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Abutments placed on reinforced soil structures on partially cemented stone columns in soft peat and clay layers

Culées fondées sur sol conforté avec des colonnes ballastées partiellement cimentées dans des couches de tourbe et d'argile molle

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ABSTRACT: For the isostatic and hyperstatic bridges of the Oosterweelverbinding Linker Oever (OWV-LB) project in Antwerp Rinkoniên (contractor) proposed to Lantis (client) abutments on reinforced soil with partially cemented stone columns through soft soil layers as alternative to traditional abutments on piles. This alternative is very cost effective and reduces construction time. Since the settlement differences between the abutments and embankment are more gradual, there will be less problems at the transition zone. The system is also capable to deal with the horizontal forces coming from the abutment and embankment. The design of the reinforced soil structure and the stone columns included limit equilibrium theory models and 2-dimensional (2D) and 3-dimensional (3D) finite element method calculations. Comparison is made between the different model approaches with emphasis on the 2D Finite Element model (FEM) modelling of the closely spaced stone columns. Results of pile load tests on stone columns were used in the design. The reinforced earth structure founded on stone columns proved to be a feasible, robust and cost-effective integral design solution for abutments on soft soil conditions as alternative for the traditional abutment on piles.

RÉSUMÉ : Pour les ponts isostatiques et hyperstatiques du projet Oosterweelverbinding Linker Oever (OWV-LB) à Anvers Rinkoniên (entrepreneur) a proposé à Lantis (client) des culées fondées sur sol conforté avec des colonnes ballastées et partiellement cimentées à travers des couches de sol molles comme alternative à une culée traditionnelle fondée sur pieux. Cette alternative est rentable et réduit le temps de construction. Les différences de tassement entre les culées et le remblai étant plus graduelles, poserons moins de problèmes dans la zone de transition. Le système est également capable de gérer les forces horizontales provenant de la culée et du remblai. La conception de la structure du sol conforté et des colonnes ballastées comprenait des modèles théoriques d'équilibre limite et des calculs de méthode par éléments finis en 2 et 3 dimensions. Une comparaison est faite entre les différentes approches du modèle en mettant l'accent sur la modélisation FEM 2D des colonnes ballastées étroitement espacées qui ont montré un comportement du sol plus efficace. Les résultats des tests de chargement des colonnes ballastées ont été utilisés dans la conception. La structure sol conforté avec les colonnes ballastées s'est avérée être une solution de conception intégrale, faisable, robuste et rentable pour les culées avec des conditions de sol molles comme alternative à la culée traditionnelle sur pieux.

KEYWORDS: Stone columns, reinforced soil, soft soil, abutment.

1 INTRODUCTION

Traditionally, when embankments are used to create the elevation for a bridge, abutments can be full height, e.g. as an L-shaped retaining wall or embedded with a slope, where latter one increases the bridge span. When soft layers are present in the subsoil, these abutments are placed usually on concrete or steel piles.

Recently in the Netherlands and Belgium abutments have been placed on reinforced soil (e.g. Linthof et al., 2013 and Brok & Alexiew, 2012). In case of favorable soil conditions (e.g. sand) usually no additional measures are necessary. However, if soft soil layers (e.g. peat and/or clay) are present in the subsoil, stability and settlements may become an issue. In some cases, these layers can be replaced. However, in many cases this is expensive or prohibited e.g. from a geohydrological point of view.

To overcome stability and settlement issues in case of presence of soft soil layers, stone columns have been used underneath reinforced soil at the Oosterweel Verbinding Left Bank (OWV-LB) project in Antwerp. This paper presents the design philosophy used at the OWV-LB project for the reinforced soil bodies and stone columns and presents pile load tests on stone columns.

Embankments on soft soil layers will settle in time, due to consolidation and creep. The piled abutment will not settle in time. Hence, in time a settlement difference will occur between the embankment and the adjacent abutment, for which transition plates are necessary. In case of an abutment on reinforced soil placed on stone columns, the settlement difference between the embankment and abutment will be more gradual because reinforced soil bodies are more flexible than a piled abutment, provided that the stone columns will spread out in the direction away from the abutment. Additionally, the method of an abutment on reinforced soil is faster to construct and very cost effective compared to a piled abutment.

2 DESIGN CONSIDERATIONS

The reinforced soil bodies at the OWV-LB project consist of sand bodies with horizontal geogrids with a vertical spacing of 0,5 m to 1 m distance (Figure 1). The geogrids are connected to a gabion wall, which form the front of the reinforced soil body. The foundation of the reinforced soil body is formed by a gravel layer reinforced with a geogrid on top of stone columns.

