2022

EXEO CAMPUS OFFICE. FOUNDATIONS AND EARTH RETAINING STRUCTURES. LISBON

General characteristics

Location: Lisbon – Conclusion year: 2022
Type: earth retaining structures and deep foundations
Purpose: office and commercial buildings
Maximum depth: 12 m – Excavation Volume: 300 000 m^3
Diaphragm wall area: 11.500 m^2 – R.C. driven piles foundations: 5 000 m
Geological conditions: landfills and Lisbon Miocene, mainly: sands and clays, with average low permeability
Geotechnical instrumentation sensors: 15 geoSmart / edgeHub, 10 load cells MEMS and 9 tiltmeters
Owner: AVENUE – Main Contractor: HCI
Geotechnical Contractor: Terratest – Designer: JETsj Geotecnia
Monitoring: JETsj Geotecnia / Senceive / Soil Instruments

Solutions descriptions

- Main objectives

The EXEO Campus Office buildings consist of 3 plots with maximum excavations depths ranging from 12m at plot 1 to 8m at plots 2 and 3. The proximity to the Lisbon Metro Red Line tunnel at the Oriente Station entrance, at a variable distance between 5m and 20m, was the main constrain, that led to an important risk management and control of the Lisbon Metro tunnel, as well as of the surrounding excavation area (Figure 1). Within this framework, the observational method, supported by a full-automated instrumentation and monitorization plan was adopted, allowing the permanent control of the Lisbon Metro tunnel behavior. Thus, for each main excavation stage, a back analysis and evaluation of the ground parameters was performed. The large volume of data generated, sometimes readings at every 5 min, allowed a considerable amount of data and thus the reliability increase of the most relevant geotechnical parameters characteristic values.

Figure 1 – Site plan location (left hand side) and aerial view (right hand side)

- Geological conditions

At the surface the excavation intersected the quaternary deposits (Holocene), consisting of landfills and soft soils with variable thickness up to a maximum of 10m, covering the Lisbon Miocene, with interbedded layers of silts, sands and clays. Holocene formations have weak to medium resistance and deformability characteristics with PMT test limit pressure values between 0.10MPa and 1.30MPa. The water content of these formations is between w = 17.9% and 28.9 %. The Lisbon Miocene has characteristics of resistance and deformability, hard to very hard and very compact, with limit pressure values of the PMT between 0.70MPa and 3.0MPa. These formations have a water content between w=19% and 23%. (Figure 2).

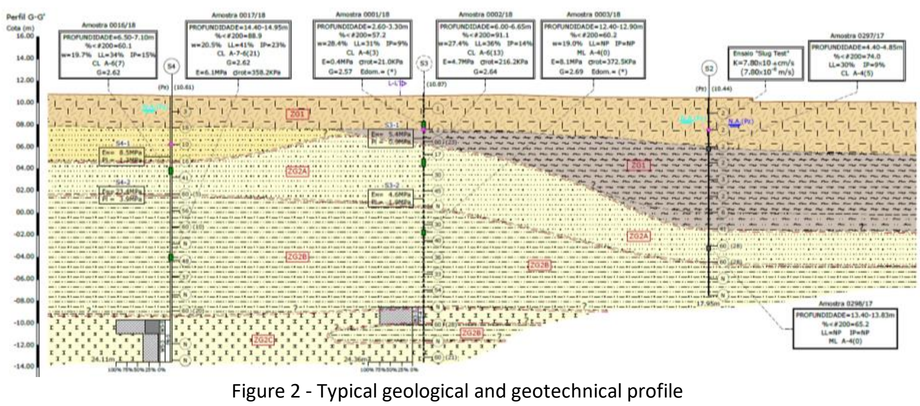


Figure 2 - Typical geological and geotechnical profile

- Design and general description of the earth retaining walls solutions

For all the 3 plots diaphragm walls were built as earth retaining solution with 0.60m thickness, braced in general by temporary ground anchors and corner steel struts. For the plot 1 and at the Lisbon Metro tunnel alignment, the diaphragm wall was braced by buttress performed by barrettes, with the objective to avoid the execution of temporary ground anchors, due to the Lisbon Metro tunnel small distance.
To model the earth retaining structures and Lisbon Metro tunnel behavior, stress-strain models were developed with the objective to predict the walls deformation, considering the reference value of h/500 = 24mm for horizontal displacement, as well as the horizontal and vertical limits imposed by the Lisbon Metro.

- Design and general description of the foundations solutions

As foundations solution for the plots 2 and 3, given the presence of Holocene and Miocene layers, the option was a CPRF (combined piled-raft foundations) solution to better control the possible differential settlements. At the more loaded regions (internal walls), a barrettes solution 0.8m x 2.6m up to a variable depth between 8 and 12m was adopted. At the remaining areas, reinforced concrete driven precast piles, type Terratest T270, with variable depth between 6 and 8m were installed. For plot 1 a simple reinforced concrete mat foundation solution was adopted.
The foundations numerical models were developed using Plaxis and BIM Revit software and calibrated with preliminary dynamic load tests (PDA) results, to previously evaluate the precast piles deformability and bearing capacity, as well as their integrity in relation to the impact hydraulic hammers energy (Figure 3, Figure 4 and Figure 5).

