined according to the requirements of the geotechnical Swiss standard SN SIA 267-1 (SIA, 2013b). In accordance with the standard, the time-dependent displacement at constant load is plotted on a semi-logarithmic diagram (Fig. 8(b)). The gradients of the plotted curves represent the creep index (units: mm). If the displacement increment between 5 and 15 min is less than 0.2 mm, the observational period should be at least 15 min. Otherwise, the loading step should last until the displacement curve approaches a straight line. If the inclination of that line (the creep index) is lower than 1, the next loading step can be applied. The failure load is determined when the creep index is equal to 2 mm.

The displacement between 5 and 15 min was higher than 0.2 mm only in the last loading step (12 MN). After
Fig. 7. (a) Steel cages of the TBs and the eight GEWI bars used for the uplift. (b) Surface of the excavated rock after bentonite removal. (c) Casting of the barrette. (d) Loading frame and load cells. (e) Details of the LVDTs for measuring the uplift of the TB.

Fig. 8. (a) Load–displacement curve for the two tested barrettes, TB1 and TB2. (b) Time history of the displacement at constant load for TB1.
A steep increase in the displacement took place, whereas later the curve stabilised, approaching a straight line with inclination (creep index) almost equal to 1 mm. The fibre-optical measurements showed that this increase was most probably due to the non-linear behaviour of the concrete.

Both tests were terminated after 12 MN load without reaching failure (creep index = 2 mm), because, owing to the bearing limitations of the steel frame, no additional load could be applied.

An interesting insight into the problem of soil–structure interaction is given by the fibre-optical measurements. Fig. 9(a) shows the development of the strains (measured with SWI technology) in the TB during the last four loading stages. The dashed straight lines represent the upper and lower sides of the wall. It can be clearly observed that between 5400 and 7600 kN, the strains in the upper part of the barrette increase significantly.

In the same load range, the rate of displacement of the TB increases (Fig. 8(a)). Although the displacements are relatively small, the peak strains measured with the SWI measuring technique reach remarkable values of approximately 2000 με. These measured strain values correspond very well to the theoretical values that are obtained if only the GEWI steel bars are stretched under the load (GEWI steel area $A_{GEWI}=25.335 \text{ mm}^2$, tension in the steel at load $N=12 \text{ MN}$, $\sigma_t=474 \text{ N/mm}^2$, steel strain $\varepsilon_{GEWI}=2255 \mu$).

The measured strain peaks with the SWI measuring techniques correspond to the strain as if only the steel is withstanding the load. Therefore, it can be concluded that the peaks in the strain correspond to cracks in the TB, as the concrete resistance makes no contribution to reducing the strain in the GEWI bars. Generally, it can be clearly seen that the area in which cracks are observed increases as the load becomes higher, and it reaches half of the vertical length of the TB at 12 MN load. Fig. 9(b) shows the deformation of the TB obtained by integrating the strains, which reached 1.6 mm at 12 MN. Since the final uplift of the wall was 4.6 mm, the total final heave of the bottom of the TB is about 3.0 mm.

Comparison between the measurements obtained with the SWI and BOTDA technology (Figs 9(a), 10(a) and 10(b)) shows that the latter is unable to detect the position of the cracks in the TB because the strains are measured over a length of 1 m, and not a few millimetres, as with SWI.

Originally, only BOTDA measurements were planned, because the non-linear behaviour of the concrete was not expected and the Ditest device was considered more robust for field application. However, as the measured strains in the TB2 were much higher than would be predicted considering only the rock–concrete interaction and linear concrete behaviour, it was also decided to use the SWI interrogator OBR4600 to have a more in-depth look into the structure (TB1). The failure load was expected to be around 7–8 MN maximum, whereas in reality it was found to be more than 12 MN (failure load was not reached in the test). As explained in the next sections, the high bearing capacity is due to dilatancy at the contact between rock and concrete.

In the following section, the test is modelled with the FE code Abaqus, which allows a better understanding of the soil–structure interaction during loading. The results are validated against the closed-form solution for the load–displacement behaviour of a shaft pull-out test in rock according to the method of Kulhawy & Carter (1992). The extrapolated failure load is determined by adopting a hyperbolic curve (Fleming, 1992).

