# Project of the Highest Segmental Block Wall in Brazil

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ABSTRACT
This study describes the main aspects considered in the project of a segmental retaining wall reinforced with geogrids and with facing in Terrae blocks extending 250 m in length, and with a maximum height of approximately 25 m. In addition to the design challenges of a retaining structure of this magnitude, the local geological characteristics, with the presence of low bearing capacity of soil in the foundation, the operating road during the construction phases and the extremely tight executive schedule, make this work practically unique. In this job the treatment of foundation soil comprised the execution of stone columns in 2,500 m² of area, excavation of slopes up to 25 m, installation of 120,000 m² of geogrids, 5,000 m² of facing blocks and a total fill volume of 130,000 m³. The technical project for the retaining wall was developed by the company Engecorps Engenharia S.A., and the construction, carried out within a period of less than 6 months, was conducted under the supervision of the concessionary company Nova Tamoios.

RESUMO
Neste estudo são descritos os principais aspectos considerados na elaboração do projeto de um muro em solo reforçado com geogrelhas e face em blocos tipo Terrae, com 250m de comprimento e altura máxima de aproximadamente 25m. Além dos desafios relacionados ao dimensionamento de uma contenção dessa magnitude, as características geológicas locais, com a presença de solos com baixa capacidade, a rodovia operante durante as fases construtivas e o cronograma executivo extremamente apertado, tornam esta obra praticamente única. Nesta obra foi indicado o tratamento da fundação com colunas de brita em 2.500m² de área, escavações de taludes com desníveis de até 25m, instalação de 120.000m² de geogrelhas, 5.000m² de face em blocos e um volume total de aterro de 130.000m³. O projeto técnico da contenção foi desenvolvido pela empresa Engecorps Engenharia S.A., e a obra, executada em um período inferior a 6 meses, conduzida sob supervisão da Concessionária Nova Tamoios.

## 1. INTRODUCTION

The construction of geosynthetic-reinforced soil retaining walls currently corresponds to the most common retaining solution implemented in North America, Europe and Japan. In Brazil, this technology has been used for decades and the construction of reinforced soil retaining walls remains in solid growth.

Among the main advantages of reinforced soil retaining walls are simplicity and constructive speed, ease of acquisition and control of the properties of the geosynthetic materials, overall performance and final aesthetics of the structure. When compared to metallic strips, the geosynthetic reinforcements cost considerably less, and allow for the use of soils with higher gradation (larger fraction of fines) – in general local soils – along with the use of commoner equipment for soil compaction and for lifting concrete elements.

This study presents the main aspects considered in the elaboration of the Tamoios Highway Toll Station P2 implementation project, which comprised the execution of a soil retaining wall reinforced with PVA geogrids and block facing, with approximately 250 m in length and maximum height of 25 m. This is the largest work in height built in Brazil with segmental block face system of the type Terrae.

The implementation of the toll station is part of a project comprised by a set of improvements and increase in capacity of Tamoios Highway, developed by Engecorps Engenharia S.A., under the supervision of the concessionary company Nova Tamoios.

## 2. AREA OF IMPLEMENTATION

### *2.1 Location*

Tamoios Highway (SP-099) is the main route connecting the Paraíba Valley with the northern coast of São Paulo, thus connecting the city of São José dos Campos to Caraguatatuba. The highway is approximately 80 km long, 65 km of which being in plateau and the rest in mountain range.

With the beginning of the highway concession contract in 2015, studies and projects were initiated for the improvement and expansion of the highway capacity. In this context, the construction of three new toll plazas were demanded, including the implementation of the P2 toll plaza, located at km 59 + 300, in the region of Paraibuna – SP.

The area of implementation of the P2 toll was selected after a comparative analysis with three other areas located between km 56 and 61 of the highway. In this comparative matrix, several aspects were evaluated, such as: presence of escape routes, environmental impacts, interference with the user during construction, operational safety and construction costs.