The dry-bottom-feed method was used for the execution of stone columns at the OWV-LB project. In the dry-bottom-feed method gravel is brought into the ground using a vibrator which is mounted at the lower end of a steel tube. The gravel is densified during the installation by moving the vibrator up and down. In the soft (peat and clay) layers, wet or dry cement is usually added to the gravel, which results in a stronger column. When using wet concrete, the stone columns become more rigid and may break successively. At the OWV-LB project dry cement was used. Dry cement stone columns are more flexible. When loaded under shear stress, the columns will deform until any overload is transferred to neighboring columns, thus mobilizing the bearing capacity of each column in the group.

Stone columns are used to reduce the settlements and increase the stability of the reinforced soil body. To better redistribute the loads from the reinforced soil body onto the stone columns through arching, a reinforced gravel layer is placed on top of the stone columns.

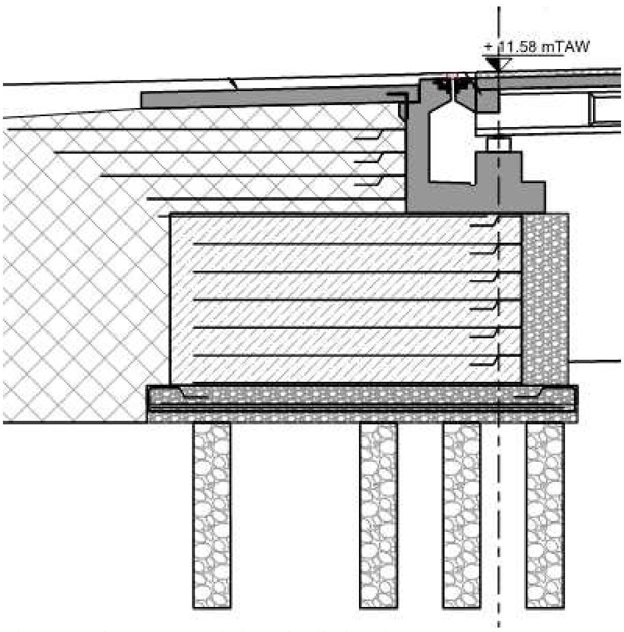

Figure 1. Abutment on reinforced soil placed on stone columns.

2.1 *Reinforced soil*

CUR 198 (2017) provides guidelines for the design of reinforced soil structures. This guideline was used in the design of the reinforced soil structures of the OWV-LB project. Limit equilibrium methods were used to assess the dimensions of the reinforced soil body. Checks included sliding, tilting, bearing capacity and a deep Bishop failure mechanism. Once the dimensions were determined, Plaxis 2D and Plaxis 3D calculations were performed to check the failure mechanisms and displacements. Final design included checks on internal stability of the reinforced soil structure according to guidelines provided in CUR 198 (2017).

2.2 *Stone columns*

Limit equilibrium methods were used to assess the dimensions of the stone columns. For vertical loads basically three failure mechanisms may occur (Figure 2**Error! Reference source not found.**):

- Bulging of the stone column
- Shearing of the stone column and the surrounding soil
- Bearing failure (sinking)

Bulging of stone columns was assessed using the following equations (De Cock and D'Hoore, 1994):

$$\boldsymbol{\sigma_{vc,ult}} = \boldsymbol{\sigma_{hc,ult}} \cdot \mathbf{tan^2}\left(\frac{\pi}{4}+\frac{\varphi}{2}\right) + \mathbf{2} \cdot \boldsymbol{c'} \cdot \mathbf{tan}\left(\frac{\pi}{4}+\frac{\varphi}{2}\right) \quad (1a)$$

$$\boldsymbol{\sigma_{hc,ult}} = \boldsymbol{\sigma'_{h0}} + \boldsymbol{k} \cdot \boldsymbol{c_u} \quad (1b)$$

$$\boldsymbol{k} = 1 + \frac{E_s}{2\cdot(1+\nu_s)\cdot c_u} \approx \mathbf{4} \quad (1c)$$

Where $\sigma_{vc,ult}$ is the ultimate vertical stress on the stone column, $\sigma_{hc,ult}$ is the ultimate horizontal stress on the stone column, φ is the angle of internal friction of the stone column, c' is the effective cohesion of the stone column, σ'_{h0} is the vertical effective stress in the soil, c_u is the undrained shear strength of the soil, E_s is the Youngs modules of the soil and ν_s is the Poisson ratio of the soil.