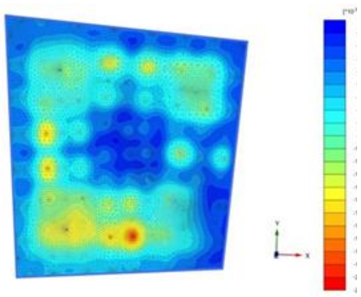


Figure 3 – Settlements mapping

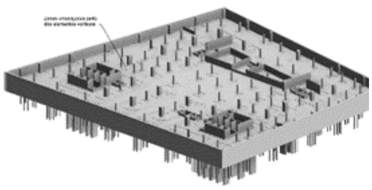


Figure 4 - BIM Revit 2022 model

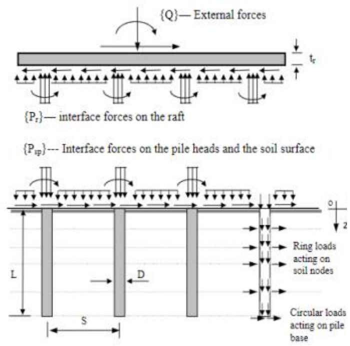


Figure 5 – CPRF conceptual model

- Monitoring

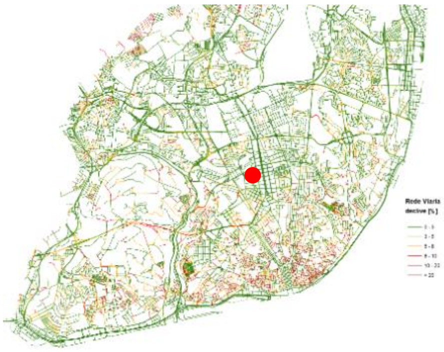
The implementation of an automated monitoring system allowed to collect a large amount of data and thus a permanent and continuous back analysis using the stress-strain models for each main excavation phase, at the more sensitive stages for every 5 minutes. Another key issue was the increase of readings during the periods of intense rainfall to study the variation of pore pressures and their consequent on the ground effective stresses. The presence of vibrating wire piezometers, with low time lag, fast response, allows also to associate a particular event with the change of the wall displacement (Figure 6).

Figure 6 - Automated monitoring installed in plot 1

2021

PHOENIX BUILDING. EARTH RETAINING STRUCTURES. LISBON

General characteristics



Location: Lisbon – Conclusion year: 2022 – Purpose: office building
Type: earth retaining structures
Maximum depth: 19.0 m - Area: 5 900 m^2 - Excavation Volume: 125 000m^3
Geological conditions: landfills, covering sands and sandstones
Owner: FIDELIDADE - Designer: JETsj Geotecnia - Contractor: DST Group
Site Supervision: Teixeira Trigo
Remarks: urban deep excavation highly restrained by the adjacent structures and infrastructures

Solutions descriptions

- Main objectives

The Phoenix Building will be the new head office of Fidelidade group, in Portugal, and is located at Avenida Álvaro Pais, in Lisbon. The building has 7 upper floors, 1 ground floor, 2 semi-underground floors and 3 underground floors. The ground floor is divided into 3 sub-levels, to connect the different peripheral streets. Due to local topography the excavation depth was bigger at the South side, led to the execution of 2 semi - underground floors.

Site location aerial view (left hand side) and building virtual perspective (right hand side)

- Geological and geotechnical conditions

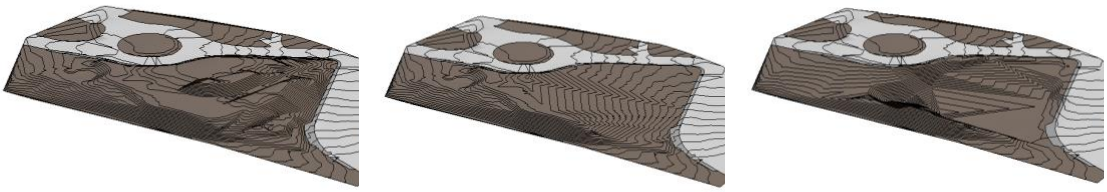
At the surface, landfill deposits, covering layers of sandy soils ("Areolas de Estefânia" and "Argilas de Prazeres") with increasing resistance in depth, over the sandstones bed rock ("Formação de Benfica"), with average low permeability.

- Other conditions

Design solution considered the surrounding infrastructures, in particular the underground ones, which were mostly sewage collectors and water pipes. At the southern boundary, the existing railway lines (IP), imposed deformation restrictions.

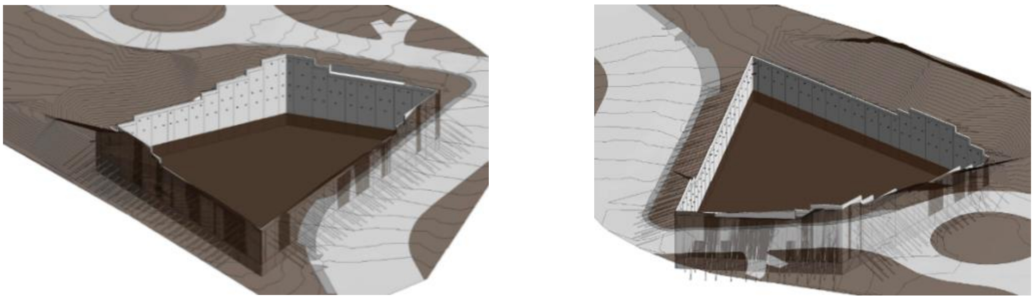
- Design and general description of the adopted solutions

In the first phase, preliminary earthworks were carried out with the purpose of remove the excess landfill above the peripheral street's levels. After these works, were defined temporary slopes, generally with 2(H):1(V) inclination, with greater expression at the West and South side of the lot, in which the Landscape project defined an exterior amphitheatre and the access ramp to the underground parking floors. These slopes aimed to minimize subsequent excavations and define a work platform for carrying out earth retaining wall works.



Revit model Southeast view: original topography (on the left-hand side), topography after preliminary earthworks (at the centre) and topography after temporary slopes (on the right-hand side)

Considering all the relevant conditions, mostly the geological, geotechnical and hydro-geologic, described above, the excavation was executed with a diaphragm retaining wall with 0.50m minimum thickness. To control and minimize the small water inflow into the excavation pit and also to ensure the wall bearing capacity, the diaphragm walls were built with a 3.0m minimum embedment length intersecting the sandstones.

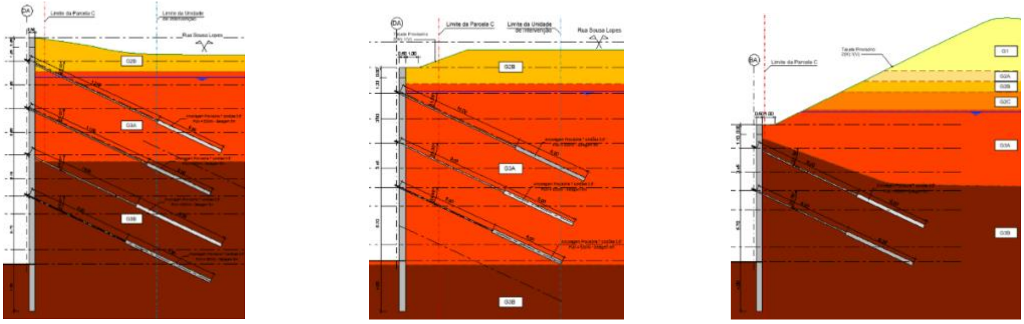


Revit model: Southeast view (left hand side) and Northwest view (right hand side)

The earth retaining wall has variable top levels to be compatible with the temporary slopes, as well as with the peripherical streets levels. The building superstructure, executed from bottom to top, will be the responsible for the retaining wall bracing.

