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Risk Evolution during the Design of the Adaptation of a Railway Bridge Foundation

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Abstract. During a design process the risk profile of a project continuously evolves. Often a project progresses through a diamond shaped risk envelope. The initial concept is simple, but during the engineering process several risks are defined and mitigated, making the design increasingly complex. With additional investigations and calculations, some of the identified risks can be better defined. As a result, the design should evolve into a matured design in which only a few risks remain. These risks should be manageable. If uncontrollable risks remain, even though the chance of occurrence may be small, this could render the project unfeasible. This paper will show how the risk profile of a bridge foundation adaptation has evolved.

Keywords. risk evolution, diaphragm walls, risk management

1. Introduction

The Department of Waterways and Public Works (Rijkswaterstaat) wants to broaden the river Waal near Nijmegen (figure 1). The existing flood plain north of the summer bed of the river will be excavated 10 m to provide for a permanent channel parallel to the primary river bed.

The railway bridge across the river (founded in 1876) has footings in the flood plain and on both banks of the primary river bed. The foundation footings of the pillars in the flood plain needed to be adapted as the existing foundation level is 3 m above the future river bed level.

The concept of the adaptation evolved from a simple rigid box around the footings, to a more and more complex structure with soil improvement, anchoring and pre-stressing. Each of these mitigating measures introduced new risks that could have been mitigated again, making it even more complex. About halfway the design process it was decided to not further mitigate risks, but to try to eliminate uncertainties causing the presumed chance of occurrence of the calamity. It turned out that by using additional soil investigation and 2D and 3D finite element calculations, most of the formerly assumed risks actually could not occur or that the actual results of a calamity were easier to deal with than to eliminate them in advance. Some of the mitigating measures even turned out to be causing more risks than the risk they were intended to mitigate.

The final concept returned to something very close to the original idea: a rigid box around the footings to fixate the soil underneath and beside the raft foundation. After the design evolution one had a better grasp of where the critical phases in the project occurred, what dimensions were really needed in order to cope with the loads imposed by the bridge and to handle the location specific conditions in which the construction had to be built. The paper will explain how the design evolved and why design decisions were made and justified.

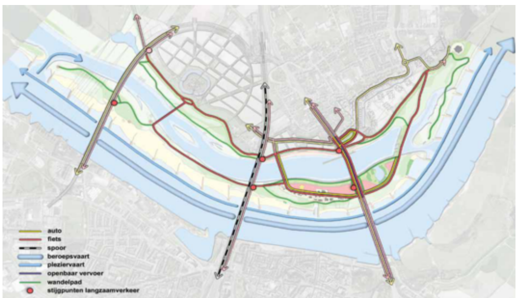

Figure 1. Project area

Initially, maintaining the existing pillars was considered too risky. Building new pillars however was expected to cost much more and more importantly, came out to have a similar or even less favorable risk profile than maintaining the existing pillars. Especially the necessary phased construction with temporal support below the stressed concrete beams of the bridge was expensive and had a high chance of unfavorable side effects.

The available space for the adaptation was limited, because of the overhead bridge and the requirements of the Department of Waterways. Each pillar was assigned a maximum width of 16 m.

During construction, rail traffic should not be disrupted and the adaptation should be ready in time for the works on the side channel to commence as planned. During and after construction, the bearing capacity and the stiffness of the foundation should remain (at least) the same. The stiffness requirement resulted from the train braking forces exerted on the bridge deck. The braking forces account for the majority of horizontal forces on the pillars.

2. Project Characteristics

The surface level in the flood plain has a mean height of +10,5 m above sea level. The top of the foundation slab starts at +8 m above sea level. The foundation slab ends at +5 m above sea level. The future profile of the side channel is projected at + 2 m above sea level (figure 2). The width of the foundation slab is 10 m, the length is 20 m.

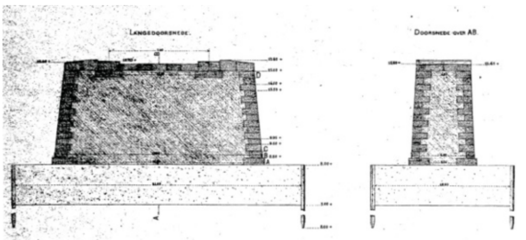

Figure 2. Cross sections of the bridge pillars

The soil profile mainly contains very dense sand (up to 50 MPa cone resistance) and gravel layers. Only the top layer and a layer around -5 m to -8 m below sea level consist of clay (figure 3). Because of the short distance to the river bed, the groundwater table around the foundation slabs is equal to the river level, with only a few hours delay.

