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Observational Method, Case A2 Maastricht

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Abstract. The A2 tunnel in Maastricht has been built within a dry building pit with a maximum depth of 22 meters with sheet pile walls, suspended in a cement bentonite trench supported by struts at 2 to 4 levels. Dewatering is done by deepwells. The excavation reaches into Limestone layers so the maximum mobilized passive resistance of the sheet pile wall is depending on the strength parameters of these layers. In the design stage of the project uncertainties occurred about both strength parameters and permeability of the Limestone for a large part of the project. These uncertainties could only partly be reduced by additional soil investigation and in-situ tests, but they were vital for the design of the retaining walls. To combine economical implementation of the project with a very favourable risk profile the "Observational Method" was adapted. This article deals with the background behind the variations in soil conditions and the residual uncertainties. The elaboration of the Observational Method is explained, including the measurement results and the decisional system during the construction. Finally the results of the adaption of this method in the project will be evaluated. In May 2014 the building pit was fully excavated, so final conclusions are drawn.

Keywords. Observational Method, building pit, Limestone, cohesion, passive resistance, permeability, monitoring, dewatering, uncertainty soil conditions, variation in soil conditions, risk profile, economic design.

1. Introduction

The highway A2 in Maastricht is an important route for traffic passing Belgian and German borders. The highway runs straight through the city of Maastricht which causes traffic jams and separates the city into two parts (see also figure 2). Ballast Nedam and Strukton (Avenue2 consortium) are currently building project 'De Groene Loper' which is tackling the traffic issues around Maastricht and melting the city of Maastricht into a unified habitat. Part of this project is the construction of a double deck tunnel underneath Maastricht (see also figure 1). This tunnel is very unique and will be the first double deck tunnel in Europe which is open to all traffic.

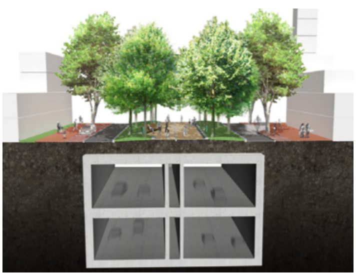

Figure 1. Artist impression.

The main part of the tunnel has been built within a cut and cover dry building pit of approximately 16 m deep and 30 m wide. The phreatic groundwater table is approximately 3 m below surface level. A typical cross section is given in figure 3.

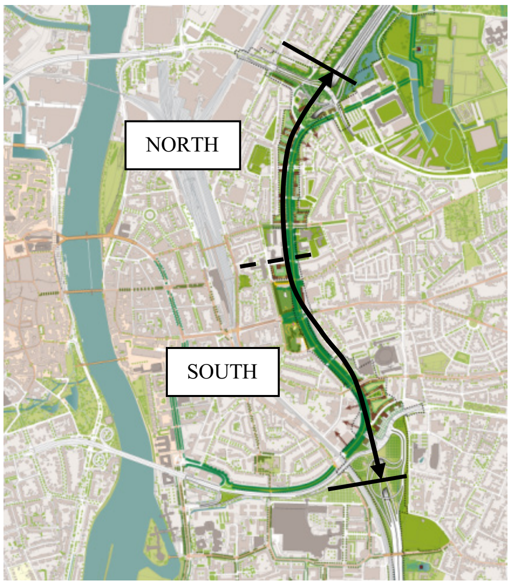

Figure 2. Location of tunnel in Maastricht.

The project has been used by the Geo-Impuls workgroup "Observational Method" as an example. Geo-Impuls is a five year long, joint industry programme which aims at reducing geotechnical failure substantially in 2015, Cools (2014).

2. Soil Investigation and Design

As shown in figure 3 the soil consists for a large part of Limestone, a soil type which is, in the Netherlands, exclusively present in the Limburg province. Subsequently, Dutch experience with constructing in this soil type is limited, which is why experts from Belgium and Germany were consulted during the design process and why also part of the soil investigation was carried out by a German consultant.

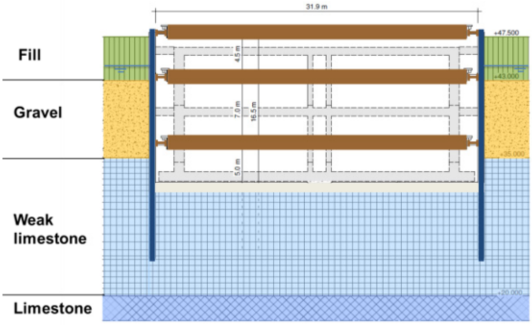

Figure 3. Cross section building pit, including temporary struts and the final construction (dotted lines).