Excavation preliminary works and execution of diaphragm walls

Earth retaining wall temporary horizontal bracing was ensured with ground anchors and corner steel struts, with 4 levels in the highest areas (19 m), 3 levels in the intermediate ones (15 m) and, finally, 2 levels in the lowest areas (11 m), as illustrated.



Earth retaining wall main cross sections

Excavation works and temporary ground anchors

- Monitoring

With the purpose of monitoring the earth retaining wall structure and peripheral infrastructures during the excavation works, was defined a monitoring system composed by tiltmeters, load cells, inclinometers, piezometers and topographical marks. Due to the high number of equipment, they were integrated in an automated reading system, allowing online readings of all devices and, consequently, a better earth retaining wall behaviour control along the excavation works.

References

Gondar, J.; Pinto, A.; Fartaria, C. The use of BIM Technology in Geotechnical Engineering. 17th European Conference on Soil Mechanics and Geotechnical Engineering, September 2019, Reykjavik, Iceland, Invited Plenary Paper. ISBN 978-9935-9436-1-3.

2021

MELIÃ HOTEL. EARTH RETAINING STRUCTURES. LISBON

General characteristics

Location: Lisbon – Conclusion Year: 2021
Purpose: hotel building
Type: earth retaining structures
Maximum depth: 22 m - Area: 1 428 m^2
Geological conditions: landfill, Lisbon Miocene, clays and limestones, and Lisbon Volcanic Complex
Owner: EXSPRING - Designer: JETsj Geotecnia
Contractor: Mota Engil – Site Supervision: Engexpor
Remarks: urban deep excavation at Lisbon centre restrained by the adjacent structures and infrastructures, mainly the sensitive Lisbon Metro tunnel and the Marques roadway tunnel.

Solutions descriptions

- Main objectives

The deep excavation works implemented had as main goal the excavation about 22m depth, for the execution of the 3 underground floors and 3 semi-underground floors for the new Meliã hotel located next to Marques de Pombal Square, in Lisbon. The adopted solution was strongly conditioned by the small distance to the Lisbon Metro tunnel, as well as to the Marquês roadway tunnel. A key issue was also the high elevation difference between the east and west adjacent streets, due to the 3 semi-underground floors.

- Geological and geotechnical conditions

At the surface, landfill deposits and alluvial soils, covering the Lisbon Miocene, mainly clays and limestones, and also the Lisbon Volcanic Complex, all with average low permeability.

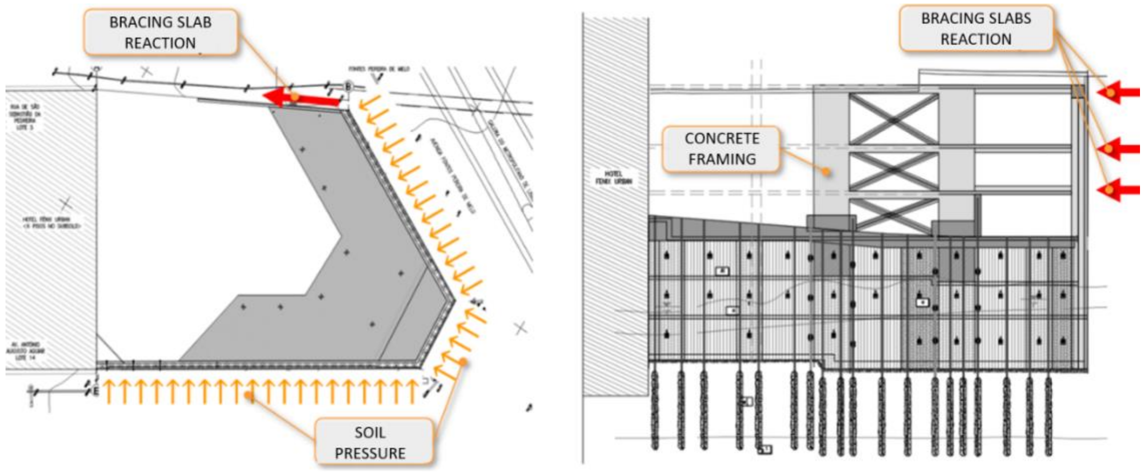
- Other restraints

The excavation site is located on a heavily urbanized area of the Lisbon city. In urban scenarios it was required to develop solutions compatible with the integrity preservation of the nearest structures and infrastructures. The excavation site presented the following main adjacent structures and infrastructures:

- North side: Fénix Urban Hotel with 11 floors above surface level and 6 underground floors.
- East side: São Sebastião da Pedreira Street with a gravity earth retaining wall with masonry structure.
- West side: António Augusto Aguiar Street with the Marquês roadway tunnel.
- South side: Fontes Pereira de Melo Street with the Lisbon Metro tunnel, with about 60 years old, built using the cut and cover method and with a plain concrete structure.

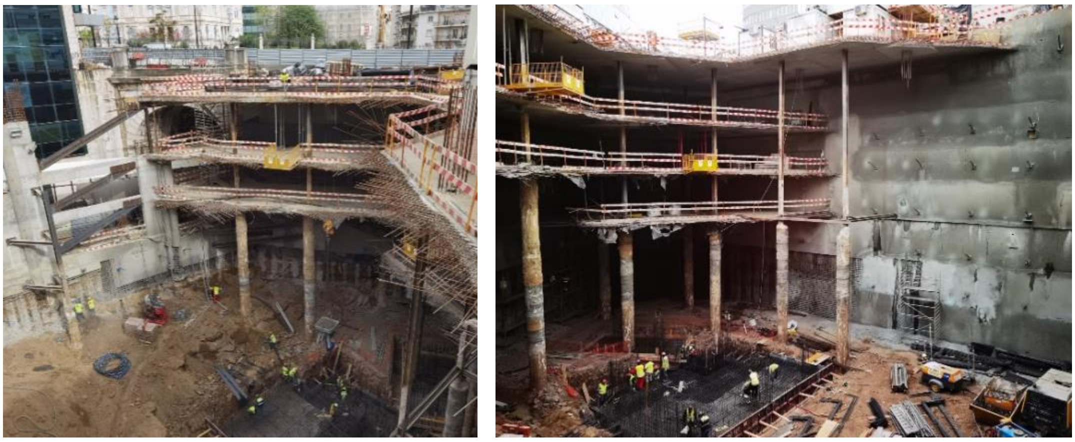
- General description of the adopted solutions