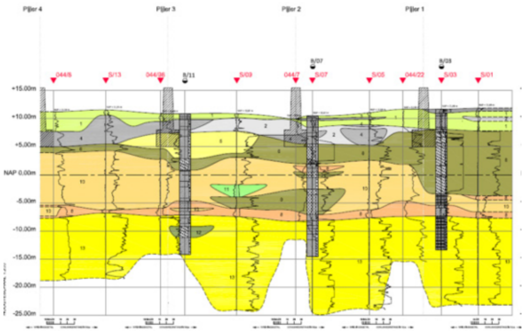

Figure 3. Geotechnical profile

As can be seen in figure 4, pillars 1 to 4 are located in the flood plain. Only pillars 1 to 3 need adaptation. The current summer levee including the part of the flood plain with pillar 4 will be included in a future island between the main and side channels of the river.

Figure 4. Flood plain before adaptation

Figure 5 illustrates the future view from the north bank of the side channel, looking towards the south-west. The three adapted pillars can be seen in the water, pillar 4 can be perceived on the island, just left from the center of the frame.

Figure 5. Side channel after adaptation

3. General Design Concept and Risk Evolution

Provided the requirements and characteristics stated above, the basic design consisted of diaphragm walls around the existing raft foundation. The diaphragm walls, together with a reinforced concrete top cap connecting all diaphragm wall panels and surrounding the masonry pillar (but not rigidly connected to the pillar), will provide an in-situ formed rigid box. Inside this box all soil particles around the raft foundation will be locked up, while allowing the masonry pillar to rotate like before. The rigidity of this box must be high enough to keep the stresses around the raft foundation equal to the existing conditions.

A diamond shaped risk evolution in 5 levels is shown in figure 6.

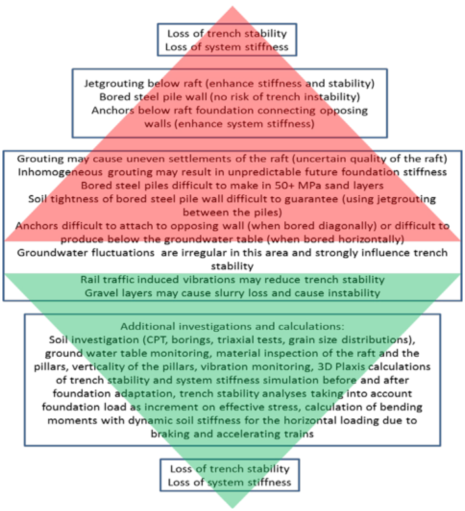

Figure 6. Risk evolution

- On the top level, the main initial risks can be found.
- On the second level there are mitigating measures.
- On the third level new risks are introduced, caused by the mitigating measures.
- On the fourth level uncertainties are eliminated with additional investigations and calculations.
- The final level only shows the remaining risks that now have a much friendlier profile due to broad investigation and consideration.

4. Risk Reducing Elements

4.1. Type of Wall

Diaphragm walls were considered ideal because of their high stiffness and vibrationless execution. Because of the high loads (around 350 kN/m^2) below the raft foundation, trench stability was assumed to be (almost) impossible to achieve. This was confirmed with preliminary calculations, taking into account the negative effect of the foundation loads on the trench stability. The positive effect of higher effective stresses due to the foundation loads on the stability was not yet implemented.

Because of the presumed stability issues, alternatives were considered:

1) Jet grouting the soil below the raft, increasing strength and stability but introducing the chances of new (jet grout) obstacles within the perimeter of the trench. It was considered difficult to achieve a homogeneous quality of grout and the jet grouting could cause uneven settlements of the raft foundation. Investigation of the assumed 3 m thick concrete slab was considered necessary.
2) Bored steel pile wall with jet grout closing the gaps between the piles. The steel piles would not cause stability problems like the diaphragm walls, but other risks emerged which were just as dramatic: soil tightness (jet grout filled gaps between the piles), introducing the piles in very dense sand (50+ MPa cone resistances), erosion protection and durability of exposed steel / jet grout wall.

More detailed stability calculations with Plaxis 3D revealed very decent levels of trench stability. Regular DIN 4126 trench stability calculations taking into account the positive effect of the higher effective stress due to foundation loading resulted in the same stability levels as obtained in the Plaxis 3D calculations. Because of the permeable soil and the trench length of only 15% of the foundation length, this was considered legitimate.

After investigation on the raft foundation, the risk of jet grouting in the vicinity of the foundation slab was considered worse than the risk of settlements due to excavation for the diaphragm walls.

Based upon the interpretation of the soil investigation, jet grouting would not add significant stiffness to the already very dense soil.

It was decided that diaphragm walls without soil improvement would be safely achievable.

4.2. Research

Based upon the findings of the preliminary design calculations, additional research has been executed.