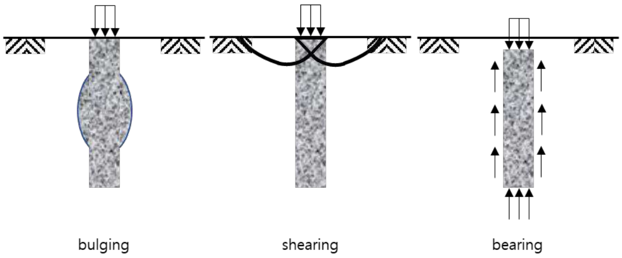

Figure 2. Failure mechanisms of stone columns.

These equations describe the failure state when the ultimate horizontal stress in the stone column cannot be balanced anymore by the resisting horizontal stress of the surrounding soil, in analogy with a triaxial test.

Shearing may occur if the characteristics of the soil are close to the characteristics of the stone column (Berthelot et al, 2004). Since usually stone columns are used in soft soil layers, this failure mechanism is rare.

Bearing failure was assessed using the Belgian standard (WTCB, 2019) for bearing capacity of piles, where it is noted that for very short piles with L/D factors less then say 6, the stone columns may not behave as piles but will behave as intermediate or shallow foundations.

The reinforced soil acts as a retaining wall. Hence the stone columns under the reinforced soil will be loaded both vertically and horizontally. This may result in a failure plane near the top of the stone columns, or going through the stone columns (sliding, Prandtl wedge and/or Bishop failure mechanism) and/or rotational failure of the stone columns (Figure 3). These failure mechanisms have been studied with Plaxis.

Especially in case the abutment is part of a statically indeterminate structure and to a lesser extend for a statically determinate structure, allowable vertical displacements are small (few centimeter). Additionally, the joint between the bridge deck and the abutment need to be capable of absorbing horizontal displacements of the abutment, which means also the allowable horizontal displacements of the reinforced soil body are small (few centimeter). Vertical and horizontal displacements have been assessed using 2D and 3D Plaxis models.

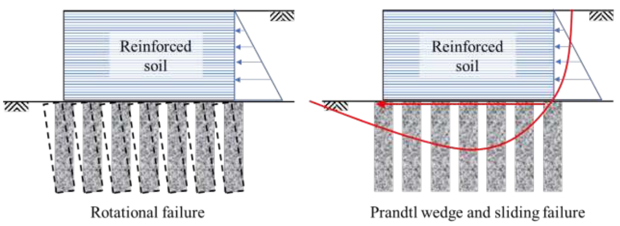

Figure 3. Failure mechanisms of stone columns.

Modelling the stone columns within the soil is complicated when using limit equilibrium models such as sliding, bearing capacity and Bishop failure mechanisms. For these limit equilibrium models (sliding, overturning, bearing capacity (NBN EN 1997-1) and Bishop failure) used in design of the reinforced soil body, the stone columns and surrounding soil were modelled as an equivalent homogenous soil body. The shear strength parameters of the equivalent homogenous soil body were assessed using design recommendations from Priebe (1995). The Priebe (1995) method was developed mainly for the deformation of vertically loaded stone columns in loose or soft soils, but it also considers shear strength parameters which can be used for stone columns in loose or soft soils with vertical and horizontal loads such as is the case for the reinforced soil bodies at the OWV-LB project. In the assessment of the shear strength the Priebe method considers the stress concentration on the stone columns due to differences in rigidity of the soil and the stone columns.

In Plaxis 2D the stone columns and surrounding soil can be modelled either as a homogenous soil body, analogue to the method used for the limit equilibrium models and can be modelled as longitudinal gravel trenches inside the surrounding soil (Figure 4).

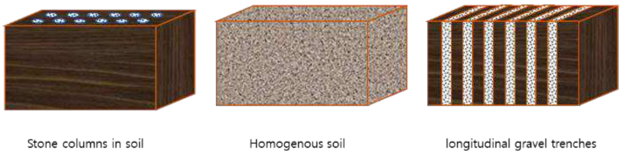

Figure 4. Modelling of stone columns in the soil.

According to Castro (2017) modelling the stone columns as longitudinal trenches is better than modelling them as a homogenous soil body for stability analysis. Hence, in the Plaxis models, for the more critical abutments of the statically undetermined bridges, longitudinal trenches were used whereas for the other abutments a homogenous soil body was used. In very critical cases, 3D Plaxis models were used, where the actual stone columns could be modelled as in reality.

3 OOSTERWEELVERBINDING ANTWERPEN

3.1 *Project description*

The construction of the Oosterweel Verbinding will close the R1 highway around Antwerp. A new highway tunnel under the river Scheldt will connect the Left and Right Bank on the north side of the city. The extra crossing of the Scheldt should ensure a better distribution of traffic and therefore less traffic jams. In addition, currently dangerous locations of the R1 are eliminated and rat-run traffic is reduced. The project also respects the environment and local residents. This improves the quality of life in the Antwerp region.