Based on these aspects, the area located in the region of km 59+300 was the one that presented the best overall performance, being therefore selected for the construction of the structure. Figure 1 shows the area of implementation of Toll Station P2.


Figure 1. Toll Station P2 construction site (source: Google Earth).

### *2.2 Relief/Topography*

The region has extensive vegetation cover and relatively rugged relief consisting of several hills and valleys near the backwater / influence areas of the Paraibuna reservoir.

At km 59+300 the highway is in half-slope section, with a 50-meter high cut slope on the west bank (coastal direction), and a 20-meter natural slope on the east bank (city direction) where the retaining structure would be constructed.

### *2.3 Geology*

Regionally the area of interest is inserted in the geotectonic context of the Coastal Domain, formed by a metamorphic terrain located between the Cubatão shear zone and the Coast.

In the area of implementation of the Toll Station P2, there are metamorphic rocks of the Coastal Complex that have as lithotype: biotite, migmatite and / or porphyroclastic granite gneiss.

### 2.4 Site Investigation

For characterizing the soil, two investigation campaigns were carried out. In the first campaign, 4 SPT surveys were performed, while in the second campaign, 15 SPT percussion probes and 6 CPTU static probes were performed to enhance the details of subsurface information.

The location of the boring holes sought to characterize the subsoil throughout the entire projection of the toll station but with greater concentration in the alignment of the face of the retaining wall, since in this region, apart from having less removal of the surface layer, there is a higher concentration of stresses originating from the reinforced soil mass. Figure 2 indicates the upper view with the location of the surveys carried out for the job.

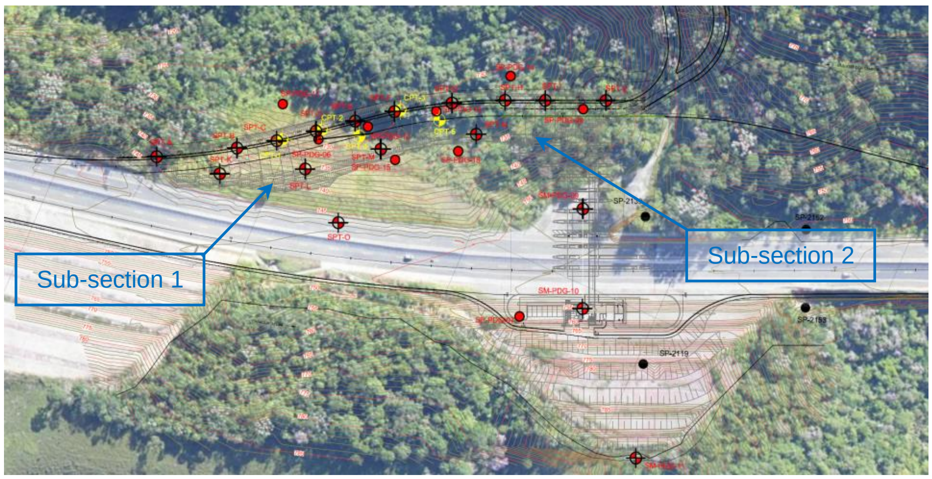


Figure 2. Location of boring holes in Toll Station P2.

The results of the investigations indicated that the region presents a subsurface profile with relatively distinct soil resistances, depending on the position.

In the region of the parapets (higher elevations) there is a small cover of colluvial soil or existing fill, while more competent residual soil layers (NSPT≥10 blows) can be found at lower depths. In this region, the ground water level was verified to be at depths greater than 6 m.

In the central portion, located in the bottom of the valley, there was an alluvial deposit with an average thickness of 5 m, NSPT values ranging from 2 to 9 blows, and composed of clayey soils with boulders and organic matter. Immediately below this layer, there was a silty residual soil with NSPT less than 10 blows up to approximately 12 m of depth and increasing in shear strength from this depth. The water level in this region was found to be near the surface.

Figure 3 shows the geological/soil profile in the central portion which represents the most compressible region.