During and prior to the design phase, an extensive soil investigation has been performed in two phases. The soil investigation consists of boreholes including laboratory tests, CPTs and SPTs. Additionally Geophysical tests have been performed to increase knowledge of layer separations and anomalies, such as fractures and Karst phenomena. Due to the local presence of relatively hard layers of rock (flint) within the Limestone, CPTs could not be carried out on all locations.

Based on soil conditions, the tunnel trajectory can be divided into a northern and southern part of about equal sizes. Climatological conditions in the period the Limestone was shaped, caused the Limestone in the Northern part to be less cemented than in the Southern part. Especially the top meters of Limestone of the Northern part are very weak due to weathering. Consequence is that the passive soil resistance, which can be mobilized by the retaining walls, in this part is less than for the Southern part. The lack of passive resistance is especially important for the design of the sheet pile walls during the deepest excavation phase of the building pit.

The strength of the Limestone has mainly been determined with one-axial unconfined compression tests. For the relevant top layers, in these tests, UCS values of 0.03 to 1 MPa were found for the Northern part and 1 to 8 MPa for the Southern part. Figure 4 shows the relation between saturated volumetric weight and UCS value for both parts.

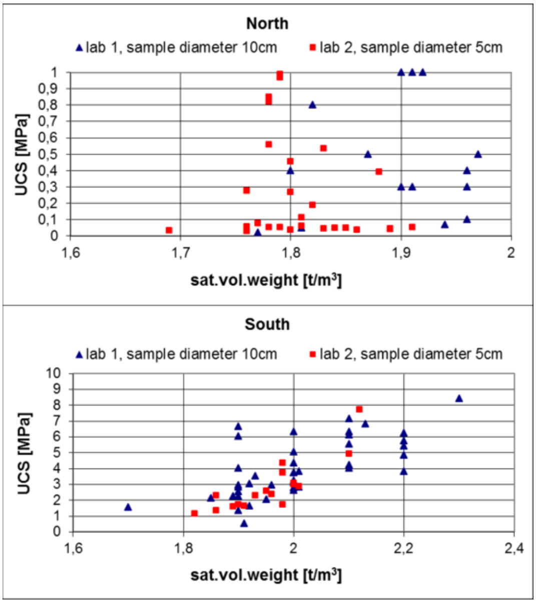

Figure 4: UCS Test results on Limestone for the southern and northern part. Mark the difference in scale.

Figure 4 demonstrates that a clear relation between UCS value and saturated volumetric weight can be derived for the Southern part but not for the Northern part. Remarkable however is the fact that there seems to be a link with the laboratory which performed the tests. Laboratory 2 finds many values <0.1MPa, laboratory 1 hardly any.

Apart from several FEM analyses, most of the design calculations of the retaining walls have been done with the 'Beam on elastic foundation' model, which also uses a Mohr Coulomb failure condition. For the design, the UCS values have therefore, based on Tirant (1994), been translated into values for internal friction Φ' and cohesion c' (see also table 1).

Table 1: Cohesion (kPa) Limestone used in design (characteristic values), northern part; for all cases the angle of internal friction is 32.5°.

Layer	Thickness	laboratory 1	laboratory 2
Top	1 to 5 m	20	8
Medium	5 to 15 m	40	8
Deep	-	80	80

Two possible causes for the extreme difference between both laboratories were considered: First, the fact that laboratory 2 has trimmed the samples to a relatively small diameter before performing the UCS tests, could have damaged the samples and lead to lower results. However it is also possible that laboratory 1 has tested a non-representative selection of samples, because of the fact that stronger samples are more easily treated to test and/or the samples of the weakest limestone got lost during the drilling process. Comparison with additional in situ measurements (CPTs) has shown that the first explanation is the most plausible, indicating that the results of laboratory 1 would be most representative. However the second cause could not be fully excluded, leading to the conclusion for the design, that a large uncertainty in strength had to be taken into account.

Another issue that influences the passive resistance of the retaining walls is related to the geohydrology during construction. To provide a dry building pit, drainage consisting of deep wells, placed on both sides within the building pit have been applied. Prior drainage tests with tracers indicated a ratio between horizontal and vertical permeability of 5:1. This large ratio ensures almost hydrostatic water pressures as function of depth on the passive side of the retaining walls. If the ratio between horizontal and vertical permeability would locally be lower, this would lead to a more progressive increase of water pressures with depth, thus reducing the passive resistance for the retaining wall. Environmental issues blocked the possibilities for extended investigation on this issue in terms of additional drainage tests.