Considering the main local restrains, the earth retaining wall solutions implemented at the South and West sides were built with concrete bored piles of 800 mm diameter and spaced 1m, while at the East side, where excavation height was smaller, concrete bored piles of 500 mm diameter and spaced 0,75 m were built. On both solutions a lining of shotcrete with reinforcement steel mesh was executed over the space between piles to confine the ground between piles. Geodrain pipes were installed between piles to prevent the installation of a hydrostatic pressure. Locally due to the presence of the gravity earth retaining wall at São Sebastião Street, not allowing the execution of concrete bored piles, a reinforced concrete wall supported temporary by ground anchors was also executed for the excavation for the 3 underground floors (Berlin type wall). This excavation was preceded by the ground improvement with vertical cement slurry columns.



Level 0 bracing slab band plan (left hand-side) and reinforced concrete frame over Berlin type wall view (right hand-side)

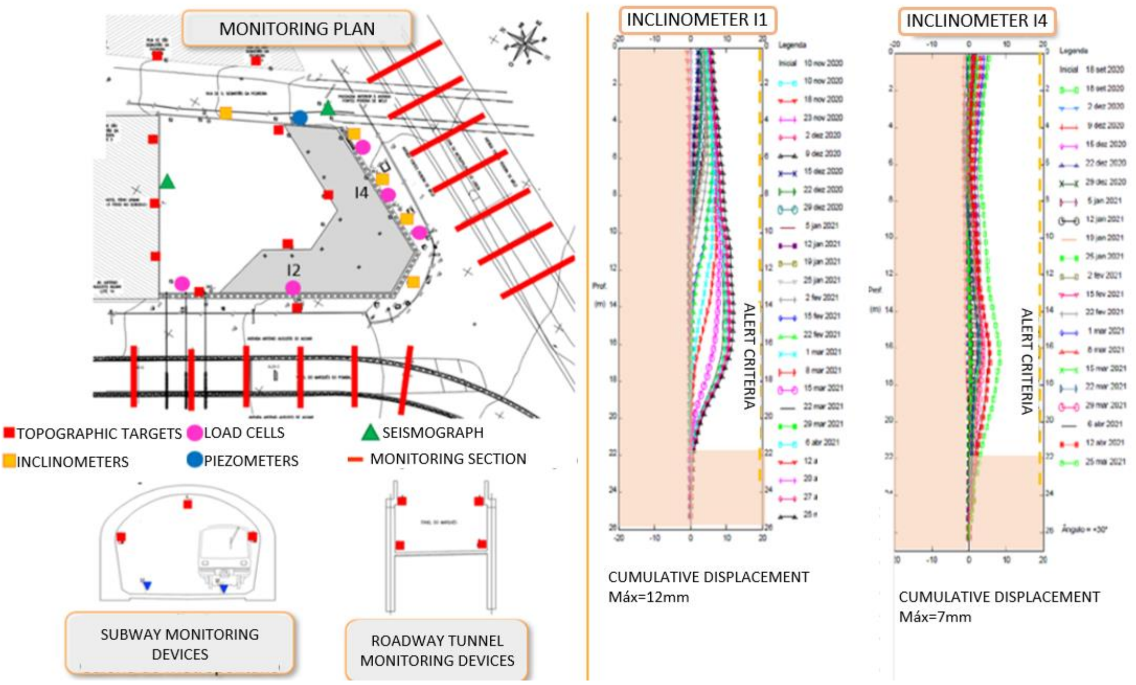
Regarding the horizontal equilibrium of the retaining walls, given the close location of the Lisbon Metro line and the roadway tunnel the use of temporary ground anchors was only possible at East side and locally at West side. Thus, the concrete bored piles retaining wall on South and West sides was mainly supported by concrete bracing slabs using the partial top-down excavation method which determined the excavation of each underground floor preceded by the execution of the bracing slab of the above floor. The bracing slabs were executed on levels 0, -1, -2 and -3 and were temporarily supported steel piles. Given the elevation difference between West and East side, there was a need to support the horizontal forces from the bracing slab bands above the -3 floor and therefore a reinforced concrete frame was executed aligned with the East retaining wall, built using the Berlin type method.



Views of the excavation works

- Monitoring

According to current practice in this type of projects in urban areas the implementation of a Monitoring Plan was the main key for the management of geotechnical risk. The adopted Monitoring Plan included the installation and measure of devices on the roadway tunnel, Lisbon Metro line, neighbour buildings and the retaining structures themselves, which made it possible to manage the geotechnical risk and confirm the suitability of the adopted solutions.



Monitoring Plan – Devices location (left) and some main measures (right)

References

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Pinto, A.; Fartaria, C.; Pita, X. e Tomásio, R. (2017). *FPM41 high rise building in central Lisbon: innovative solutions for a deep and complex excavation. 19th International Conference on Soil Mechanics and Geotechnical Engineering*. Seoul, Korea. pp 2029 – 2032, TC 207 (Soil Structure). ISBN 978-89-952197-5-1.

2017

FONTES PEREIRA DE MELO BUILDING, 41. EARTH RETAINING STRUCTURES

General characteristics

Location: Lisbon - Year: 2017 - Purpose: office building
Type: earth retaining structures
Maximum depth: 20 m - Area: 2 031 m^2
Geological conditions: landfills and Lisbon Miocene, clays, marls and limestones
Owner: Edifício 41 - Promoção Imobiliária e Hotelaria SA. - Designer: JETsj
Geotecnia Contractor: Mota Engil / Casais – Site Supervision: Rockbuilding
Remarks: deep excavation at the Lisbon centre highly restrained by the adjacent structures and infrastructures, mainly the Lisbon Metro tunnel

Solutions descriptions

- Main objectives

The adopted earth retaining structures were designed to allow the excavation of about 20 m depth, for the execution of 6 underground floors, as part of office building located next to Saldanha Square in Lisbon centre. The solution was strongly conditioned by the small distance to the sensitive Lisbon Metro tunnel line, as well as to centenary adjacent buildings.