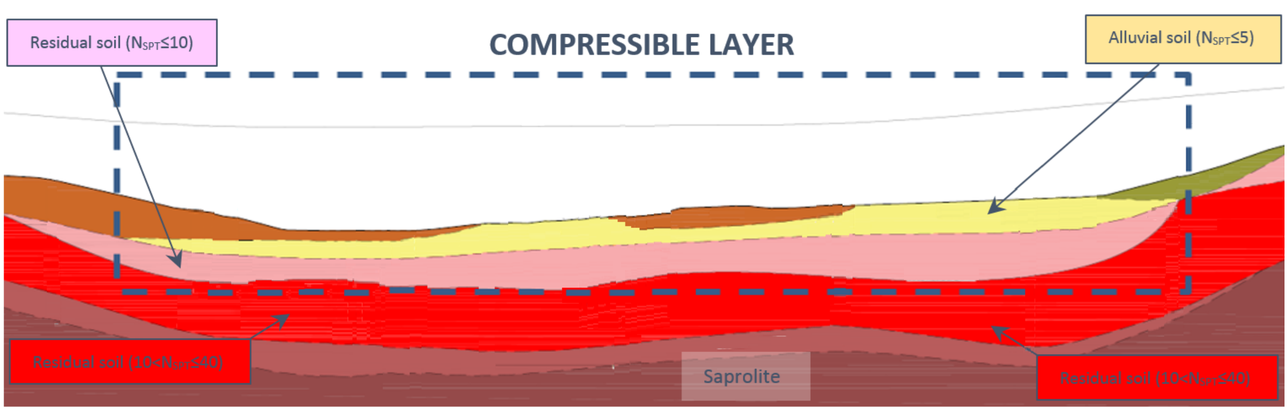


Figure 3. Detail of the geological/soil profile and the compressible layer.

## 3. EXCAVATION PROJECT

The Toll Station P2 job comprises the construction of a 15-lane toll station, including the widening and tapering section of the 860 meter-wide lanes.

For the implementation of these works, projects were prepared for the excavation of the final slopes, located above the highway level, and the excavation of the temporary slopes, located below the highway elevation.

### *3.1 Excavation project criteria*

For the definition of excavation sections, the criteria of the standard NBR 11682 were adopted, which require a factor of safety of 1.30 (FSPROV≥1.30) for the temporary condition, and 1.50 (FSDEF≥1.50) for the final slopes. The analyses were performed by the Limit Equilibrium Method, with the aid of Rocscience Slide software, version 6.0.

The parameters of the materials were estimated from similar soil tests obtained from nearby regions and correlated with the results of the investigations at the toll site, especially for the CPTu results. The parameters are shown in Table 1.

Table 1. Provisional excavation - Soil parameters.

| Layer | SPT | $\gamma$ (kN/m$^3$) | c' (kPa) | $\phi$' (°) |
|---|---|---|---|---|
| Alluvial soil | <6 | 16 | 10 | 20 |
| Existing Fill | - | 18 | 20 | 26 |
| Compacted Fill | - | 19 | 20 | 30 |
| Residual Soil I | <10 | 17 | 15 | 25 |
| Residual Soil II | 10 a 15 | 17 | 20 | 26 |
| Residual Soil III | 16 a 20 | 18 | 20 | 30 |
| Residual Soil IV | 21 a 40 | 19 | 25 | 32 |
| Residual Soil V | > 40 | 20 | 30 | 35 |

For disclosure only, since the final slopes (located above the road's elevation grade) have no significant influence on the retaining structure, they were built with a 1V:1H inclination, with berms located every 8 m and a total height of up to 50m.

### *3.2 Temporary Excavations*

In the region below the highway level where the retaining wall was planned to be built the project of the provisional excavations was developed to build the reinforced soil in the required dimensions and geometry, meeting the requirements of external stability and internal stability of the structure.

As a basic premise, temporary excavations should keep the highway operational throughout the construction period. In addition to the road operation, it was established that the longitudinal sections should be multiples of 5 m (geogrid width), and the step height should be a multiple of 60 cm, as a function of the vertical spacing adopted for the reinforcements.