The Oosterweel Verbinding consists of five subprojects. The first subproject is the Left Bank or "Linkeroever" (OWV-LB). New highways and reconstructed interchanges will connect the Scheldt tunnel with the highway E34 to Knokke, the highway E17 to Ghent and the Kennedytunnel. (https://www.oosterweelverbinding.be/)

At 15 locations of the OWV-LB reinforced soil structures that act as retaining walls will be installed. On 10 reinforced soil structures foundations for abutments of bridges will be placed.

3.2 *K13/14 bridge*

3.2.1 Description K13-K14 bridge

K13-K14 is a two-span bridge over the E34 highway constructed from prestressed prefabricated beams. The construction is isostatic, and the spans are +/- 21 m. The elevated concrete foundations are placed on a reinforced soil body which is supported by stone columns.

The reinforced soil body consists of 11 m long Paralink 500 geogrids which are installed in different layers with a spacing of 50 cm. The geogrids have a relatively high short-term stiffness of $EA_{short} = 5284$ kN/m and long-term stiffness of $EA_{long} = 3963$ kN/m, which limits the deformations of the reinforced soil. Two Paralink 500 geogrids are also connected to the backside of the concrete abutment to reduce the rotation of the abutment. Limiting the deformations is very important to avoid failure of the bearing devices between the bridge and the abutment.

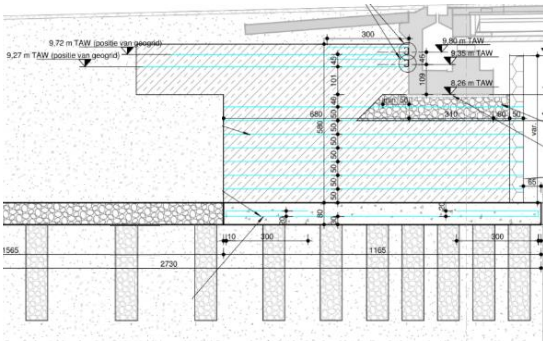

Figure 5. Reinforced soil body K13-K14

Because of a 2 m thick top soil layer consisting of quarterly clay with a cone resistance of 1.5 MPa soil improvement is needed. Therefore +/- 3 m long stone columns with a diameter of 0,8 m are installed below the reinforced soil. Over the height of the soft layers, the stone columns are stabilized with dry cement to increase the strength of the stone columns and avoid groundwater flow between the permeable layers above and underneath the clay layer.

3.2.2 *Limit equilibrium*

The calculation of the reinforced soil started with rules of thumb from the CUR 198 guideline. The minimum length of the geogrids $L = 0.6 \times (H_m + H_{add}) + 2 = 6.9$ m. The retaining height of the reinforced soil H_m is 6.2 m. An additional fictional extra height H_{add} of 2.0 m was considered due to the governing uniform load of 35.7 kN/m² on top of the reinforced soil. The minimal embedment depth $D_m = (H_m + H_{add})/ 10 = 0.8$ m.

The bearing capacity, sliding and tilting are checked with analytical calculations according to NBN EN 1997-1. The bearing capacity (Prandtl wedge) calculation determined the 11 m length of the geogrids. The effect of the stone columns on the soil parameters is included in this calculation. The original quarterly clay layer has an angle of internal friction of 20°. Using the method as described by Priebe (1995) the improved angle of internal friction of the homogenized soil is calculated as 28.6°. The calculated safety factors are 3.83 for sliding, 10.54 for tilting and 1.26 for the bearing capacity.

The last analytic check is the sliding equilibrium according to the Bishop method. The verification of the deep sliding surfaces provides a safety of 1.76.

The overall conclusion is that the bearing capacity is governing and determines the length of 11 m of the geogrids. The design focusses on the external stability of the reinforced soil body. The internal stability, including the tensile strength of the geogrids was assessed by the supplier of the geogrids.

3.2.3 *Plaxis 2D*

More advanced calculations are performed with Plaxis 2D. The Plaxis 2D assessment provide information on the expected deformations of the reinforced soil body. In addition, these calculations also provide a check for the analytical calculations.

The model is constructed starting from the different soil layers. The stone columns are modeled as a homogeneous soil layer with improved characteristics according to Priebe (1995), layers [4] and [5] in Figure 6. The viscoelastic material type is used for the geogrids. In this way the stiffness can be specified for the short term EA_{short} and the long term EA_{long}. The load from the bridge deck (311 kN/m vertical and 17.2 kN/m horizontal) is modeled as a line load on the abutment. Behind the abutment the self-weight of the transition plate, road and mobile traffic load are modelled as a uniform load of 35.65 kN/m².

