Improved design for bridge abutments using reinforced soil

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ABSTRACT: Creating robust and sustainable constructions in geotechnical engineering has become an upcoming topic in terms of reduction of carbon footprint as well as cost reduction on PPP projects. Combining technologies for slim precast concrete panels with stiff geosynthetic reinforced walls allows for the use of local and, if required, even treated soils. Design of precast panels in practice requires attention to the transport phase as well as on the construction steps during execution and serviceability limit state. Full-scale laboratory tests have been performed at high stress levels with 450 kPa according to bridge abutments as well as at dynamic loads. Additionally, a pilot wall three years in service has been loaded with 250 kPa and stress as well as deformation conditions have been measured. Monitoring allows for satisfactory back-analysis of the construction steps. The measured stress conditions fit with the expected low stress approach for the combined structure. The findings combine the current results of international research and updated design approaches (EBGEO) to a new stage of earthworks technology for steep walls. Attention has to be paid to the connection stress and details of concrete vs. polymeric reinforcement, intensive testing has been performed therefore. The implementation of the test results and the calibration of finite element models give the required input for a safe and economic design of the innovative structure. Beside the test results and the design concept taking complex soil-structure-interaction into consideration, a project using the innovative construction and design concept of this soil structure as bridge abutment and retaining wall is presented.

Keywords: reinforced soil, bridge abutment, geogrid, active facing, passive facing

1 INTRODUCTION

Reinforced soil constructions have become standard for the use on infrastructural projects as cost-effective and variable structures (Heerten et. al, 2013), allowing the use of local soils and even soils with a significant amount of harmful substances when covered and immobilized appropriately (Egloffstein et al. 2014, Schmidt et al. 2014). The constructions can be integrated in the natural surrounding and architectural aspects can be met by the use of different types of facings. Beside sound barriers and steep reinforced embankments, reinforced soil constructions are even used as bridge abutments since decades based on international design codes as e.g. BS 8006 (British Standard 8006).

Nevertheless, it has been questionable for authorities whether the structures used for bridge abutments allow for the same safety and serviceability as conventional concrete structures. Having a close look to Japanese experience during very heavy earthquake situations on geogrid reinforced

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walls and geogrid reinforced bridge abutments used for high speed tracks, these concerns have to be neglected. So, nowadays, it can be stated that geogrid reinforced soil constructions allow to combine very high load bearing capacities with a very high ductility of the structures in terms of subsoil settlements resp. differential settlements. The diversity of facing types is more or less of visual interest, but ensures long-term protection of the front in terms of erosion and impacts (Herold & Vollmert, 2013). Therefore, robustness plays a significant role.

For the facing under a bridge where on the one hand no vegetation can be ensured, but on the other hand impacts by crash and fire might lead to significant safety reduction, concrete facings are often preferred.

Typically, at first the geogrid reinforced wall is installed using temporary formwork while, secondly, the concrete facing is placed in front of the wall (Fig. 1, a)). In this case the temporary formwork causes costs and requires additional installation time. On the other side, the reinforced wall itself fulfills the static requirements and the facing ensures long-term protection and visual requirements (passive facing). In case of damage, the concrete panel can be easily replaced and leads to improved robustness. To improve this installation technique, full-height panels are used. The full-height panel is placed first and temporarily supported before the soil is installed in layers. The panel and the reinforced soil structure are connected by geogrids which are connected by bodkins to geogrid sections, which are precast into the concrete. This technique leads to the result that the concrete has to take lateral earth pressure (active facing element) and is integral part of the overall construction (Fig. 1, b)). Relatively high efforts are required for the installation of the bodkin. For a more economic approach the authors follow the principle to use full-height panels as facing and temporary formwork, but to separate the function of the facing (robustness/protection) and the geogrid reinforced structure (static function).