- Geological conditions

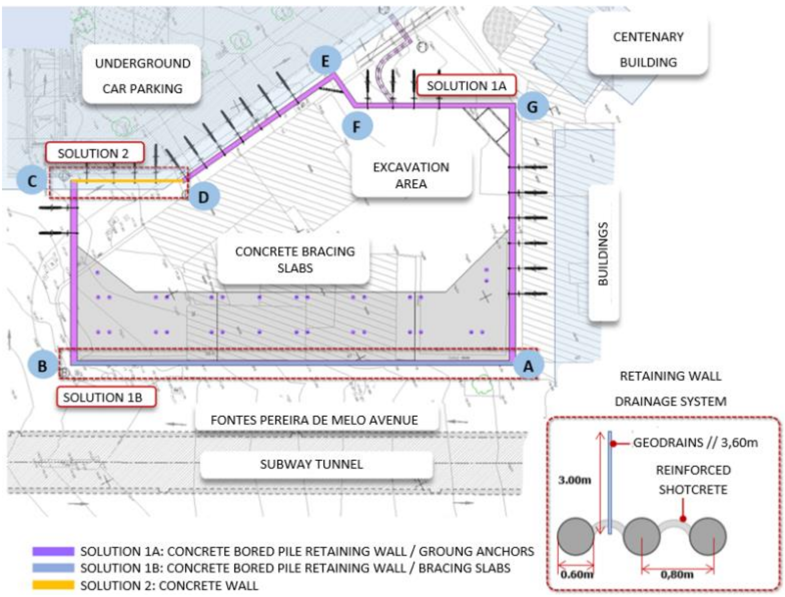
At the surface, heterogeneous landfills, covering the Lisbon Miocene, mainly clays, marls and limestones, with average low permeability.

- Other restraints

Should be pointed out the proximity to the Lisbon Metro tunnel under the Fontes Pereira de Melo Avenue, located just 9 m from the retaining wall. The tunnel with about 60 years old was built using the cut and cover method and with a plain concrete structure.

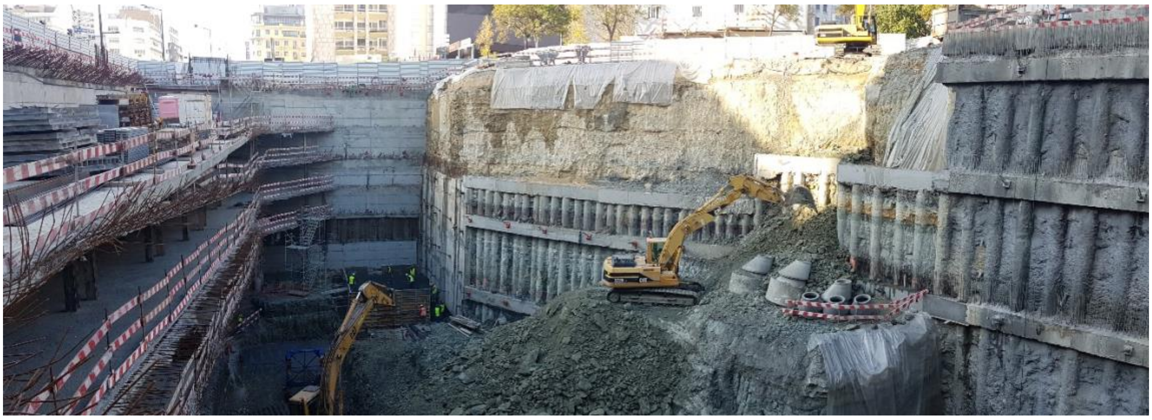
- Design and general description of the adopted solutions

The general solution included a reinforced concrete bored pile wall, with 600mm diameter piles, spaced 1,2 m. Between the piles the ground was confined by a shotcrete lining drained by geodrain pipes, preventing the installation of a hydrostatic pressures. Locally due to the presence of an underground car park adjacent to excavation border, a concrete reinforced wall was executed with no need for horizontal structural support. Regarding the overall solution of piles retaining wall the horizontal bracing system included temporary ground anchors where possible. Due to close location of the Lisbon Metro tunnel under the Fontes Pereira de Melo Avenue the use of ground anchors was not possible and therefore a bracing solution of reinforced concrete slab bands were adopted. The slab bands with spans over 55 m were implemented at levels -1, -2, -3 and -4 and below that level the excavation would already intersect the most competent Miocene materials that would impose reduced soil pressures on the retaining wall.



Earth retaining walls solutions

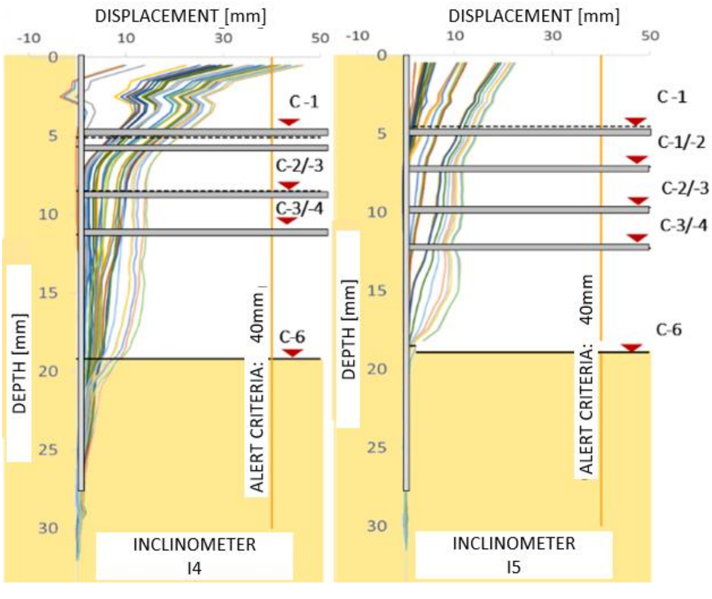
Regarding the execution of the excavation works there was an interruption of all works in winter of 2015/2016 for extra-technical reasons when the excavation level was close to -1 level without the same level bracing slab being executed. This situation determined the retaining wall with no lateral support for a long period of time with an excavation depth of about 4 m which led to an increase of horizontal deformations over time. Despite the unpredictability of the event that determined the interruption of the excavation works and its consequences, the retaining wall solutions implemented were suitable and allowed minimal interference with the Lisbon Tunnel infrastructure.