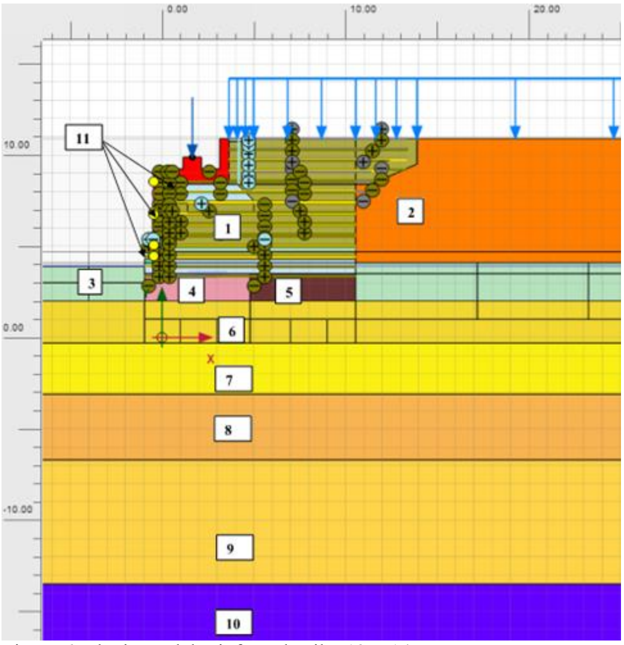

Figure 6. Plaxis model reinforced soil K13-K14

Following staging is used in Plaxis:

1. Construction of reinforced soil & abutment
2. Construction bridge deck
3. Construction road on deck
4. Mobile load
5. Removal of mobile load
6. Mobile load
7. Long term 50 year – mobile load
8. Long term 50 year – without mobile load
9. Long term 100 year – mobile load
10. Long term 100 year – without mobile load

The global stability is checked with a phi-c reduction for each stage. The safety is 1.25, which is sufficient. Furthermore, the internal creep strain of the geogrids is 0.14%, which is less than the maximum requirement of 0.5 %. Finally, the deformations of the reinforced soil body are checked. The residual settlement is 28 mm after 100 years. The maximum horizontal displacement is 25 mm after 100 years. The horizontal displacements are important for the design of joint and bearing devices, their design determined the (high) stiffness of the geogrids.

The stress on the stone columns is also derived from the Plaxis model, namely 188 kN/m². According to De Cock and D'Hoore (1994) the serviceability limit state (SLS) bearing capacity is 350 kN/column. Assuming the stone columns carry all the weight, the center-to-center distance of the stone columns is taken as 1.3 m in a square grid. The spacing of the stone columns can be increased to the back of the reinforced soil as the vertical stress decreases and more settlements are allowed. The 350 kN/column will be validated with a pile load test, see §3.2.5. The failure mechanisms of the stone columns as described in §2.2 are also checked. Bulging is not a problem because the stone columns are cement stabilized. The bearing capacity of the stone column is assessed using the Belgian standard (WTCB, 2019) and proved sufficient. However, since the L/D of the stone columns is 4, the stone columns were also assessed using Belgian standards for shallow foundations, which also proved sufficient.

3.2.4 *Limit equilibrium versus FEM*

The factors of safety (SF), defined as the reduction factor on the internal friction (tan (φ)/SF) and cohesion (c'/SF) are 1.25 (Plaxis) and 1.41 (NBN EN 1997-1 limit equilibrium). The factor of safety from the Bishop analysis is 1.71 (Resistance/Load). The safety factor of the FEM is much smaller than that of the limit equilibrium methods. From the failure wedges in Figure 7 the failure wedge from the NBN EN 1997-1 limit equilibrium method (red line) is much deeper and wider, which leads to a higher resistance. The Bishop failure circle (yellow dashed line in Figure 7 is much shallower and wider dan the FEM failure wedge. Therefore, it was concluded that the NBN EN 1997-1 limit equilibrium model and Bishop failure circle can be used as a first approximate for the indicative dimensions of the reinforced soil body but should not be used for final design purposes. The authors recommend the use of finite element models (FEM) for design of reinforced soil structures.