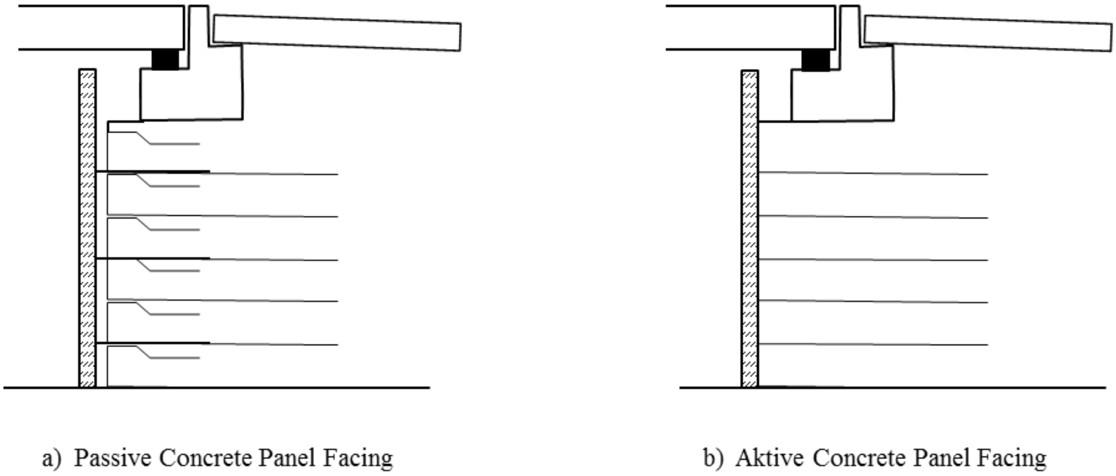

Figure 1: Layout of concrete panel facing systems

2 NEW DEVELOPMENTS USING FULL-HEIGHT CONCRETE PANELS

2.1 *Improved concept for design and installation technique*

The improved design for panel facing systems asks for two aspects:

- Easy but safe installation technique: Instead of using a bodkin or installing the full static length of a geogrid, just a reduced section (approx. 1.5 m) of a geogrid is installed in the panel during the precast process and overlapped with the geogrids required for the reinforced soil structure, computed by design.
- Static approach: The improved design assumes that due to the reinforced soil fill behind the facing the assumption of active earth pressure as lateral pressure on the facing is not valid. This assumption has been discussed already by Pachomow & Herold (2009) and Vollmert et al. (2013). The latter has shown that just approx. 70 % of the active earth pressure has to be taken into account even for stiff concrete panels. For other systems like wrap-around technique, gabion basket facings etc., reduction factors for reduced earth pressure are given by

EBGEO (Recommendations for Design and Analysis of Earth Structures using Geosynthetic Reinforcements). It has to be taken into consideration here that the active earth pressure is just of help using existing formulas, but is not a scientific description of the soil-mechanical correlations as anticipated.

2.2 *Test site KWS Utrecht*

In addition to large-scale tests on full-height panel walls in laboratory conditions (Pachomow & Herold, 2009), a full-scale trial has been performed by KWS Utrecht, Netherlands, supported by NAUE, Germany. The test site of the KWS cube in Utrecht has already been described by e.g. Vollmert et al. (2013). To reduce the required amount of structural steel for the construction, the steel required for the panel has been designed optimized for transport forces only.
To make the system as easy as possible, only 1.5 m long strips of the reinforcement have been pre-casted to the concrete (Fig. 2). The required length of the reinforcement to fit the overall safety according to EBGEO has been placed on site, just overlapped by friction. For the earth pressure distribution satisfactory pull-out resistance of the strips (precast into the concrete) from the reinforced soil structure has to be ensured.
While under laboratory conditions high loads up to 450 kPa could have been applied to the structure, the influence on installation procedure, weathering (changes of moisture content and temperature) and long-term effects caused by the thermoplastic characteristics of the used high strength Polyester geogrid can be investigated in situ. Fig. 2 gives the cross section of the test set-up for the in-situ test with geometry comparable to the laboratory test by Pachomow & Herold (2009).
Several types of instrumentation have been used, taking the static principle of $\Sigma H = 0$ into consideration. The sum of forces acting on the backside of the panel shall be equal to the sum of forces acting on the geogrids, the temporary prop and the friction on the toe of the panel. Therefore, the toe of the panel has been designed as plain bearing, using geosynthetic components for sliding purposes with tested and well-known friction parameters for back-analysis of the forces acting at the toe.