View of the final excavation works

- Monitoring

According to current practice in this type of projects in urban areas the implementation of a Monitoring Plan was the main key for the management of geotechnical risk. The measures of multiple devices took place since the beginning of the excavation and even with the interruption of the works allowed the continuous monitoring of the retaining wall and the Lisbon Tunnel which allowed the confirmation of displacement increase over time and the urgent need of reinforcement measures and exaction works restart. After the works restart some reinforcement measures were implemented like the execution of a reinforced concrete lining of the retaining bored pile wall and the execution of a horizontal bracing slab ate -1 level. An additional bracing slab between level -2 and level -3 floors was also built. At the final excavation works the total horizontal displacement of the retaining wall achieved over 40mm at the capping beam level.



Inclinometers located at Fontes Pereira de Melo Avenue

References

Pinto, A., Pereira, A., Villar, M. (2007) Deep Excavation for the new Central Library of Lisbon, *Proceedings of the 14th European Conference on Soil Mechanics and Geotechnical Engineering*, Madrid, Spain, pp. 623 – 628.

Pinto, A., Pita, X. (2011) Deep Excavations in Luanda City Centre, *Proceedings of the 15th African Regional Conference on Soil Mechanics and Geotechnical Engineering*, Maputo, Mozambique, pp. 269 – 274.

2010

MOSCAVIDE AND AIRPORT LISBON METRO STATIONS. EARTH RETAINING STRUCTURES

General characteristics

Location: Loures and Lisbon – Conclusion year: 2010
Purpose: connection between Oriente and Lisbon Airport Station, Lisbon Metro Red Line
Type: earth retaining structures
Maximum depth: 2 5m
Geological conditions: Lisbon Miocene, mainly: marls, sandstones and limestones
Owner: Metropolitano de Lisboa – Designer: JETsj Geotecnia
Contractor: Aerometro ACE – Site Surpervision: Ferconsult
Remarks: urban deep excavation highly restrained by the adjacent structures and infrastructures

Solutions descriptions

- Main objectives

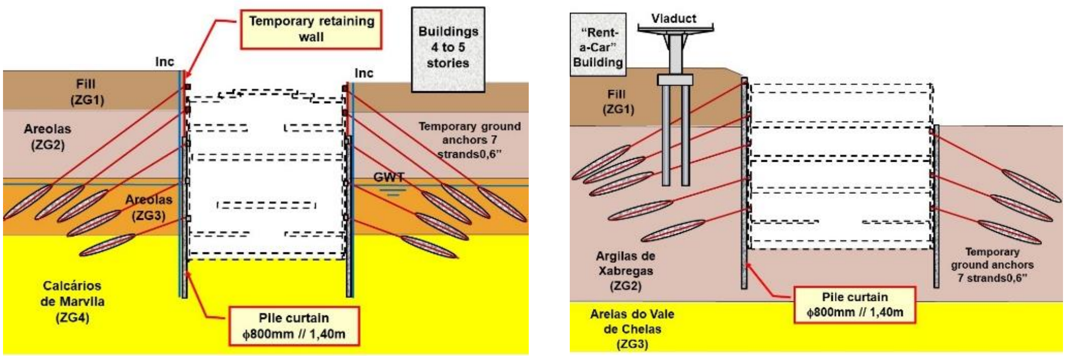
As part of the expansion of the Red Line of the Lisbon Metro network, between Oriente Station and Airport, the Moscavide and Airport stations were built, using the cut and cover method. Considering local conditions, both stations were built using reinforced concrete bored pile walls, braced by temporary ground anchors and corner struts during the excavation works.

View of the excavation sites (Moscavide and Airport)

- Geological and geotechnical conditions

Moscavide Station: the excavation works intersected the sedimentary materials dating from the Lisbon Miocene, "Areolas de Braço de Prata", resting over the "Calcários de Marvila". The areolas were predominantly composed of hard silty-clay sediments, sometimes with abundant fossil shells and small amounts of fine sand. The "Calcários de Marvila" were composed of interbedded layers of sandstone, marls, limestones and clays. At the base of the "Calcários de Marvila" the formation of "Arenitos de Grilo" was observed. At the surface, landfill deposits and alluvium materials were intersected, with a maximum thickness of 2.5 m.

Airport Station: the excavation works also involved the sedimentary materials dating from the Miocene, consisting of the unit called "Argilas de Xabregas", composed of dark brown silty clays, with small limestone fragments, as well as the unit called "Areias do Vale de Chelas", composed of fossiliferous limestone and brownish marly limestone. At the surface landfill deposits were intersected, with a maximum thickness of up to 6m.



Geotechnical profiles (Moscavide at left hand side and Airport at right hand side)

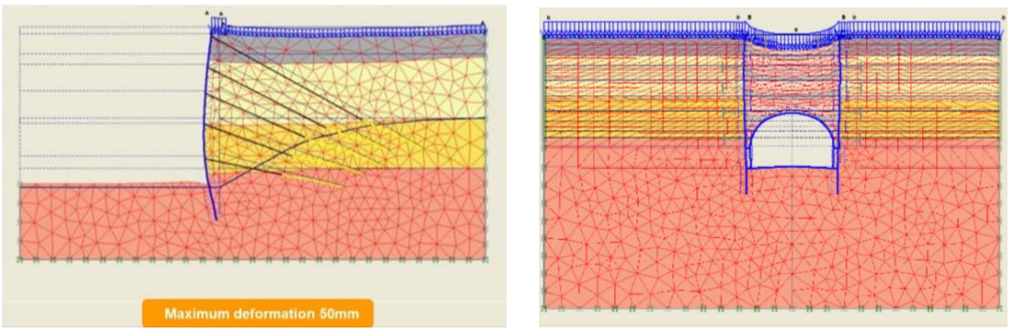
- Design and general description of the adopted solutions for earth retaining walls

Retaining walls solutions using reinforced concrete bored piles, with 800mm diameter and distance between axes of 1.4 m, were adopted. The piles were integrated as part of the internal structure retaining wall and were capped by capping beams. To resist to the earth pressures the piles wall was temporary braced by corner struts and ground anchors, at several levels. All ground anchors were submitted to reception or suitability tests, in the latter case when monitored with load cells. After the construction of the internal structure, the piles wall was horizontally braced by the slabs of the underground floors. To increase the redistribution capacity, the piles were connected by the capping beam and the distribution beams, both in reinforced concrete. The distribution beams were also incorporated into the internal structure. To avoid erosion of the ground between the piles, a shotcrete lining was applied. During the construction of the internal structure, an interior wall was cast against the piles, to accommodate the hydrostatic pressures and increase the water tightening of each Station.