Both uncertainties mentioned: The strength (cohesion) of the Limestone and the water pressures at the passive side, may have a negative impact on the passive resistance of the wall. It was decided not to base the calculations of the retaining walls on the worst case situation, but working out the Observational method for this case.

3. Observational Method

The Observational Method is a design method which does not deal with the uncertainty of the subsoil by assuming the worst case scenario and applying the full safety factors. Instead the performance is extensively monitored during construction, and for all foreseeable, but uncertain events, a follow up scenario with mitigation measures is present. These mitigation measures will be applied when and where necessary. In this way, the most economical solution in terms of desired reliability level and investment can be achieved. The moment of taking measures is determined by signal (S) and intervention (I) values.

The case in which no additional measures are needed, is based on the situation where only one of both uncertainties (strength of the soil or water pressure in the passive zone) would be unfavorable at the same location.

Since the passive resistance cannot be measured directly, it was decided to monitor the indirect parameters that change with a decrease of the passive resistance significantly:

- *Monitoring of pore pressure below the excavation level;*
  Since too high pore pressures in the passive zone are a major cause for a reduced passive resistance (especially in case of low cohesion for the Limestone), the effectiveness of the dewatering is monitored by monitoring wells.
- *Monitoring of strut forces;*
  If the strength (cohesion) of the Limestone is lower than expected, the force in the lower strut will exceed the expected values, with increasing excavation. This strut layer is provided

with strain sensors. Since the normal force in the circumference of the pipe may vary due to moments, four strain sensors are spread over the circumference. The S- and I-values are graphically presented in figure 5.

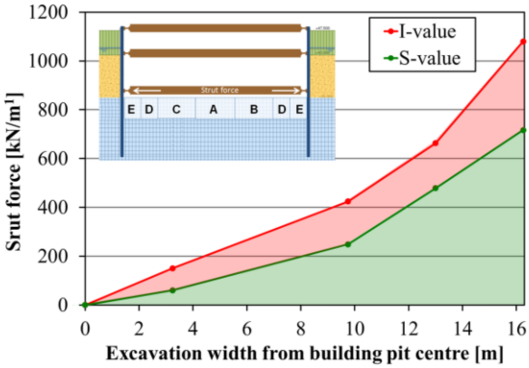

Figure 5: Excavation scheme and pre-calculated force in the deepest strut.

- *Monitoring deformation of the sheet pile foot;*
  By monitoring the deflection of the sheet pile wall (from excavation level to bottom sheet pile wall), an indication is received if passive collapse occurs. The foot of the sheet pile wall will undergo a displacement when the passive resistance is too low. The monitoring is performed with inclinometers mounted to the sheet pile wall.

The monitoring data is guarded by S- and I-values that are sent by text message and email.

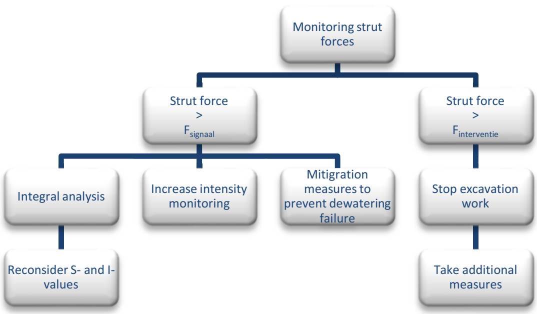

Figure 6: Example decision scheme Observational Method

Important part of the Observational Method is an elaboration of mitigation measures when exceeding the limits (see also figure 6):

- *Increase intensity monitoring;*
  The intensity of the monitoring is increased from discontinuous to continuous registration, to gain more accurate information about the situation.
- *Increase capacity dewatering;*
  By increasing the capacity of the dewatering, the water pressure in the passive zone is lowered to increase passive resistance.
- *Partially refill the building pit with gravel;*
  A 2 m thick layer of gravel is applied on the excavation level to stabilize the building pit.

In case of the last mitigation measure, instability of the building pit (or an undesirable level of safety against instability) is prevented, but to be able to continue construction of the tunnel, one of the following additional measures is taken:

- *Placing an additional strut layer*
  An additional strut layer, to be placed underneath the lowest strut layer.

- *Injecting the Limestone.*
  If the strength of the Limestone is extremely low (cohesion around 0 kPa), the ultimate measure is to strengthen the Limestone by injecting it to create an underground strut. In this case, it is important that the injection layer may not be closed completely, otherwise there is a chance that uplift will occur.