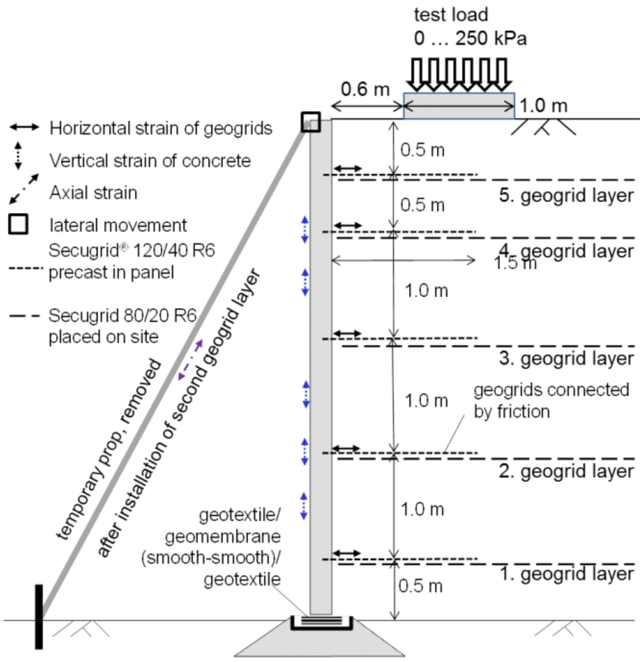

Figure 2: Test setup full-height panel wall, KWS Utrecht

Temporary wooden wedges, applied during placing the panel, have been removed after the installation of the first 2.0 m of backfill material, so lateral movement of the panel toe was possible, but has not been observed. Up to a fill level of 4.0 m the panel was supported by a temporary prop, removed afterwards. For the installation of the full wall different steps of deformation of the wall have been expected due to the change within the static system by removing the temporary prop.
After a period of 2.5 years in service the KWS cube has been loaded on site by calibrated test weights up to a pressure of 250 kPa on top of a cast-in-place concrete beam of 1.0 m width with a distance of 0.6 m from the backside of the facing. Prediction of strains and deformations have been

computed in advance as the load of 250 kPa has been found to be the maximum allowable load according to EBGEO for the existing structure by back-analysis, assuming conventional failure modes in the Ultimate Limit State (ULS).
The predictions have been made using Finite Element calculations (Plaxis 2D) and assuming Hardening Soil Model with small strains (HSS) as well as Mohr-Coulomb (MC). Varying the stiffness of the sandy soils used as fill material in a realistic range, the expected lateral deformation at maximum load was within the range of 3.5 cm to 4.5 cm.

Figure 3: Test load of 250 kPa at the test site KWS cube, Utrecht, using calibrated weights

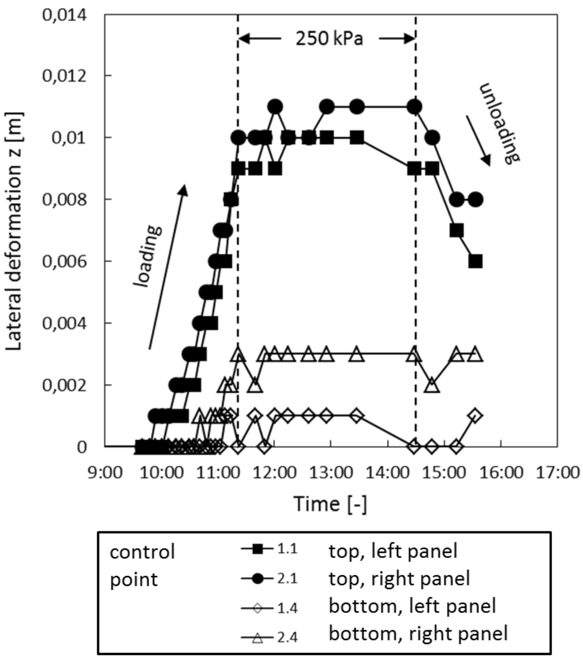

Figure 4: Measured lateral deformation during the test duration at the test site KWS cube, Utrecht, exemplarily given for the top and the bottom of the panels in the center line, left and right panel each