MODELLING

Finite-element model

An FE model of the rock and the TB was built with the FE software Abaqus (Abaqus 6.14 (Abaqus, 2014)). Taking advantage of the problem’s symmetry, only one-half of the TB and the rock was considered in the model (Figs 11(a) and 11(b)). The symmetry axis was positioned in the middle of the TB and cut its shortest side.

The model depth is 20 m, the length is 9.6 m and the width is 7.5 m. The dimensions were optimised to reduce the model size in order to prevent the boundaries affecting the results. The GEWI steel bars (four out of eight) and the vertical...
The rebars of the steel cage were modelled as volume elements inside the TB. The shape of the steel bars was chosen to be quadratic instead of their true circular shape in order to reduce the number of volume elements.

Linear elements of type C3D4, a four-node linear tetrahedron, were adopted. Preliminary analysis had shown that the use of quadratic elements produced less accurate prediction of the cracks inside the element. The mesh size of 6.25 cm (for linear elements) corresponds to one-quarter of the horizontal rebar spacing. To study the influence of the mesh size, models with coarser mesh were also tested (12.5 and 25 cm). The effects of element size are discussed later.

Constitutive models

The rock was modelled as a homogeneous medium with a linear elastic constitutive model and non-associated Mohr–Coulomb failure criterion. This strong simplification of the layered rock mass was necessary to obtain a homogeneous and simple bearable value of the contact strength for the design. No interface was modelled between rock and concrete, and therefore full bonding was assumed.

The horizontal mesh element size at the contact between the TB and the rock was chosen to be the same size as the difference between the wider top and narrower bottom excavated portions of the trench (10 cm). In this way, the shear distribution in the elements at the contact is optimised and the elements represent a kind of volume element interface (see also Fig. 11(b)). The dilatancy, which in the analysis is considered a homogeneous rock property, is in reality a property of the rough contact between rock and concrete. This simplification is justified by the good reproduction through the FE analysis of the observed behaviour of the TB during the tests. The dilatancy of the ‘glued’ rock elements at the contact with the TB reproduces well the increase in contact strength due to the rough contact.

For modelling the mechanical behaviour of the concrete, the concrete damage plasticity model (CDP) was chosen (Tao & Chen, 2015). The parameter values for the compressive strength, stiffness and density of the adopted concrete (C30/37) were chosen according to default values and are shown in Table 2. The choice of default constant values is motivated by the fact that compressive (shearing) failure is not relevant for the current boundary value problem, since failure takes place only in tension. Only the tensile properties of the concrete are back-calculated in order to fit the experimental results (see next section).

The CDP model implemented in Abaqus allows yielding in the concrete to be modelled: hardening in compression before failure and stiffening in tension after failure. The model is formulated in the framework of plasticity theory, taking into account the effects of the cracks by means of a damage parameter, D, which reduces elastic stiffness after yielding occurs. The Abaqus formulation allows the user’s own choice of hardening and softening (stiffening) rules to be implemented.

The adopted tensile strength and post-peak behaviour of the concrete (obtained with inverse analysis of the experimental results) is described by equation (1). The pre-failure behaviour ($\varepsilon_t < \varepsilon_{cr}$) is modelled as linear elastic, while the post-failure softening is predicted by the power load...
model proposed by Belarbi & Hsu (1994) and Belarbi et al. (1996).

\[
\sigma_i = f_{ct} \left( \frac{\varepsilon}{\varepsilon_{ct}} \right)^m
\]

This equation is valid for \( \varepsilon_i \geq \varepsilon_{ct} \), where \( \varepsilon_{ct} \) is the critical tensile strain.

In equation (1), \( \sigma_i \) is the predicted stress, \( f_{ct} \) and \( \varepsilon_{ct} \) are the tensile cracking stress and strain, \( m \) is the exponent which regulates the drop rate of the strength against the strains and \( \varepsilon_i \) is the total strain.

The damage evolution (according to Lubliner et al. (1989)) is calculated according to the following equation

\[
D = 1 - \frac{\sigma_i}{f_{ct}}
\]

This equation is valid for \( \sigma_i \geq f_{ct} \).

