Monitoring Analysis of Foundation Deformations of the Botlek Lifting Bridge

B. RIJNEVELD and J.A. JACOBSE

Geotechnical Engineering, Fugro GeoServices B.V. (seconded to Ballast Nedam Engineering B.V.), Netherlands

GEO2 Engineering (seconded to Ballast Nedam Engineering B.V.), Netherlands

Abstract. A new lifting bridge is being constructed crossing the river Oude Maas in the Rotterdam harbour area in the Netherlands. Since foundation deformations of the main piers have a large influence on the performance of the superstructure, extensive deformation analyses have been performed during the design phase. In order to minimise the risks and optimise the costs, the observational method was applied. Based on an extensive monitoring system the deformations of the bridge were monitored. Since the deformations remained well below the intervention values no control measures were necessary. Valuable lessons with respect to dealing with uncertainties in the analysis of measuring and estimation of residual settlements were obtained.

Keywords. Observational method, shallow foundation, monitoring, settlements, bridge foundation

1. Introduction

On behalf of Rijkswaterstaat, A-Lanes A15 realises the widening of the A15 in de Rotterdam harbour area. The consortium A-Lanes A15 is a co-operation between Ballast Nedam, John Laing, Strabag and Strukton. One of the main challenges in this project is the construction of a new lifting bridge crossing the river Oude Maas. Consisting of two lifting spans of about 100 m length and 60 m width, reaching more than 60 m above water level, this new bridge is one of the largest lifting bridges in Europe. In figure 1 a photo of the bridge during construction is presented.

The three main bridge piers are founded on rigid concrete blocks with footing dimensions of 15 m x 60 m, at 8 m below river bed on top of the first dense (Pleistocene) sand layer. In order to be able to pour the underwater concrete of the foundation blocks temporary building pits were used. In the foundation design, the foundations are effectively treated as shallow foundations. The serviceability limit state (SLS) foundation pressures for the different main piers are in the range between 500 to 700 kPa.

During the design process it was recognised that deformations of the foundation have a large influence on the performance of the superstructure, especially for the mechanical and moving parts of the bridge. Due to the large ratio between the height of the pylons and the width of the foundation, a small rotation of the foundation base results in a large deflection of the pylons heads. Due to strict tolerances of the mechanical and moving parts of the bridge this is a critical aspect in the design and construction of the bridge. Besides this effect, excessive (differential) settlements of the piers or differential settlements between the piers can result in jumps in the alignment of the road and/or a too low headway for the ships.

Figure 1. Photograph of the new Botlek lifting bridge with three main piers P30, P40 and P50
In order to determine safe tolerances which are taken into account by the other design disciplines, a thorough deformation analysis was performed in the geotechnical design. In order to minimise the risks and optimise the costs, the observational method was applied. This means that during construction the deformations were monitored, so control measures could be implemented if necessary. Possible control measures are e.g. an inclined construction of the main piers or an increased construction level of the bearing plates of the steel decks.

In this paper the results of the monitoring of the bridge, a back calculation of the measured deformations and the uncertainties during the application of the observational method are described.

2. Site Characterisation

In the design phase an extensive soil investigation has been performed. For a detailed description of the soil investigation reference is made to (Rijneveld, 2012) and (Jacobse, 2013). From the soil investigation the soil stratigraphy has been determined, see table 1.

<table>
<thead>
<tr>
<th>Top of layer [m NAP]</th>
<th>Soil description</th>
<th>Soil layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>-7 à -14</td>
<td>SAND, clayey</td>
<td>cover layer</td>
</tr>
<tr>
<td>-14 à -20</td>
<td>SAND, (medium) dense</td>
<td>1st sand layer</td>
</tr>
<tr>
<td>-33 à -39</td>
<td>CLAY, stiff</td>
<td>deep clay layer</td>
</tr>
<tr>
<td>-34 à -42</td>
<td>SAND, (medium) dense</td>
<td>2nd sand layer</td>
</tr>
<tr>
<td>-60</td>
<td>Max. investigation depth</td>
<td></td>
</tr>
</tbody>
</table>

In figure 2 the (variation in) thickness of the clay layer is presented. This deep clay layer was a major point of attention with respect to the deformation analysis.

In order to apply the observational method, insight in the deformation behaviour of the piers during construction is necessary. Therefore extensive monitoring has been performed with different systems. These systems are described in more detail.