In fact, the measured deformation has been measured to be maximum 11 mm as given in Fig. 4. As it can also be seen from Fig. 4, the bottom deformation has been found within the uncertainty range of geodetic survey, but showing a very limited reaction. For the readings at the top of the panel facing the deformations are significantly less compared to the predicted ones and do not show any significant differences between the left and the right panel element.
It is also of interest that obviously approx. 30 % of the lateral deformation is of elastic characteristic in the limited time of loading period. If creep deformations would have been of significance, they are expected to be found in the readings at the top, but have not been found.
The corresponding strain within the geosynthetic reinforcement remained within the range measured in the 2.5 years of service period of the unloaded structure with a maximum strain of approx. 0.7 % in the first layer (Fig. 2), due to the installation process. Applying the test load, the strain recordings gave an increase of max. 0.2 % strain in layer 1 to 3 with an elastic amount of approx. 70 %, what is vice versa to the deformations, interestingly. Layer 4 and 5 are not significantly loaded, neither during the installation process and time in service, nor during the test load. The recordings of layer 5 even show depression instead of elongation what can be explained by the fact that the strain gauge is outside the load distribution angle of the test load.
The settlement of the cast-in-place concrete slab has been recorded to 5 mm maximum at 250 kPa in opposite to a computed settlement of up to 7.5 cm depending on the assumption made and model used.
The measurements show again, as already described for the installation phase, that the construction using an active panel with friction connection of the reinforcement shows a very stiff behavior and that computations worked out using typical parameter for the reinforcement and soil might give very conservative results.

3 REFERENCE PROJECT BOELE STAAL N237

3.1 *Project description*

Nature in the heart of the *Utrechtse Heuvelrug* is highly fragmented and is under great pressure due to the need for residential and office locations. The program *Hart van de Heuvelrug* was founded to strengthen nature with creating space for living and working. The construction of five wildlife crossings is part of this program, including *Ecoduct Boele Staal*. This crossing will be part of an ecological corridor connecting the *Leusderheide* with the northern part of the *Utrechtse Heuvelrug.*

3.2 *Project requirements*

By realizing Ecoduct Boele Staal, wildlife will be able to safely cross the Dutch national road N237, its bicycle paths and sidewalks. Alongside the sidewalks cables and ducts are located which need to be accessible for maintenance or renewal.
The crossing has a span of approximately 38.0 m, a height of 4.6 m and a width at the crossing itself of 60.0 m. The effective distance of the total crossing that has to be covered by wildlife is reduced by increasing the width of the approaches on both sides, using a total of 110.000 m^3 of ground. By doing so, the factor of length over width improves which optimizes the crossing for red deer.
In order to minimize the difference in vegetation between the direct surroundings and the wildlife crossing a minimal ground cover of 1.0 m is required.
The object has a lifespan of 100 years, in which the allowed settlement over the first 30 years may not exceed 0.15 m.
Furthermore, an expression of art is demanded on the walls adjacent to the sidewalks alongside the N237. The total project should be realized with a minimum of hindrance to traffic and its surroundings.

3.3 *Draft concept*

Because of the large span in combination with a groundcover of 1.0 m, girders ZIPXL1600 are needed. These girders will be supported at both sides on a foundation beam with a length of 61.0 m and a width of 4.5 m, resulting in a subgrade reaction of 258 kPa (SLS).
The subsoil at the project location (Soesterberg) is characterized by a medium dense to dense sandy soil with a very low water table which makes a shallow foundation feasible.

3.4 *Reinforced bridge abutment*

The deck will be supported by a reinforced soil abutment with geogrids behind a facing of concrete panels. Geogrids with a length of approximately 2.0 m are cast in these panels which also act as formwork during construction of the reinforced soil. As no spacing between the concrete panel facing and the warp-around method used is present due to the use of the concrete panel as temporary facing it also acts as active facing. By adding the wrap-around method in the back of the panel a conservative and very robust building environment is created where no additional measures to prevent failures have to be taken into account. Thus an active and a passive construction are combined here by combining the advantages of both of them.
During construction the panels are temporarily supported reducing hindrance to an absolute minimum. An additional advantage of the use of full-height concrete panels is the possibility of adding an art expression on the facing of the panel.
The concrete panels are placed on a L-shaped retaining wall, making both an accurate positioning of the panels and allowing for future excavations in front of the abutment (Fig. 5).