Due to the large amount of the mass to be excavated (about 250 m), the variability of the foundation ground, and the need to clear construction fronts, the temporary excavation project was divided into two sub-sections, and several scenarios were evaluated combining these executive premises to the factors of safety necessary for the construction.

In sub-section 1, located between km 600+00 and 606+10, greater complexity was observed due to the space restriction between the environmental clearance lane and the highway, the greater difference between the base and the top of the excavation (facing the highway) and the thicker layer of low bearing capacity material at the toe of the slopes. In sub-section 2, located between km 606 + 10 and 613 + 4.5, due to the greater distance from the highway and the presence of stronger materials in the foundation ground, no significant problems were observed.

Hence, for sub-section 1, a cross section was adopted with 3-meter wide berms every 6m in height, and in the critical section (larger elevation difference), it was recommended to widen the last berm (width of 6m) to relieve the load on the top. For sub-section 2, berms with the same dimensions indicated in sub-section 1, matching the cross sections, however with a width of 1 m were adopted.

Figure 4 shows the result of an analysis performed for the definition of the temporary excavation cross section (FSprov≥1.30) in the sub-section 1 region (highest gap).

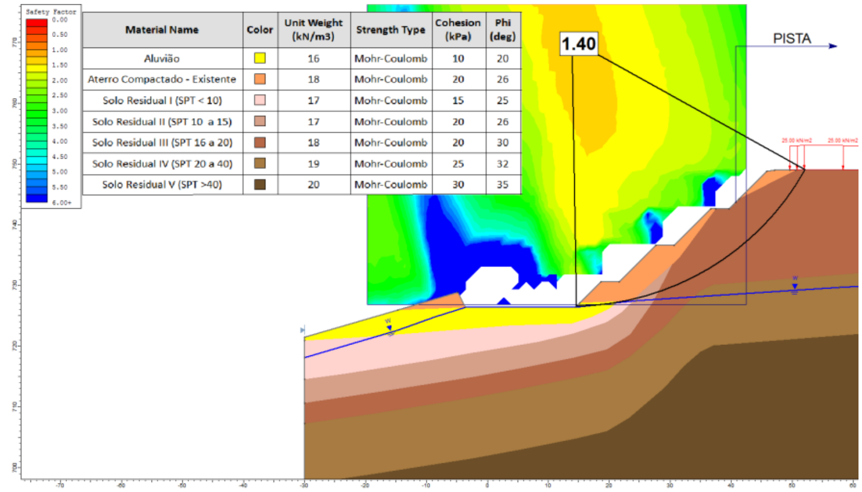


Figure 4. Temporary excavation - Stability analysis.

In Figure 5 a) it can be observed the general appearance of the excavations in the Sub-section 1 region, near the highway limit, while in Figure 5 b) it can be observed in more detail the execution of the levels for the installation of the geogrids at the base of the Sub-section 2 region.

a) Temporary excavations in sub-section 1

b) Temporary excavations and geogrids in sub-section 2

Figure 5. Excavation at the base of the retaining wall

## 4. FOUNDATION TREATMENT

Because of the compressible layers in the local terrain and the magnitude of the acting loads, a project was developed for the treatment of foundation soil, which sought to establish a solution to minimize the total stresses and strains at the base of the reinforced soil, guaranteeing the factors of safety required for implementing the work.

Considering the high thickness of compressible material (> 10 m) and the presence of water close to the surface, the alternative treatment consisting of soil substitution of the foundation material was discarded. Solutions with rigid inclusions (piles) proved to be costly and time consuming, while chemical treatments such as jet grouting proved costly and would require care with the waste generated near the environmental preservation area.

Thus, the execution of stone columns to improve the foundation was opted, reducing the deformations in the base whilst increasing the overall performance of the soil body against the acting forces.

### *4.1 Performed Analyses – FEM*

For treatment of foundation soil was development a project with stone columns, a design of the column mesh was carried out with the charts from Priebe (1995), verifying the increment in the parameters required for the foundation ground as a function of the influence of the columns.