3. Monitoring System

To monitor the horizontal and vertical deformations of the new piers prisms are installed on different construction objects. The prisms are monitored by two (coupled) total stations located on shore at a distance of about 100 m during the total construction time. After pouring the underwater concrete also manual measurements of the monitoring points were performed.

Since the piers are build up from an excavation inside a building pit up to a pylon of 80 m height it was not possible to monitor the same points during the total construction time. Initially about 20 points are installed on the sheet piles of the building pits, and additionally settlement plates are placed at foundation level inside the building pit. Before these monitoring points on the temporary building pits were removed, new prisms were placed on the permanent concrete parts at a higher level. The total deformation of the piers is determined by summing up the contributions of the different subsequent monitoring points.

Due to large distance which had to be covered by the continuous monitoring system on the shore, a large scatter in the measurements occurred. This scatter is caused by meteorological and tidal influences. However, the trend in measured values can be considered quite accurate, especially after regular manual measurements could be performed to verify the measured values.
3.2. Rotation Measurements

Initially the rotation of the piers was derived from the vertical settlement of the different measurement points. Since the inaccuracy of this method turned out to be quite high, additional rotation measuring tools were installed. These precision inclination sensors for simultaneous measurement of inclination, direction of inclination and temperature based on an optoelectronic concept, have an accuracy of approx. 0.005 mrad (1/12000).

3.3. Extensometers

At the north-east, north-west, south-east and south-west side of the building pits extensometers (so 4 in total per pier) are installed. These are located in the subsoil below the foundation level and measure the relative deformations at different depths below the foundation level. On four depths anchors are placed. Because only the relative deformations is measured, a fixed point should be assumed. In this case the lowest measuring point (at NAP -45 m) is used as a fixed point. With the extensometers it is possible to gain more insight in the contribution of the different soil layers to the total foundation deformations.

4. Deformation analysis

In this chapter the calculated, measured and prognosticated final deformations during construction are described.

4.1. Measured Deformations

As stated before different monitoring systems are used during construction. The results of these different systems are presented in figure 3 (only the first and last measurement). From this figure it can be seen that three phases can be distinguished.

Up to about 200 MN the measurements are performed on the temporary construction works and building pit. Between 200 and 300 MN several changes in monitoring points occurred. During this phase the first construction works above water level took place. Due to construction activities the measurement points had to be moved several times and some gaps in the time series occur. For the remaining construction phases permanent measurement points could be placed and a continuous measurement could be performed.

During the design a probabilistic deformation analysis was performed. For a detailed description reference is made to (Jacobse, 2013). The lower-, upper bounds and expected value of the calculated settlements are presented in figure 4. In this figure also the measured deformations are presented.

4.2. Uncertainties in Deformation Analysis

Since the observational method was applied, during construction the monitoring data was periodically analysed and compared to the calculated values in the design phase. For this analysis the final deformations and residual deformations are of importance. For the estimation of the final and residual deformations the main uncertainties are the uncertainty in soil behaviour and the uncertainty/inaccuracy of the measurement system. It is reasonable to model these uncertainties as random variables with a
normal distribution. In this case the total uncertainty, expressed in a standard deviation, can be determined according to:

\[ \sigma_T = \sqrt{\sigma_g^2 + \sigma_m^2} \]  

(1)

In which \(\sigma_g\) represents the standard deviation of the soil deformation and \(\sigma_m\) represents the standard deviation of the measurement system. The uncertainty bounds are defined by 1% exceedance bounds, or the +/- 2.5 sigma value.

For the uncertainty in soil behaviour a coefficient of variation of 0.2 to the residual settlements was applied. This value is based on a rule of thumb. The estimation of the final and residual settlements was calculated by applying the same ratio between the calculated and measured settlements at a certain time to the calculated final settlements.

For the measurement uncertainty the total inaccuracy of the monitoring systems is estimated as 20 mm. This value is build-up of several components:

- Inaccuracy in the first construction phases due to indirect measurements at temporary construction works and not directly to the foundation/concrete structure. This inaccuracy is estimated as 15 mm.
- Inaccuracy during the next construction phase where the measurement are disturbed by the construction activities and gaps in the time series are present. This inaccuracy is estimated as 10 mm.
- The inaccuracy of the deformation measurements itself is estimated as 5 mm. Since the monitoring points are changed about 3 times.