**Forward analysis**

The FE analysis was carried out in two steps. In the first calculation step, a gravity load is applied to the whole model, which takes into account the rock, the excavation and the TB. This first step simulates: (a) the excavation under bentonite; (b) the casting of the TB under water; and (c) dredging of the bentonite. This simplified procedure was adopted to allow a faster inverse analysis, after preliminary analysis proved that the three phases could be modelled in a single step without significantly affecting the results of the next pull-out step. In a second step, the pull-out was carried out.

A simple forward analysis adopting the analytical solution proposed by Kulhawy & Carter (1992) was also carried out.

**Inverse analysis**

The inverse analysis was carried out by varying the elastoplastic properties of the rock (elastic modulus, cohesion, friction and dilatancy angle) and the tensile strength \( (f_{ct} \) and \( m \)) of the concrete. In the analytical solution, only the already mentioned rock elastoplastic properties were varied. The initial values were those from the geological investigation described in Table 1.

The goal was to obtain the best possible match between measured vertical displacement (analytical and FE) and strains of the TB (FE) for each applied loading step. The analysis was carried out manually rather than with an optimisation algorithm.

The parameter variation was simplified by considering the following important aspects.

**Rock properties:** the rock elastic properties mainly affect the vertical strains in the TB before the cracks occur and the inclination of the load–displacement curve before yielding. The rock cohesion influences the yielding load, while friction and in particular dilatancy regulates the gradient of the load–displacement curve after yielding (straight line for analytical solution).

**Concrete properties:** in the numerical analysis, the term \( f_{ct} \) influences the crack initiation; therefore, it was varied to obtain crack initiation in the model at the same load level as in the experiment. The parameter \( m \) strongly influences the spatial development of the cracks, because it regulates the rate of strength drop after the failure occurs. The exponent \( m \) was therefore varied to obtain a good agreement between calculated and measured cracked area for each load level.

**Inverse analysis results**

The inverse analysis was first carried out by varying the material parameter for obtaining the best fit between measured and calculated load–displacement (uplift) curve. The focus here was to determine the material properties of the rock.

The best-fit load–displacement curves obtained with the inverse analysis and FE as well as the analytical approach are shown in Figs 12(a) and 12(b), respectively. The analytical and the numerical calculations were both carried out with the same rock and (elastic) concrete properties.

The back-calculated rock properties obtained with the inverse analysis are shown in Table 3. Not surprisingly, the parameter values obtained with inverse analysis adopting the analytical and the numerical solutions are the same: the analytical Kulhawy and Carter approach represents the closed-form solution of the adopted FE analysis when the mechanical behaviour of the concrete is linear.

It can be observed that the analytical solution predicts the load–displacement behaviour of the TB during the test very well, if the 1·6 mm final displacement due to the cracks in the concrete is taken into account (Fig. 12(a)). As there is no cut-off deformation value for dilatancy, the FE model is not able to predict the failure. Failure is predicted instead with a simple hyperbolic curve (Fig. 12(b)).

The failure load predicted with the formula proposed by Fleming is 15,000 kN. The derivations are given in the Appendix.

The model is capable of reproducing the acceleration in the displacements, which takes place between 5400 and 7600 kN.

The second part of the inverse analysis was carried out by varying the concrete tensile properties to obtain the best fit between measured and calculated strains (Figs 13(a)–13(c)) as well as the deformation of the TB (Fig. 13(d)). The back-calculated concrete tensile strength and softening parameter \( m \) are equal to 2 N/mm² and 0·9, respectively.

Figure 13(a) shows the calculated and measured vertical strains in the TB for the last two loading steps, before the first cracks occur. The analysis was carried out with the optimised parameters from the back-calculation of the load–displacement curve.

The size and the area of extensions of the experimental and calculated strains match quite well, although the predicted strains at the bottom of the TB in the first loading steps are negative. The strains in the FE calculation are negative (in Abaqus, compression is negative) because of the heave at the bottom induced by the excavation. In the measurements, the strains at the bottom lie on the reference point and they are set equal to zero.

Figure 13(b) shows the strains measured in the four vertical segments of the fibre-optical cable loop plotted against the
calculated strains. It can be observed that the spacing and distribution of the measured strain where cracks occur highly depend on the cable position. The comparison between measured and calculated strains at two representative loading stages during which cracks occurred is shown in Fig. 13(c). Fig. 13(d) illustrates the very good agreement between integrated and measured vertical displacement of the TB.