1) The quality of the concrete foundation slab was essential for all further design considerations. Material inspection has shown that the assumed concrete was actually more like dense gravel with only locally some bonding. Material strength and stiffness were nowhere near what nowadays is considered as ‘concrete’.
2) Loss of slurry in coarse grained layers can cause instability of the trench. Additional grain size distribution investigation took place on the identified gravel layers. The coarse grained layers contained enough fines not to cause a risk of slurry loss.
3) Additional tri-axial tests were executed to improve the reliability of the strength and stiffness parameters of the soil.
4) Groundwater table variation has been analyzed and monitored.
5) During World War II the area was bombed. A search for unexploded bombs has been executed (no objects found).
6) Former (temporary) support foundations have been located and have been assigned for removal if they conflicted with the future excavation for the diaphragm walls.
7) Verticality of the pillars has been measured. This was considered an important proof of homogeneity of the soil characteristics. The pillars were still completely vertical after almost 140 years.
8) Ambient vibrations have been monitored to assess the influence of these vibrations on trench stability. The level of horizontal accelerations was low, causing no significant negative results on trench stability.

4.3. Optimizing the Design

Considering the low strength of the foundation slab and the high strength and stiffness of the soil beneath it, jet grouting below the slab seemed to offer no significant benefits while introducing risks for the foundation slab. The verticality of the pillars shows a ‘proven’ homogeneity of the soil beneath the slab and of the slab itself. The foundation system will perform just as well in the future, provided safe execution of the rigid box, consisting of diaphragm walls, is ensured and the stiffness of the soil-structure system is high enough.

The available space below the bridge is limited to 7 m. As a result, only small equipment can be used, making the verticality of excavation to the required depth of 22 m below surface level a matter of high operator skill.

The available space around the pillars was only 16 m. Taking into account that the foundation slab is 10 m wide, per side only 3 m were available in which an optimum had to be found between rigidity of the wall and safe execution. The smallest trench length with standard excavation equipment is slightly less than 3 m. With such trench dimensions, the trench stability became critical when the distance between foundation slab and trench was less than 1.5 m. This resulted in a maximum wall thickness of 1.5 m.

With a 1.5 m thickness it seemed impossible to remain within the uncracked stiffness trajectory of the bending stiffness of the wall. It was therefore assumed that additional measures like anchoring between the opposing diaphragm walls or pre-stressing of the diaphragm walls would be necessary. These measures however seemed difficult to implement without introducing new risks.

Plaxis 3D calculations showed that the 3D effect was reducing the bending moments in the diaphragm walls by more than 30% compared to a plane strain 2D calculation. This made an uncracked stiffness of the concrete within reach.

Because the braking force exerted on the bridge accounts for almost half of the bending moments in the walls, it was worthwhile to investigate the influence of dynamic soil stiffness on the bending moments during short term loading. Depending on the load duration and the initial stiffness of the soil, a dynamic stiffness which is 2 to 10 times higher can be realistic. It was assessed that for the very short loading time during braking, combined with the initial soil stiffness, the dynamic stiffness would be 3 times higher than the static stiffness. With this soil stiffness during brake loading, the bending moments in the wall remained within the stiff trajectory of the bending characteristics of the D-wall.

As a result, no anchoring or jet grouting below the foundation slab was needed to reach the required stiffness.

4.4. Predicting Deformations

With the aforementioned stiffness characteristics of the soil and the diaphragm wall, the expected deformations during phased construction were simulated with Plaxis 3D and Plaxis 2D (with foundation load reduction to compensate for the 3D to 2D simplification).

Beside the effects of finding a new equilibrium between stress and stiffness in the soil and structure, the installation effect has been assessed with a finite element code (Plaxis 3D) and with previous monitoring data (Clough and O'Rourke 1990). The experience from previous projects seems to include effects that cannot be simulated with a finite element code, as the finite element calculations show a much smaller expected installation effect (about 25% of the Clough and O'Rourke graph values), figure 7.

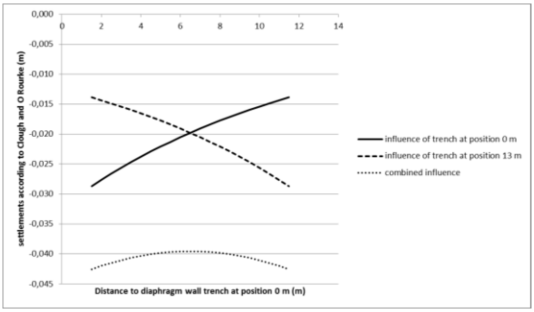

Figure 7. Excavation effect of two opposed diaphragm walls, using superposition of Clough and O'Rourke graphs.