The total measurement inaccuracy is estimated as \(\sqrt{15^2 + 10^2 + 5^2 + 5^2 + 5^2} = 20\) mm.

The development of uncertainties for the estimation of the final settlements are given in figure 5. From this figure it can be seen that the uncertainty in soil behaviour reduces in time, since the residual settlements reduce in time. Since hardly any time dependent settlements occur, the uncertainty at the final load step reduces to zero. However, the measurement inaccuracy increases with time.

Another interesting aspect which can be seen in figure 5 is that the reduction in uncertainty of the soil behaviour is not linear. This mainly caused by the soil stiffness after 200 MN is much stiffer compared to the calculations.

4.3. Settlement Back Analysis

Based on the measurements a back analysis of the calculated values in the geotechnical design has been performed.

During the design a probabilistic deformation analysis was performed. The analysis consists of 3D FEM calculations with the software program Plaxis and the application of a fit-for-project probabilistic deformation model. For a detailed description of the probabilistic model reference is made to (Jacobse, 2013).

Based on correlations with the cone resistance according to (Lunne, 1997) and results of oedometer and triaxial tests the stiffnesses of the different soil layers were determined. For the 3D FEM calculations the Plaxis Hardening Soil model has been used. The reference stiffnesses for respectively the 1st sand, deep clay and 2nd sand layer were determined as \(E_{\text{ref}} = 40, 3.25\) and 30 MPa, with a power for the stress dependent stiffness of respectively \(m = 0.5, 0.75\) and 0.5.

The calculated and measured settlements are presented in table 2. From table 2 it can be seen that especially the measured settlements of P40 are much smaller compared to the calculated settlements.
Table 2. Calculated and measured total settlements [mm]

<table>
<thead>
<tr>
<th></th>
<th>P30</th>
<th>P40</th>
<th>P50</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Calculated</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lower bound</td>
<td>70</td>
<td>140</td>
<td>70</td>
</tr>
<tr>
<td>Expected value</td>
<td>120</td>
<td>230</td>
<td>110</td>
</tr>
<tr>
<td>Upper bound</td>
<td>190</td>
<td>370</td>
<td>180</td>
</tr>
<tr>
<td><strong>Measured</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lower bound</td>
<td>60</td>
<td>50</td>
<td>30</td>
</tr>
<tr>
<td>Expected value</td>
<td>80</td>
<td>70</td>
<td>50</td>
</tr>
<tr>
<td>Upper bound</td>
<td>100</td>
<td>90</td>
<td>70</td>
</tr>
</tbody>
</table>

In figure 6 the measured and calculated settlements (expected values) are presented graphically.

The change in soil stiffness at a load of about 200 to 300 MN is the result of overconsolidation effects. Note that the load-settlement behaviour is approximately linear. For 1D settlement situations a logarithmic load settlement curve would be expected, which correspond to oedometer test conditions. However, since the width of the foundation is relatively small, also deviatoric loading, which corresponds to triaxial test conditions with a hyperbolic load-settlement behaviour can be expected. The combinations of these two loading conditions can be an explanation of the calculated linear load settlement behaviour.

To get more insight in the reasons why the calculated settlements differ from the measured settlements the results of the extensometers are considered. The calculated and measured (expected values) of the settlements per layer are presented in table 3. The results of the extensometers tend to give lower total settlements compared to the results of the measured deformations at the prisms. It is assumed that the contribution in percentages of the total settlement is reasonably captured by the extensometers. Therefore the measured settlements in every layer are multiplied by the ratio between the measured total settlements by the prisms and the extensometers.