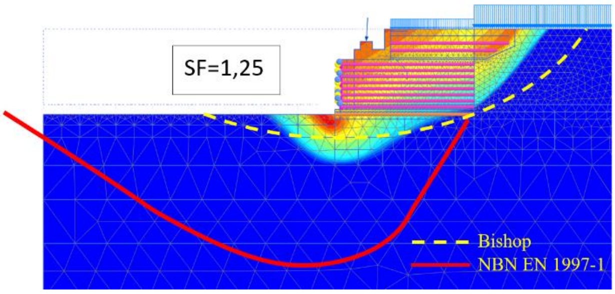

Figure 7. Plaxis 2D results for Priebe model

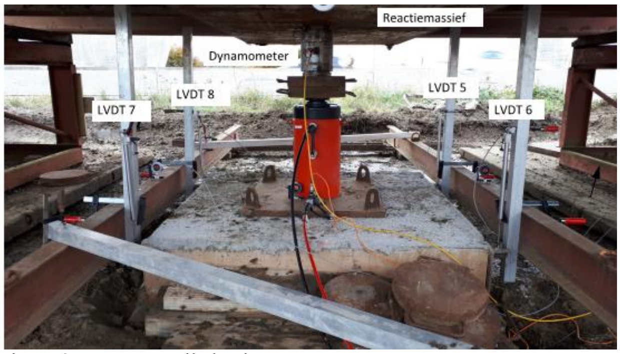

Figure 8. Test setup pile load test

3.2.5 *Pile load test stone columns*

Pile load tests on stone columns were performed by Wetenschappelijk en Technisch Centrum voor het Bouwbedrijf (WTCB) in Belgium at the project location. The applied test method is based on Berthelot et al. (2004). After execution of the stone column, the top meter will be excavated. A concrete block of 1.4 x 1.4 m is poured, and a hydraulic jack is placed on top to apply the load. The counterweight is created by a sand-filed container. The load is applied in steps of 0.25 x Fc,d up to a maximum load of 1.50 x $F_{c,d}$, where $F_{c,d}$ is the design value of the axial compressive load in SLS. The duration of each load step is at least 30 minutes. At $F_{c,d}$ the load must be maintained till the settlements are stabilized. After the maximum load had been reached, the stone column is relieved in 3 steps (10 minutes/step).

The measured load displacement curve is presented in Figure 9. The measured settlements are 4.03 mm at $F_{c,d}$ (350 kN) and 8.59 mm at 1.5 x $F_{c,d}$ (525 kN). The design of the stone columns was based on a bearing capacity of 350 kN/column and an associated deformation of 5 to 10 mm. Therefore, the stone columns meet more than expectations.

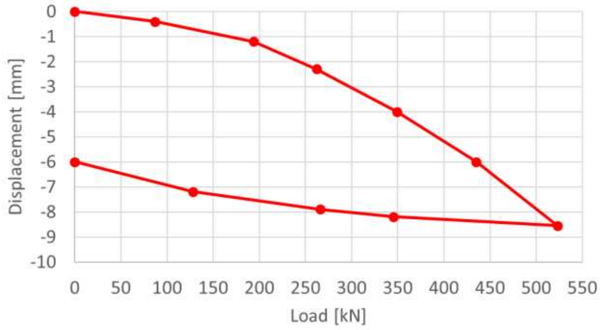

Figure 9. Load displacement curve pile load test

3.3 *K04 bridge*

3.3.1 *Description K04 bridge*

The K04 is a statically undetermined bridge. The northern abutment of the bridge is located on the G08 embankment, which also contains the abutments for 3 other bridges (Figure 10). The G08 has a phased construction from the south to the north (left to right on

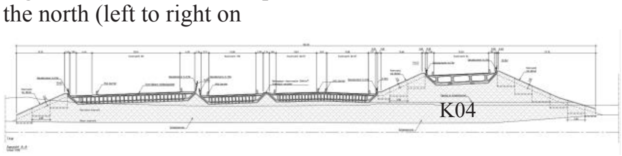

Figure 10), which complicates design.

Figure 10. Abutments on G08

In front of the G08 is a railway with strict deformation limits.

The reinforced soil body consists of Paralink 400 and 500 geogrids with a vertical spacing of 0,5 m. The geogrids have different lengths varying between 14 m and 17 m because of optimization. The stone columns underneath the geogrids are placed in a triangular grid and a center-to-center spacing of 1,4 m. The diameter of the stone columns is 0,8 m. The abutment is placed 3 m behind the front wall to increase stability. This also improves safety during construction of the abutment.

3.3.2 *Limit equilibrium*

Limit equilibrium assessments were performed, analogue to the K13/K14 bridge assessments. Based on these assessments, the general dimensions of the geogrids were determined.