The main advantage of the FE compared to the analytical approach is therefore the ability to predict the non-linear behaviour of reinforced concrete and the transition zone between initial and full slip at the contact.

Sensitivity analysis and discussion

The analysis allows the crucial contribution of the contact dilatancy for the bearing capacity of the barrette to be identified. If the dilatancy is not taken into account in the analysis, the failure takes place immediately after the contact reaches its maximum strength, long before the maximum applied load can be reached (see Fig. 12(a)). The development of the calculated cracked area has been found to be highly influenced by the post-failure behaviour of the concrete. The failure of the TB under tension is prevented only by the dilatancy, which allows strength increase during yielding.

Table 3. Back-calculated rock properties

<table>
<thead>
<tr>
<th>Material</th>
<th>( \rho ): kN/m(^3)</th>
<th>( E_r ): MN/m(^2)</th>
<th>( v_r )</th>
<th>( \phi' ): degrees</th>
<th>( \psi ): degrees</th>
<th>( c' ): kN/m(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock</td>
<td>27</td>
<td>2000</td>
<td>0-2</td>
<td>32</td>
<td>10</td>
<td>280</td>
</tr>
</tbody>
</table>

Fig. 12. Back-calculation of displacements. (a) Finite-element simulation and measured displacement for TB1 for three different mesh sizes. One curve shows the results from a calculation carried out without taking into account dilatancy. (b) Analytical prediction of the displacement according to Kulhawy & Carter (1992) for the elastic and plastic slip. The prediction of the displacement and failure load according to Fleming (1992) is also plotted.

Fig. 13. Back-calculation of the strains: (a) measured and back-calculated vertical strains before the cracks occur; (b) measured (four measured segments, S1 to S4) and back-calculated strains for the final loading step of 12 MN. (c) Measured (S1) and back-calculated strains for 7-6 and 12 MN loading stage. (d) Comparison of the integrated calculated and measured strains (displacement). L, load level; LE, linear element
The simulated development of the yielding in the rock in the model is shown in Figs 14(a)–14(d). Comparing Figs 13(c) and 14(c), it can be observed that extensive cracking develops once all the rock elements at the contact reach yielding, which happens at approximately 7.6 MN load (60% of final load = 12 MN). This value also corresponds to the failure load for the design pressure of 360 kPa and lateral surface of $2 \times 3 \times 3.5 = 21$ m$^2$.

The increase in the deviatoric strength can be clearly observed in Figs 14(e) and 14(f). Fig. 14(e) shows the stress–strain behaviour in a rock element (integration point) at the contact with the rock. Owing to the effects of the dilatancy (increase in confining pressure), after yielding occurs (deviatoric stress $q = 600$ kPa), the stress keeps increasing as the deviatoric shearing strain gets higher. Fig. 14(f) shows the stress path followed in $p'–q$ space, where $q$ is the deviatoric stress and $p'$ is the mean (effective) total stress at the same integration point. The influence of dilatancy after yield occurs can be clearly observed.

Thanks to the dilatancy, catastrophic failure of the contact is prevented, and the plastic strain areas in the rock continue to increase until the final loading step of 12 MN, at which failure still does not take place.

Figs 14(g) and 14(h) show the tensile damage inside the TB for the two last loading states. These areas with reduced stiffness reproduce the effect of the cracks in the concrete and the increase of deformation due to the reduced stiffness of the barrette.

**Quality control**

As it is well known that the results of a CDP model are mesh-dependent (Tao & Chen, 2015), the influence of the mesh size for the linear elements was studied, and the results are shown in Figs 12(a) and 15(a).

The mesh size has almost no influence on the load–displacement curve (Fig. 12(a)), but affects the prediction of the maximum strains inside the TB (Fig. 15(a)). This is not surprising, as the influence of the crack is calculated in the model as increased strains, and because the strains are localised, these become higher the smaller the mesh size becomes. However, the fibre-optic sensors also allow the strains to be measured over a length of 1 cm and do not directly measure across the crack width.