- Design and general description of the adopted solutions for West end of Moscavide Station and both ends of Airport Station
At the western end of Moscavide Station and at both ends of the Airport Station, to ensure the compatibility with the solutions defined for tunnels (executed by NATM), cut and cover tunnels were built with about 150 m^2 of cross section. The space between the tunnel crest and the surface level was later backfilled by layers of selected soil, interbedded with layers of lean concrete, at the level of the temporary ground anchors, preventing the mobilization of undesirable deformations of the pile walls during the deactivation of the temporary bracing elements.

Execution of cut and cover galleries

- Main design issues
The behaviour of the different peripheral bored piles walls, in terms of efforts and deformations, was analysed, for all construction phases and considering the soil/structure interaction phenomena, through finite element programs designed for this purpose, Plaxis Professional V9.02.



Examples of used calculation models

- Monitoring
Based on the works framework, mainly the complexity of the identified geotechnical scenario, the Monitoring Plan was implemented before the start of the excavations works, as a fundamental part for the online the validation of all the design assumptions. About 100 topographical targets, 20 settlement marks, 18 load cells and 15 inclinometers were installed in the excavation of the Moscavide station and about 50 topographical targets, 3 settlement marks, 18 load cells and 4 inclinometers were installed at the Airport Station.

References

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Pinto, A.; Pereira, A.& Vilar, M. (2007). Deep Excavation for the new Central Library of Lisbon. 14th European Conference on Soil Mechanics and Geotechnical Engineering, Madrid, Spain, pp. 623 – 628, Volume 2 - 2.1: Effect of open excavations on nearby structures and facilities in urban areas.

2009

INTERCEPTOR OF RIVER LIMA. ATTACK WELLS FOR MICROTUNNELING. PONTE DE LIMA

General characteristics of the site

Location: Ponte de Lima – Year of conclusion: 2009 - Purpose: Wells for microtunneling
Client: Águas do Minho e Lima - Contractor: Geo-Rumo
Project: JETsj
Maximum depth – 17 and 18 m - Perimeter: 42,5 and 47 m: CSM retaining wall.

Description of the work

- Main goals

Construction of two attack wells and respective interior hydraulic structures, for the installation of a tunnel pipeline under the bed of the Lima River, in Ponte Lima, integrated in the São Jorge Water Subsystem. The main objectives of the solutions studied and implemented were aimed at making the excavation feasible, under safe conditions, to connect the pipeline to the hydraulic structures to be installed inside the wells, with the need to contain the excavation ground and minimize the water that would flow into the excavation area.

- Geological and geotechnical conditions

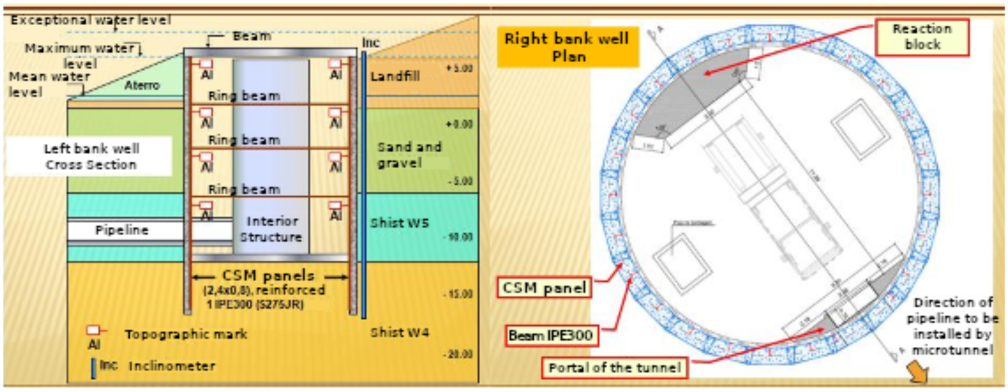
The geological and geotechnical conditions were assessed before work began by a geological and geotechnical survey campaign. According to this, this scenario was composed, from the surface, by very heterogeneous embankments, sand and gravel, resting on schists, in general, with a marked degree of alteration, but decreasing in depth.

- Other conditions

The wells were built along the banks of the River Lima, so it was considered inadvisable to build embankment platforms that would invade the riverbed excessively. Equally important was the use of techniques that limited the contamination of the riverbed with the materials included in the construction. Wells were made in a cylindrical geometry, which should be compatible with the microtunneling works, in particular with the altimetric and planimetric dimensions of the pipeline to be installed under the River Lima with a diameter of 700mm. The geometry and the resistant capacity of the wells, particularly the one located upstream, should also be compatible with the execution of reaction massifs where the hydraulic jacks that will push the various segments of the pipeline will be supported. Inside both wells it should also be possible to execute the internal structure, in reinforced concrete, which would accommodate the hydraulic components to be installed inside the wells. The project had an execution period of approximately 8 months, compatible with the execution of the other works included in the São Jorge Water Subsystem, so the solutions proposed and implemented should enable this important objective to be achieved, as any delays would result in high costs due to loss of exploration.

- Description of the solution adopted

Based on the geological scenario of the site, as well as the amplitude and type of loads to be accommodated by the walls of the wells, it was proposed and implemented the realization of a solution of temporary retaining wall by soil-cement panels with a thickness of 0.80 m, executed through the CSM technique, reinforced with metal profiles IPE300, placed vertically. These elements accommodate all the forces and were headed by a reinforced concrete beam and braced, in depth, by three metallic beams in a ring geometry, connected to the vertical profiles.