Once the improvement factors were estimated for each of the evaluated meshes, Finite Element Method (FEM) analyses were performed to evaluate the performance of the treated foundation (strain stress), with the aid of Phase2 software, version 7, and complemented with global stability analyses, performed with Slide software, version 6.

Figure 6 shows a cross section analyzed by FEM with the Phase2 software.

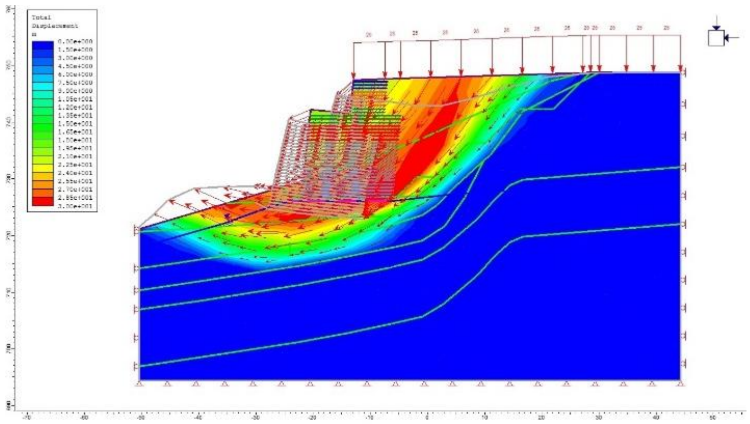

Figure 6. Stress strain analysis by FEM.

Figure 7 shows a global stability analysis with the Slide software.

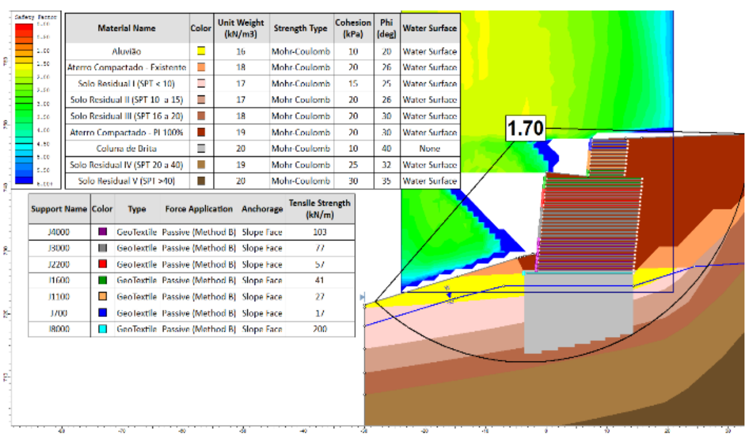


Figure 7. Overall stability analysis (post treatment).

### *4.2 Stone Columns Characteristics*

The analyses indicated the need for treatment in the central portion, between the marks of 603+00 and 611+00, which represents the area where the highest loading and the presence of the most compressible soils were verified. The ground improvement with columns should be performed in a length of 160 m, resulting in an area of about 2,500 m².

As a performance premise, after analyzing the technical literature, it was established that the treatment design should result in a maximum total settlement of 1% of the total wall height (H ≈ 25 cm) and the differential settlement should be limited to 1 / 300.

In conclusion, the stone columns were designed with a minimum diameter of 0.80 m and executed in a 1.6 m x 1.6 m triangular mesh. The average length of the columns was 12 m guaranteeing that their tip was embedded in a material with NSPT≥10 blows should be guaranteed.

In a reinforced soil structure, the block face is the component most sensitive to settlement. Considering that in the front portion there is a loss to the confinement and performance of the columns, given the absence of soil (overburden) in front of the wall, it was recommended that the last 4 m of the columns (top) located directly under the block facing system should be performed with a mixture of gravel and cement, stiffening the stem of the columns in this region.