3.3.3 *Plaxis 2D*

The strict deformation limits, combined with the significant height of the embankment required a complicated Plaxis 2D model, where the stone columns were modelled as longitudinal trenches and taking into account the installation phases and time (e.g. consolidation). Additionally, the bonding effect of the cement in the stone columns was considered, by using a cautious value for the cohesion of 100 kPa for the stone columns. Comparison was made with a model where the stone columns were modelled as a homogeneous soil body with improved characteristics according to Priebe (1995). The results are presented in Figure 11 and Figure 12. The factor of safety (SF), defined as the reduction factor on the internal friction (tan (φ)/SF) and cohesion (c'/SF) for the Priebe method and trenches method is 1,39 and 1,30 respectively. It is noted that the SF for a shallow foundation is not the same as a safety factor between the ultimate load and ultimate resistance since reduction of tan (φ) and c' has a non-linear effect on the reduction of the resistance.

The SF for the limit equilibrium assessment according to the NBN EN 1997-1 for shallow foundations is 1,38, where the stone columns and soil have been modelled as homogenized soil body according to Priebe (1995) recommendations. The failure wedge of the limit equilibrium method is also presented in Figure 11 and Figure 12 (red lines). The factor of safety from the Bishop analysis is 1,38 (resistance/load). The Bishop failure circle is presented in Figure 11 and Figure 12 with dashed yellow lines. Additionally, in Figure 11 and Figure 12 the failure wedge for the undrained bearing capacity case has been presented (dashed red lines). Although the SF is almost equal to the SF for the Plaxis 2D Priebe model (SF=1,39), the failure wedge from the NBN EN 1997-1 limit equilibrium method is much deeper and wider, which would lead to a higher resistance. In the Plaxis 2D model this is compensated by the mobilized shear resistance behind the reinforced soil, which is not considered in the limit equilibrium model. Therefore, it was concluded that the NBN EN 1997-1 limit equilibrium model can be used as a first approximate for the necessary dimensions of the reinforced soil body but should not be used for design purposes. Interestingly the Bishop failure circle is close to the failure wedges of the FEM models. The authors recommend the use of finite element models (FEM) for design of reinforced soil structures, especially in case of abutments and high structures.

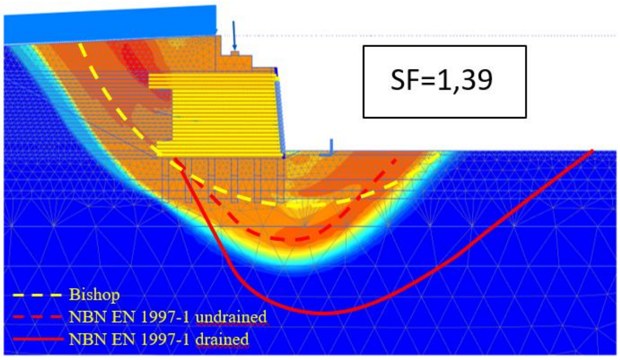

Figure 11. Plaxis 2D results for Priebe model

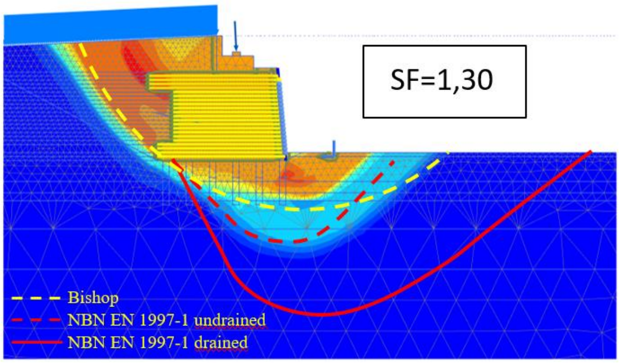

Figure 12. Plaxis 2D results for trenches method

To limit the displacements, Paralink 400 and 500 geogrids have been used. These geogrids have a relatively high stiffness and strength. After construction of the reinforced soil body, the calculated displacements (FEM) of the abutment are 14 cm vertical and 13 cm horizontal. After placement of the bridge, the calculated additional vertical displacement (FEM) of the abutment is 4,5 cm, of which 2,5 cm is due to consolidation of the Boom clay, which is approximately 65 m to 70 m thick.

Figure 13 presents the measured and calculated displacements at two locations at the front of the reinforced soil structure. The measured displacements are smaller than the calculated displacements although, it is expected that the measured displacements will continue to increase. The difference between the measured values and calculated values may be caused by 3D effects in reality versus 2D plane strain in the FEM calculations, the selection of characteristic values for the soil parameters and a cautious selection of strength and stiffness parameters for the stone columns. But since the measuring interval is still short, no final conclusions can be drawn yet.