Table 3. Calculated and measured settlements per layer

<table>
<thead>
<tr>
<th>Settlement [mm]</th>
<th>P30</th>
<th>P40</th>
<th>P50</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Calculated</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1st sand layer</td>
<td>78 (65%)</td>
<td>120 (52%)</td>
<td>65 (59%)</td>
</tr>
<tr>
<td>deep clay layer</td>
<td>15 (12%)</td>
<td>65 (28%)</td>
<td>25 (23%)</td>
</tr>
<tr>
<td>2nd sand layer</td>
<td>27 (23%)</td>
<td>45 (20%)</td>
<td>20 (18%)</td>
</tr>
<tr>
<td><strong>Measured</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1st sand layer</td>
<td>72 (90%)</td>
<td>60 (75%)</td>
<td>62 (76%)</td>
</tr>
<tr>
<td>deep clay layer</td>
<td>7 (10%)</td>
<td>16 (20%)</td>
<td>16 (20%)</td>
</tr>
<tr>
<td>2nd sand layer</td>
<td>1 (&lt;1%)</td>
<td>4 (5%)</td>
<td>2 (2%)</td>
</tr>
</tbody>
</table>

From table 3 it can be seen that especially the settlement in the 2nd sand layer is negligible and therefore much lower than calculated. Possible causes for this can be a larger overconsolidation ratio caused by e.g. aging, more spreading of the load in the 1st sand and deep clay layer and/or a larger ratio between the virgin and reloading stiffness. Since the difference is so large, most likely a combination of these aspects play a role.

The calculated settlements of the 1st sand and deep clay layer at P30 and P50 are in reasonable agreement with the measured values. However, the measured settlements of the 1st sand and deep clay layer at P40 are much smaller than calculated. This might also be the result of a larger overconsolidation ratio. In the design a relative large part of the load was applied to the ‘virgin’ stiffness for P40 compared to P30 and P50.

From figure 5 it can be seen that the stiffness after a load of 250 MN is almost equal for the different piers (the load settlement curves are almost parallel). Especially the stiffness below the pre consolidation pressure differs. This might be due to soil behaviour, but it might also be the result of measurement inaccuracy, since during this phase only indirect measurements on temporary construction works could be performed. Most likely it is a combination of both.

4.4. Residual Deformations

From table 3 it can be seen that the settlements of the deep clay layer are limited. Therefore the time dependent settlements are expected to be relatively small. This makes the determination of the residual settlements much easier, since the uncertainty in consolidation and creep behaviour

Figure 6. Measured and calculated total settlements
is not very relevant. For the residual settlements it is also important to notice that the influence of the measurement inaccuracy is much smaller compared to the determination of the total final settlements. Since in this case hardly any time dependent settlements are expected, the residual settlements are negligible if no extra load is applied. The uncertainty in the total final settlement is larger. However, these are generally of less practical interest.

In the analysis above the effect of settlements due to cyclic loading are not taken into account. For this aspect an additional settlement during exploitation of about 0 to 20 mm is expected.

4.5. Observational Method

As stated before especially the rotations are of importance for the performance of the bridge. Since during the first stages the measurement accuracy was low, no reliable prediction of the (residual) rotations could be made. Therefore additional precision inclination sensors were placed. Since these sensors were very sensible, a lot of scatter during the construction phases occurred. However, since the measurement accuracy of the prisms increased during construction a reliable prediction could be made for the residual rotation at the crucial point during construction. This was at the moment that the pylons were constructed and the decision had to be made if corrective actions should be taken. The expected residual rotations were however, just like the expected residual settlements, below or close to the expected values calculated during the design. Therefore no corrective actions were necessary.

5. Lessons Learned

Accurate monitoring of a structure in a river is complicated, especially on the temporary construction works. Parallel monitoring with a permanent monitoring system (in this case extensometers) which results in an uninterrupted time series is advised. Measurement inaccuracy can be an important factor for the determination of the total final settlements. For the determination of the residual settlements this effect however is much smaller.

Due to the different monitoring systems more insight in the (in)accuracy of the measurements was obtained. By combination of all the data a more accurate analysis could be performed and necessary action to improve the performance of the monitoring system could be made (e.g. additional manual measurements and installation of precision inclination measurements). If only one monitoring system should have been present, the inaccuracy might have seemed smaller, but this is misleading. (In such cases 'one clock is not enough to know the time').

In general the total measurement accuracy is much larger than the sensor accuracy. Reasons for this are e.g. disturbance of the measurement points, indirect measurements (no direct measurement of the soil, but for instance a plate on the soil or anchors in the soil are performed), meteorological effects and deformation of 'fixed' reference points due to tidal influences and back ground settlements.

The observational method is a good way to optimize the design of even exceptionally large structures. However, it is very important to deal with all the different sources of uncertainties and apply a reliable and robust monitoring system. Moreover the design- and construction team should be aware of the possibility that control measures might be necessary during construction.

References