3.4.1 *Design of geosynthetic reinforcement*

The design of the reinforced soil structures used for the retaining structures (side walls) as well as for the bridge abutment usually has to be worked out in accordance with the Dutch version of Eurocode 7 (NEN-EN1990+A1+A1/C2:2011) and its national annex NEN 9997-1+C1:2012. As the Eurocode in general does not give special reduction factors for geosynthetic reinforcements, national design codes as e.g. CUR 198, BS 8006 or EBGEO have to be used to ensure satisfactory safety levels.

In this specific case it has been decided to work with the German design code EBGEO in accordance with Eurocode 7 and the national German annex as the Dutch design code for geosynthetic reinforcements CUR 198 is currently under revision and significant progress is expected.
EBGEO works with combined failure mechanisms, all possible failure modes have to be checked. Furthermore, EBGEO gives load distributions for the earth pressure on the backside of facing elements depending on the stiffness of the facing itself. This is of significant advantage for this structure as it is known that the distribution of lateral forces within reinforced soil structures is undefined and cannot be determined assuming active earth pressure.
The resulting design load according to EBGEO as given in Fig. 6 is used to design the layout of the steel reinforcement for the panels as well as the connection details for the geogrid reinforcement used to back-anchor the panel to the reinforced soil construction. A series of pullout tests for the used type of geogrid has been performed in concrete with different layout to evaluate the required anchor length of the geogrid within the concrete. The length for anchorage of the used reinforcement Secugrid® R6 in concrete reinforcement shall be approx. 20 cm and requires a reduction of connection strength of 1.16 to 1.5 depending on the nominal strength of the product. The detail finally used is given in Fig. 7.

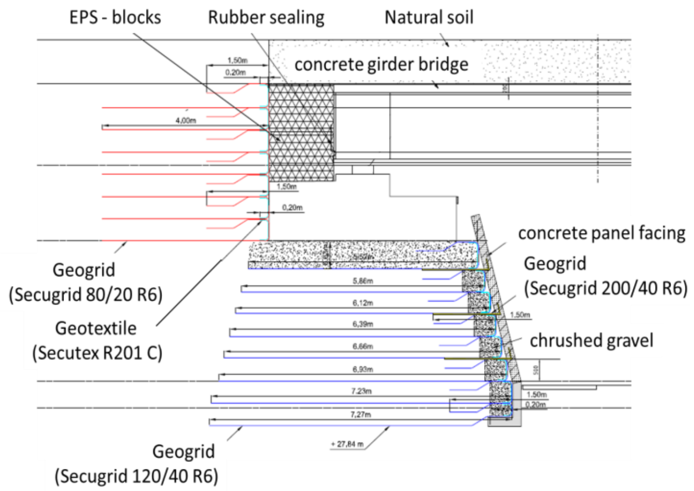

Figure 5: Cross section of the geogrid-reinforced bridge abutment

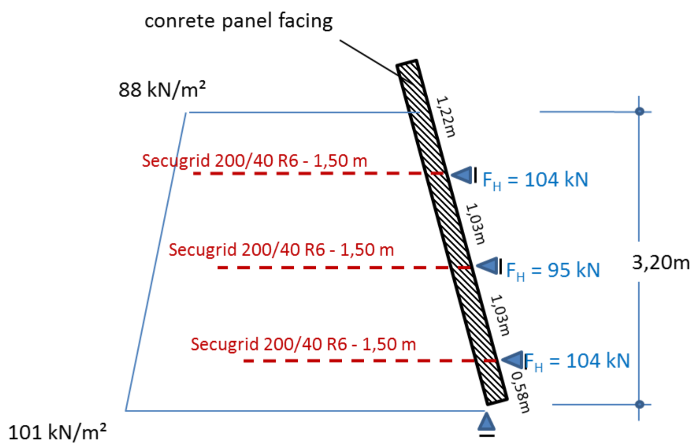

Figure 6: Example of characteristic loads for panel design

Figure 7: Implementation of the high-strength geogrid to the concrete reinforcement of the panel during the precast process