In addition to the execution of the columns, the treatment of foundation soil also included the construction of a compacted gravel mattress enveloped with a high strength geogrid under the entire base of the reinforced soil (including over the top of the columns). Such solution aims to distribute the loads more evenly and minimize strains at the base of the soil mass.

Figure 8 a) shows the construction of the stone columns, and the Figure 8 b) illustrates the execution of the gravel mattress wrapped by a geogrid.

a) Execution of stone columns

b) Execution of gravel and geogrid mattress

Figure 8. Ground improvement at the foundation of the retaining wall

## 5. REINFORCED SOIL WALL

The reinforced soil technique consists of elevating a retaining structure in compacted soil, inserting inclusions (reinforcements) with strength and predefined spacing within this body of soil.

About the retaining face system, a more flexible (bags or wire mesh) or more rigid solution can be adopted, using concrete pieces / blocks, positioned and fixed to the reinforcement elements (inclusions) of the reinforced soil.

Avesani and Geroto (2016) point out that, in the case of the construction of very high geotechnical retaining structures, such as the work in question, the reinforced soil solution becomes almost exclusively the only technically viable alternative.

### *5.1 Design and Characteristics of the Retaining Wall*

According to Ehrlich et al. (2015), for the design of reinforced soil retaining structures, external stability analyses should be performed, such as: slide verification, overturning verification, bearing capacity and overall stability; and the internal stability analyses, related to the design of the reinforcements with respect to the tensile and pullout efforts.

For the project in question, the analysis of internal stability (reinforcements) was performed according to the methodology employed by FHWA proposed by Mitchell and Villet (1987), where the required tensile strength of the inclusions (geogrids) is determined by the method of buoyancy, where each reinforced soil layer receives proportional to its vertical position, and a reduction factor is applied to determine the characteristic geogrid resistance required in each layer.

To determine the length required for inclusion, the acting forces are evaluated and, considering a hypothetical sliding wedge of soil, the minimum anchorage length of the geogrid (outside the sliding wedge) is determined which, added to the length of the geogrid in the internal section to the wedge, results in the minimum length of the geogrid.

In the construction of the reinforced soil of the Toll Station P2, PVA geogrids were used as reinforcement elements with tensile strength ranging from 35 to 200 kN/m, and vertically spaced by 60 cm.

The advantage of applying this type of geogrid is related to its ultimate tensile strength, obtained with 5% strain, minimizing possible displacements throughout the reinforced structure and especially on its face. This type of polymer also has greater tolerance to extreme values of pH, a necessary condition for the work, which employed lime-enhanced soil in the construction of the fill.

The facing was built with a 10V:1H slope, consisting of Terrae-W blocks with compression strength values of 6, 12 and 18 MPa. It was opted for the solution in rigid blocks to ensure greater uniformity (less deformation) in the face system, given the great height of the containment.

In Figure 9 you can see a typical cross section with the details concerning the treatment of the foundation, block facing system and the various installed geogrid tensile strengths.