4.5. Monitoring during construction

The groundwater table on site is governed by the water level in the river Waal. The water level was being monitored in the river upstream (in Germany), in the river adjacent to the project area and close to each pillar within the project. With this information the latency of the water system could be determined and it made accurate simulation of the groundwater table over several days possible. The maximum groundwater table for the diaphragm wall production was fixed in the contract. With the aid of the groundwater table forecast covering the following days, it was possible to decide if safe execution of a diaphragm wall panel was possible or that the groundwater table would have exceeded the allowed safety level before the planned finishing of the panel.

To check the soil tightness of the wall, all joints between the diaphragm wall panels were tested with the Cross-hole Sonic Logging (CSL) technique (Spruit et al. 2014). The CSL technique has been developed for application in diaphragm walls under Geo-Impuls, the Dutch joint industry programme aiming at reducing the occurrence of geotechnical failure in Dutch civil engineering. Based upon the CSL graphs, some joints needed inspection.

During excavation to 6 m below surface level, the wall could be inspected. The suspected joints showed a typical 'double joint' caused by spill concrete that was not successfully removed before concrete casting of the next panel (left panel in figure 8).

The core of the joint (where the joint profile had been extracted), between spill concrete and previous panel, was still filled with bentonite. Without CSL this anomaly would not have been noticed.

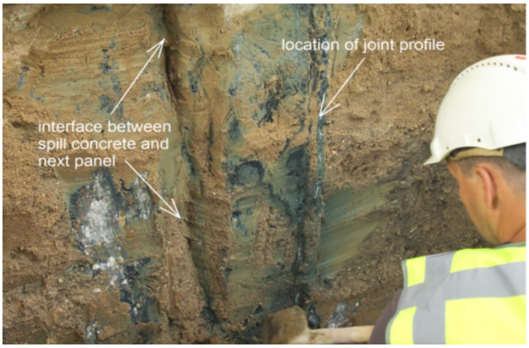

Figure 8. Typical double joint caused by spill concrete

Before, during and after installation of the diaphragm wall panels the soil eigenfrequency has been monitored with vibration measurements.

As shown in Pors 2014, monitoring eigenfrequencies can be used to verify changes in soil stiffness. The measurements have shown that during the installation of diaphragm walls the stiffness was temporarily reduced by 20%. After concrete casting of a panel, the stiffness was at the previous level almost immediately. After completion of the rigid in-situ box, the stiffness was slightly higher than before, probably caused by the volume of concrete increasing the average material stiffness.

Before and after the installation of diaphragm walls, around each pillar 10 CPT's were executed. The results showed a slight decrease in average cone resistance. Layers with an initial cone resistance higher than 35 MPa seem to be hardly affected, whereas sand layers with an initial cone resistance below 35 MPa generally show a 20% reduction. If the effect is averaged among all soil layers, the effect has been less than 20%.

Together with the observed small changes in the soil-structure system stiffness based upon eigenfrequencies, it was decided that no additional measures were needed to repair a loss in stiffness.

Settlements of the pillars were monitored with total stations. The total settlements ranged between 30 to 35 mm. This corresponds well to the estimated settlement in figure 7. Differential settlements perpendicular to the bridge were less than 10%. The verticality of the pillars was not significantly affected by the installation of the diaphragm wall panels.

5. Conclusions

When adapting existing structures, the quality of the materials and their geometry is uncertain and should not only be derived from archived design drawings. The properties and shape should always be verified in the field. Failing to do so strongly increases the chances of unexpected behavior of the construction and/or the surroundings.

The uncertainties in soil properties cause risks in many (not only complicated) projects. The quality of the site investigation should not be saved upon. To reduce the risk of unexpected soil behavior, it is necessary to reassess the soil parameters in each design stage. Due to (slight) changes in design philosophy, it can be necessary to update the site investigation with tests specifically tailored to the design calculations of that design stage.

The risk of trench instability during excavation of diaphragm walls can be reduced to acceptable levels if proper stability analyses have been carried out using both analytical and numerical (finite element) methods. Depending on site specific conditions, a higher safety level than stated in DIN 4126 can be chosen, for example if dynamic loading or vibrations are active around the trench.

Unanticipated deformations in the surroundings of diaphragm walls can be avoided if deformation estimations with empirical and finite element models are made. In this project the finite element settlement results were an underestimation of the actual settlements. The empirical settlement results were a slight overestimation of the actual settlements.

Apart from proper estimation of the expected deformations, it is necessary to execute the construction of the diaphragm walls to the highest standards and to verify their quality with the CSL technique. This technique offers a low-cost and effective way of assessing the actual quality of the joints between diaphragm walls after their production. With the information delivered by the CSL measurements, mitigating repair works can be implemented in a very early stage of the building process, strongly reducing the chances of project delay and/or calamities caused by water leakage and soil transport through anomalies in the joints.

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