Also, the geogrid has been found to withstand the high pH-values of the concrete due to the monolithic character of the bars the grid is made from. The required reduction factors A_4 for this effect($A_{4,\text{ Secugrid R6, pH 12.5}} = 1.18$) as well as for installation damage are given by certificates according to CUR, EBGEO and BBA.
The resulting concrete reinforcement is based on impact calculations as no protection barrier is planned in front of the wall.

3.4.2 *Serviceability calculations*

Despite the high foundation pressures extensive calculations have been worked out to evaluate the expected absolute settlements as well as the differential settlements for the bridge itself, but also for the panel facings to prevent tilting and rotation in an unacceptable range.
Fig. 8 shows a representative cross section of the structure with information on expected settlements computed for triangular elements in the foundation plane of the structure with mutual influence of each point to each other (GGU settle) while the overall maximum has been calculated to remain within a range of 60 mm for a 60 years period. The settlements in longitudinal direction of the panel facing, relevant for rotation and tilting against each other, have been calculated with the same system to be in the range of 30 mm without differential settlement.
Fig. 9 gives a qualitative distribution of lateral displacements calculated by 2D finite element analysis (PLAXIS) taking each construction step into consideration. The maximum differential deformation for the facing can be given to less than 10 mm what is supported by the measurement results from laboratory (Pachomow & Herold, 2009) and the KWS cube project in Utrecht, 2014.

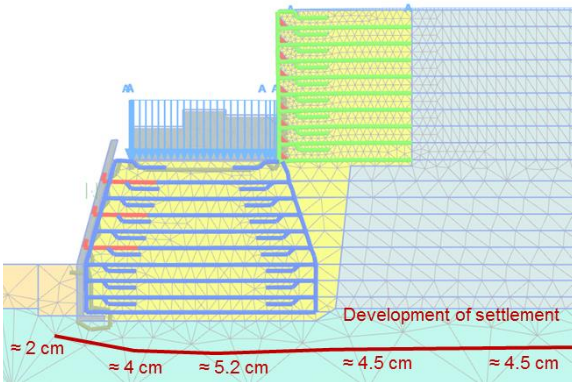

Figure 8: Development of maximum settlement, cross section

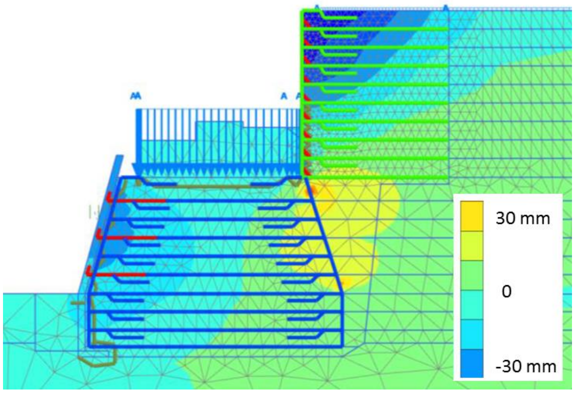

Figure 9: Cumulated lateral deformations, starting from first construction phase to final stage

4 CONCLUSIONS

As a result of intensive research and full-scale tests, it has been shown again that reinforced soil constructions react exceptionable stiff and ductile. Therefore, they are well suitable as to be used for bridge abutments where the decks are based on the reinforced soil construction itself. The variation of facings gives preference to gabion baskets in combination with wrap around technique where the static and structural elements are non-interacting elements allowing for maintenance works (Herold & Vollmert, 2013) or full height panel facings with the same benefit, but options for architectural variations by precast elements. In the project presented the later version has been used. The advantage of the installation process of active panel elements is used and combined with the additional safety of passive facings by combining both techniques. The full-scale test at the KWS cube gives reliable information and trust on the serviceability of the structure and elements used in combination, allowing for an improved, optimized design.

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