The development of the cracked area and its extension is not affected by the mesh size in the chosen range. Good agreement between predicted and measured strains is obtained on reducing the mesh size to 6.25 cm. The fracture energy is an important indicator for assessing the correctness of crack prediction in concrete, in particular when constitutive models based on the damage concept are adopted. This should lie in a realistic range.

The calculated fracture energy in a portion of the damaged area is shown in Fig. 15(b). The calculated energy in this cracked TB is 206 N/m, which is in very good agreement with the typical values for the adopted concrete calculated according to the Fédération Internationale du Béton (fib, 2013).

**Fig. 14.** Yielding in the rock elements at loads of: (a) 3.2 MN; (b) 5.4 MN; (c) 7.6 MN; (d) 12 MN (quadratic element mesh). The different symbols show the different load levels for the point of contact where the stress–strain paths in (e) and (f) are calculated. The simulated cracks in the TB are shown in (g) and (h) for two representative loading stages.
SUMMARY AND CONCLUSIONS

An innovative retaining wall design was carried out for stabilising a deep cut in a slope in the city centre of Zurich. The free-standing buttress wall was anchored to 14 barrettes with pre-stressed cables. The barrettes were positioned longitudinally in the slope in the direction of their largest static height. FE analysis showed that the crucial contribution for the stability of the structure was given by the contact strength between the rock and the concrete of the barrettes. Therefore, two pull-out tests were carried out on two TBs of reduced dimensions built in front of the future foundation where the highest contact shear stress was expected. Besides conventional instrumentation, distributed fibre-optic sensors were also used. These allowed the non-linear behaviour of the reinforced concrete to be detected qualitatively and quantitatively during the test. These experimental results were crucial for the interpretation and analysis of the test results.

With the experimental results, an inverse analysis of rock properties was carried out. The contact strength adopted for the wall design was confirmed by the tests and the analysis that were carried out. The excellent professional conduct and support of the company Bauer, in particular Niklas Haag and Andreas Simson, were essential for building and carrying out the complex tests. The company Marmota Engineering AG, in particular Frank Fischli, is gratefully acknowledged for instrumenting the TB and analysing the experimental data. Bernhard Trommer, Fred Baumeyer and Theo Keller from Basler & Hofmann AG also made a substantial contribution to the design of the wall. Melanie Prager is gratefully acknowledged for support in FE modelling of the reinforced concrete for the inverse analysis. The authors are also grateful to Hansjörg Gysi for the careful review of the project.

APPENDIX

The inverse analysis of the rock properties is validated adopting the analytical approach described by Kulhawy & Carter (1992). This method allows the displacement of a test shaft to be obtained directly from the elastoplastic rock properties, the elastic concrete properties and the applied load. The original contribution describes the full meaning and derivation of the parameters, which is not repeated in this Appendix.

The mechanical properties of the concrete and dimensions of the barrette are as follows

\[
\begin{align*}
\ell &= 3 \text{ m} \\
b &= 1 \text{ m} \\
t &= 3.5 \text{ m} \\
E_c &= 33 000 \text{ MN/m}^2 \\
\nu_c &= 0.15 \\
A_c &= lb = 3 \text{ m}^2
\end{align*}
\]
The adopted rock properties are described in Table 3. Because the formulas are only valid in a circular pile, the barrette will be transformed into an equivalent circular pile

\[ A_1 = 2l = 6 \, \text{m}^2/\text{m} \]

\[ A_2 \text{ is the area of the barrette wall skin surface; } l \text{ is the length of the barrette wall.} \]

This leads to an equivalent diameter of a pile

\[ d_{\text{equiv}} = A_1/\pi = 1.91 \, \text{m} \]

This leads in turn to an equivalent cross-sectional area of the pile

\[ A_{c,\text{equiv}} = d_{\text{equiv}}^2/4 = 2.864 \, \text{m}^2 \]

which leads to an equivalent Young’s modulus

\[ E_{c,\text{equiv}} = E_c A_2/\pi = 34 \, 557 \, \text{MN/m}^2 \]

Now the equations of Kulhawy & Carter (1992) can be applied.

\[ \zeta = \ln \left[ 5(1 - \nu)l/d_{\text{equiv}} \right] \]

\[ \lambda = E_{c,\text{equiv}}/G_t \]

where \( G_t \) is shear modulus of the rock mass.