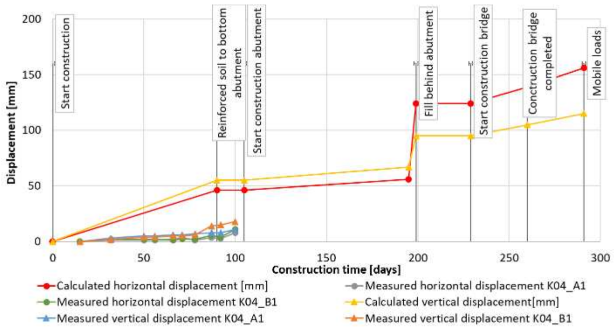

Figure 13. Measured and calculated displacements.

It is noted that during the final design, the requirements for the actual construction of the statically undetermined bridge further complicated the design.

3.3.4 *Plaxis 3D*

To check the 2D Plaxis calculations, also a 3D Plaxis model was made. The 3D Plaxis model consisted of a slice through the reinforced soil body (Figure 14), making use of (plane strain) symmetry at the sides of the slice. In Figure 14 also a close up of the stone columns can be seen, placed in a triangular grid.

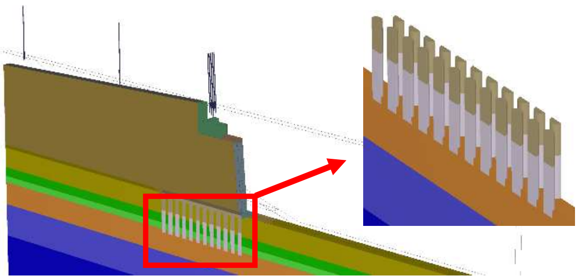

Figure 14. Plaxis 3D model with close up of the stone columns.

The 3D model has the same loads, material models, material parameters and phases as the 2D model. The stone columns have the same material parameters as the stone columns in the 2D trenches model. The only difference was the 3D modelling of the stone columns.

The safety factor for the 3D model is 1,38. This is similar to the 2D Priebe method.

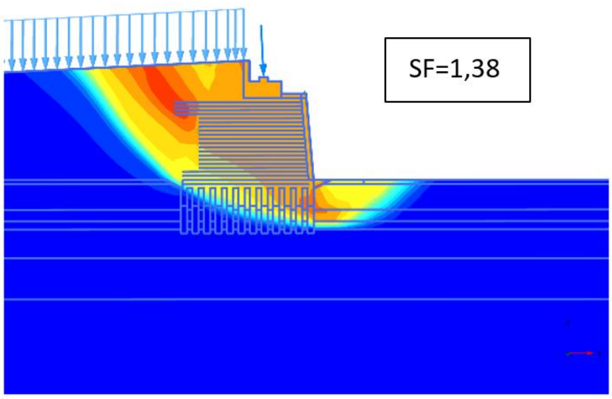

Figure 15. Plaxis 2D total displacements during the use phase.

Although the factor of safety is almost equal to the 2D Priebe method, the failure wedge is shallower than that of the Plaxis 2D Priebe method. The failure wedge of the Plaxis 3D model resembles that of the Plaxis 2D trenches method.

For the FEM modelling of the stone columns the authors therefore recommended to use the trenches method.

4 CONCLUSIONS

Reinforced soil structures on stone columns can be used as foundation for abutments of both static and hyperstatic bridges, even in case of shallow soft soil layers as a cost-effective alternative with reduced building time compared to traditional abutments on piles.

Stone columns were successfully used at the OWV-LB project to increase strength and stiffness of shallow soft (clay and peat) layers below reinforced soil structures.

The NBN EN 1997-1 limit equilibrium method for shallow foundations and deep Bishop failure mechanism can be used for a first approximation of the necessary dimensions of reinforced soil structures but should be used with care for final design of reinforced soil structures, as the actual failure mechanism might be significantly different. In limit equilibrium models, stone columns can be modelled using Priebe (1995) recommendations.

For final design 2D or 3D FEM calculations are recommended, especially in case of abutments and high structures.

The 3D model resulted in a higher SF than the trenches model. The SF of the 3D Plaxis model was close to the SF of the 2D Priebe model. The failure wedge of the Plaxis 3D model resembled that of the 2D Trenches model and is therefore the preferred 2 FEM model. The additional effort to model trenches compared to the Priebe method is usually relatively small.

Limitation of vertical and horizontal displacements is crucial in the design of the reinforced soil structures and dictate the strength and stiffness of the geogrids. Field measurements indicate that the measured vertical and horizontal displacements are smaller than the calculated displacements. The difference between the measured values and calculated values may be caused by 3D effects in reality versus 2D plane strain in the FEM calculations, the selection of characteristic values for the soil parameters and a cautious selection of strength and stiffness parameters for the stone columns. But since the measuring interval is still short, no final conclusions can be drawn yet.

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