Since the passive resistance is influenced by various factors and it can be measured only indirectly, always an integral analysis of the situation is carried out when exceeding the S- and I-values.

4. Observations during Construction

The excavations in the Observational Method were carried out between October 2012 and May 2014. Before each excavation step (see A,B,C,D and E in figure 5) and before and after each working day the geotechnical site engineer evaluated the monitoring results in relation to the established S- and I-values and inspected the building pit together with head groundwork. Based thereon an advice is given to the project leader and head foreman who made a decision regarding the next execution scenario of the building pit.

During construction, some adjustments had to be made to the working plan. Originally the only dewatering elements were the deep wells. To remove effectively the large amount of internal water, released by the Limestone during excavation, additional surface drainage had to be used, in combination with excavation under a gradient. Without these measures clogging of the deep wells occurred, due to the inflow of water, polluted with fine Limestone particles, directly from the building pit floor.

The monitoring systems are extensively described by Galenkamp (2015). The water pressures were continuously monitored in periscopes and, especially in the first excavation stages, some of these pressures exceeded S- and sometimes even the I-values. The maximum observed horizontal sheet pile wall deflection at the excavation level, was very low: 10 mm, where 40 mm was expected for the best case scenario regarding the strength of the Limestone. The continuously monitored strut forces mainly increased during the excavation of part E (see figure 5). After part E is excavated, the forces remained equal; no time effect (consolidation or creep) is observed. Maximum values of about 40% of the expected values were observed.

Originally, the mitigation measure in the case of high water pressures (exceeding the I-value) was to install extra deep wells. Based on the low measured strut forces and sheet pile wall deflections however, it was decided to make a new integral analysis of the situation first. The combination of high water pressures with very low strut forces were in fact an unforeseen scenario. This integral analyse included the back calculations of the strength (cohesion) of the Limestone on the basis of all the monitoring data. The conclusion was that the cohesion of the Limestone is significantly higher than the assumed values, and therefore the higher water pressures could easily be accepted.

5. Back Analysis

When approximately 25% of the construction was finished, a comprehensive back analysis has been performed and reported, Servais et al. (2014). This back analysis consists of recalculations with the 'beam on elastic foundation' model and FEM calculations, taking into account the as-built information. Based on the recalculated strut forces it is concluded that the cohesion and stiffness of the Limestone are at least 3 to 5 times the design values for the best-case scenario (see also figure 7). However, even assuming these higher values, the calculated sheet pile wall deflection in that case is still about 10 times higher than the observed values.

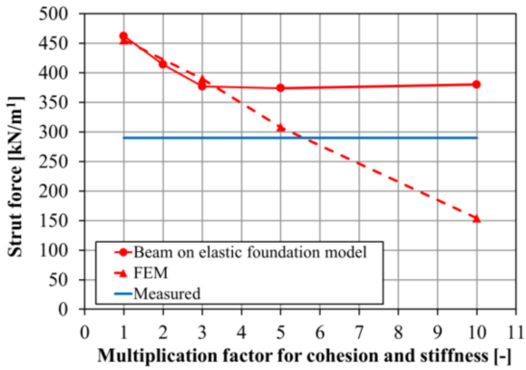

Figure 7: Recalculated strut forces for one location.

Based on these intermediate results, the question came up, if adaption of the Observational Method for the remaining part would be appropriate. New consideration of the existing soil investigation however, learned that for the remaining part to be constructed, soil conditions could be much worse. Therefore, in spite of the good results based on the monitoring and back calculations, it was decided to execute the remaining 75% also in the regime of the Observational Method. However, few optimisations were made in the working plan, i.e. less, and therefore larger excavation parts and less evaluation moments. Also, for the remaining part A (see figure 3) was excavated before, instead of after the installation of the deepest strut layer. This adaption gave a better control of the dewatering and the groundwork could be performed more efficiently.

6. Conclusions

Use of the Observational Method can lead to an economical solution with a very favourable risk profile.

In the case of A2 Maastricht, the Observational Method has proven to be an efficient way to deal with uncertainties regarding the cohesion and permeability of the Limestone. More generally, for cases in which soil investigation cannot take away all uncertainties in soil parameters, using the Observational Method should be considered.

Additional back analysis and evaluation during the construction process helped to improve and economise the working method.

The monitoring has proven that the cohesion of the Limestone in the passive zone is at least 3 to 5 times higher than assumed in the original design.

The most likely cause for the extreme difference in results from UCS tests has been the effects of trimming of samples on the strength.

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