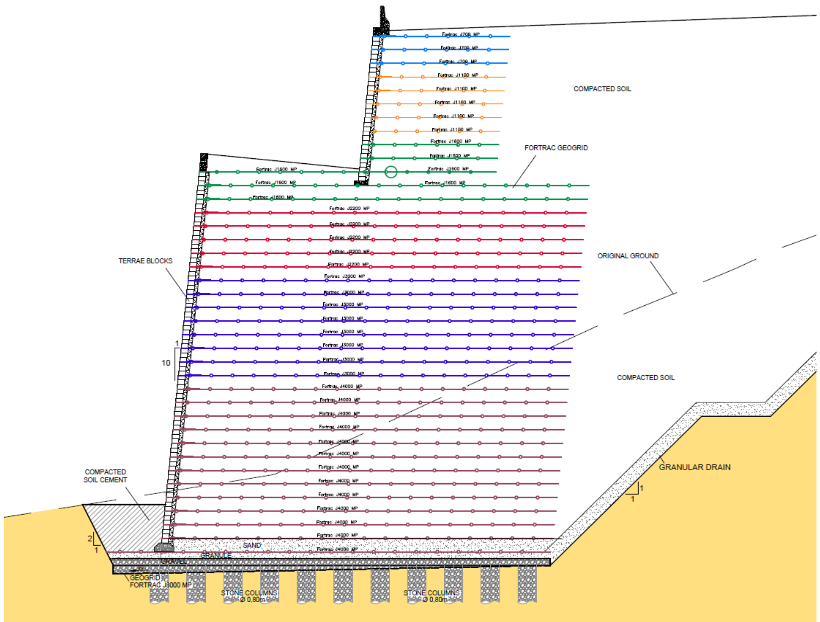


Figure 9. Cross section of reinforced soil structure.

In relation to the backfill soil, a material obtained in a deposit near the work site, composed predominantly of a sandy silt, was used. The available material presented a fraction of fines higher than the recommended (> 40%), with plasticity index also above the desirable. Thus, the soil stabilization with lime was performed to improve its properties, besides facilitating the execution of the work in periods of high rainfall, where it is more difficult to stabilize the material at the optimum moisture content.

For the construction of the massif in reinforced soil, approximately 120,000 m² of geogrids, 5,000 m² of block face Terrae-W were used and about 60,000 m³ of reinforced soil were made, out of a total volume of 130,000 m³ of compacted soil used in the construction of the whole structure.

Figure illustrates the construction of the massif in two distinct stages.

a) Execution of the earthwork near the base b) Execution of the earthwork near the top

Figure 10. Construction of the massif in two distinct stages

Considering the extremely tight executive schedule, a solution was adopted which consisted of the installation of a cover (tent) for the entire length of the work, allowing the continuation of earthwork services even during the rainy season. This solution, combined with the constructive agility of the reinforced soil technique, allowed the work to be carried out in a period of six months, with its conclusion in May 2016.

In Figure 11 a) it is possible to see the plateau for earthworks inside the roof (tent), while Figure 11 b) gives an overview of the site with the cover installed near the Tamoios Highway.

a) Execution of earthworks inside the roof (tent)

b) Overview of the site with cover (tent)

Figure 11. Execution of earthworks with cover (tent)

During and after the construction of the reinforced soil, the monitoring / instrumentation of the containment was performed with the installation of markers (prisms) for verifying and recording the displacements (vertical and horizontal) on the face of the containment.

At the client's choice, an alternative treatment of foundation soil solution was implemented to the design, also consisting of stone columns, but with the use of shorter elements / columns with greater spacing between the elements (coarser mesh).

Considering the adoption of this more flexible treatment, displacement values above those established in the original project were observed in the monitoring.

Figure 12 a) shows the final construction of the retaining wall and Figure 12 b) gives a face overview of the wall.

a) Final construction of the retaining wall

b) Face overview of the wall

Figure 12. Final construction of the retaining wall

Figure 13 a) presents the final aspect of the retaining structure upon its completion, and the Figure 12 b) illustrates the toll plaza in operation upon completion of the works.

a) Final aspect of retaining wall and toll plaza    b) Toll plaza completed in operation

Figure 13. Final aspect of a retaining wall at the tool plaza

## 6. FINAL REMARKS

In the present study, the main aspects considered in the development of the project of a geogrid reinforced soil and segmental block face retaining wall, presented at km 59 + 300 of Tamoios Highway, were presented.

For this project, a retaining wall was constructed on soil reinforced with PVA geogrids and face in segmental blocks, with approximately 250 m of extension and maximum height of the order of 25 m. This is the highest retaining wall built in Brazil with segmental block face system of Terrae type.

Several challenges were encountered in the development of the project, such as the presence of low strength soil in the foundation, the magnitude of the structure (height and extension), the operating road during the construction phase, unfavorable weather conditions and extremely tight schedule.

In this context, the construction of the retaining wall using the technique in reinforced soil proved to be the correct choice, and after two years of its completion the structure presents a fully satisfactory performance.

## ACKNOWLEDGMENTS

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