The elastic uplift can be obtained by adopting the following equation

\[ E_{c,\text{equiv}} w_u = \frac{1}{2} Q_u \left( 1 - \frac{E_c}{E_{c,\text{equiv}}} \right) \frac{2}{\mu d_{\text{equiv}}} \cosh(\mu t) \]

This allows the theoretical displacement straight line for the full slip uplift to be obtained (see also Fig. 12(b)). As shown in Fig. 12(b), the results from the analytical solution are very close to the displacements obtained from the measurements (without the contribution of the cracks in the TB).

The failure load is predicted by matching the measured displacement with those predicted by the following greatly simplified hyperbolic equation (Fleming, 1992)

\[ \Delta_S = \frac{M_d d_{\text{equiv}} P_u}{U_b - P_u} \]

where \( \Delta_S \) is the pile head displacement; \( M_d \) is a constant (according to Fleming, \( M_d = 0.0005 \) for very stiff soils); \( d_{\text{equiv}} \) is the equivalent diameter = 1.91 m; \( P_u \) is the current load level; and \( U_b \) is the failure load = 15 000 kN.

**NOTATION**

\( A_e \) area of concrete barrette

\( A_{c,\text{equiv}} \) area of equivalent circular pile

\( A_{\text{GEW1}} \) area of GEW1 steel profiles

\( A_s \) skin surface of barrette

\( c' \) effective cohesion

\( D \) damage

\( d_{\text{equiv}} \) diameter of equivalent circular pile

\( E \) elastic Young’s modulus

\( E_{A1} \) soil active earth pressure

\( E_{A2,H} \) horizontal component of active rock pressure

\( E_{A2,V} \) vertical component of active rock pressure

\( E_c \) elastic Young’s modulus of concrete

\( E_{c,\text{equiv}} \) elastic Young’s modulus of equivalent circular pile

\( E_{P2,H} \) horizontal component of passive rock pressure

\( E_{P2,V} \) vertical component of passive rock pressure

\( E_{pl} \) primary loading stiffness

\( E_r \) elastic Young’s modulus of rock mass

\( E_{u1} \) unloading–reloading stiffness

\( E_0 \) earth pressure at rest

\( f \) ratio of biaxial to uniaxial compressive strength

\( f_{ct} \) critical tensile stress (uniaxial)

\( G_t \) elastic shear modulus of rock mass

\( G_1 \) weight of the wall

\( G_2 \) weight of the barrettes and the rock in between

\( K \) second stress invariant ratio

\( L \) load on the pile

\( M_1 \) tangent of the load at the origin of the hyperbolic function

\( m \) exponent of concrete softening power law

\( N \) tensile load

\( P_S \) shaft load

\( p' \) triaxial volumetric stress invariant

\( Q_u \) uplift force

\( q \) triaxial deviatoric stress invariant

\( q_u \) uniaxial compressive strength

\( R_{S1} \) radius of the equivalent rotating mechanism

\( R_1 \) rock reaction on the steel concrete edge

\( R_2 \) rock reaction at the bottom of the barrette foundation

\( S \) resultant of the barrette skin friction

\( S \) depth of equivalent pile

\( U_b \) ultimate shaft friction

\( w_u \) uplift deformation of base of test barrette

\( \gamma \) dry density

\( \Delta_S \) settlement of shaft head

\( \varepsilon_{ct} \) critical strain (uniaxial)

\( \varepsilon_{\text{GEW1}} \) strain in the GEW1 steel profiles

\( \varepsilon_{u} \) deviatoric strain

\( \varepsilon_t \) total tensile strain (uniaxial)

\( \zeta \) geometry factor

\( \lambda \) pile/soil stiffness ratio

\( \nu \) Poisson’s ratio

\( \nu_c \) Poisson’s ratio of concrete

\( \nu_r \) Poisson’s ratio of rock mass

\( \rho \) density

\( \sigma_t \) tensile stress (uniaxial)

\( \phi \) friction angle

\( \psi \) dilatancy angle
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IN SITU TESTING OF BARRETTE FOUNDATIONS FOR HIGH RETAINING WALL