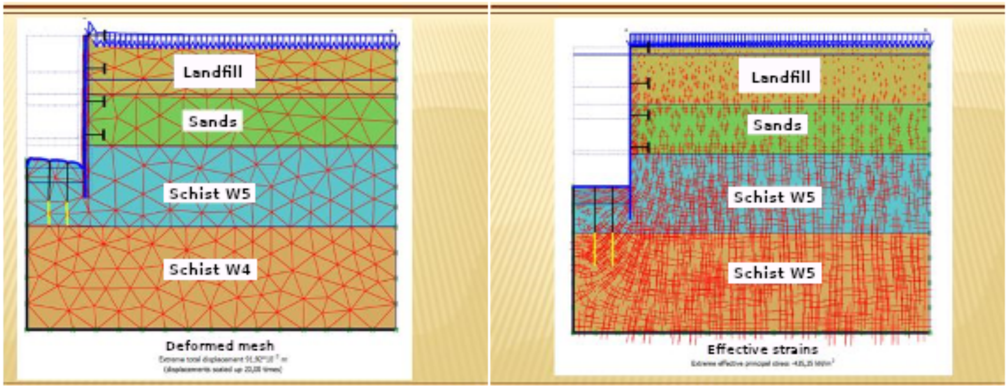


Cross section and plan of solutions designed and constructed

The minimum fixing of the panels below the bottom of the excavation was 5.0m, in order to limit the influx of water through the bottom of the excavation. In the entrance and exit areas of the microtunneling machine, the distance between the metal profiles was adjusted to facilitate drilling. The profiles were driven into the panels as soon as they were finished, using a vibrator. The retaining wall was secured at various levels, on the surface by a reinforced concrete capping beam and, in depth and internally, at three levels, by ring metallic beams made of HEB200 sections.

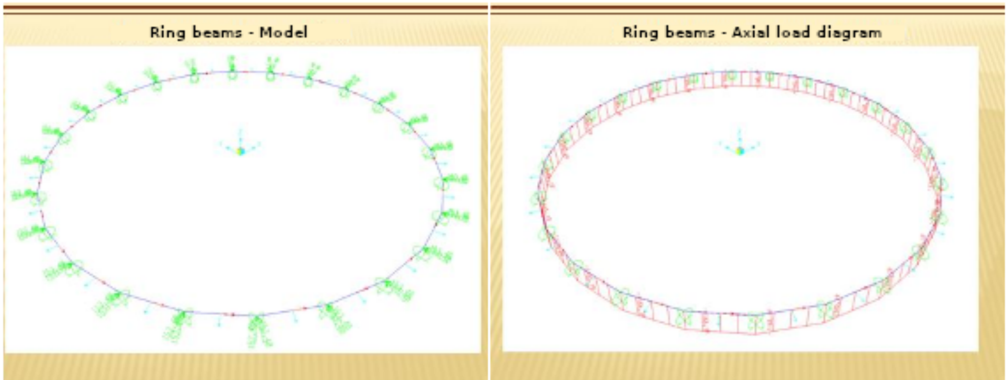
- Key aspects of the project

The behaviour of all the elements of the well retaining structures, in terms of stresses and strains, was analysed, for all the construction phases, through finite element software designed for this purpose and properly interconnected with each other. In order to evaluate the strains and stresses in the vertical elements of the proposed retaining structure, as well as the estimated water flow into the excavation, the geometry was modelled using a non-linear finite element software, especially designed for this type of application and respecting the Mohr-Coulomb failure criteria. Given the circular geometry of the wells, a axisymmetric model was chosen, in order to properly consider the effect of circumferential compressions on the well retaining structures.



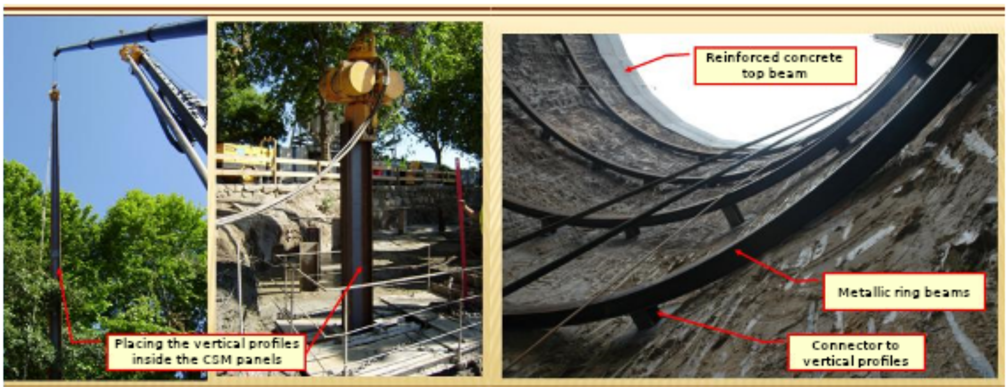
Model of the retaining wall and main results

In order to evaluate the possible development of bending and/or shear forces, at the level of the ring beams, due to the transmission of the circumferential load not being performed continuously along the entire length of the beams, but in a discrete way, at the various points of connection between the said beams and the vertical profiles existing inside the CSM panels, a complementary calculation model was carried out in which the loads were applied in a discrete way.

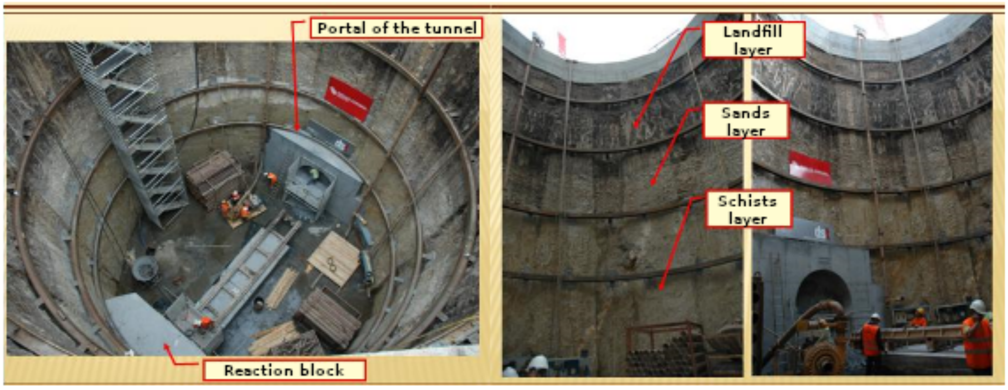


Model of the ring beams and main results

- Key aspects of the work



View of the driving of vertical beams inside the panels and the supporting system of the well

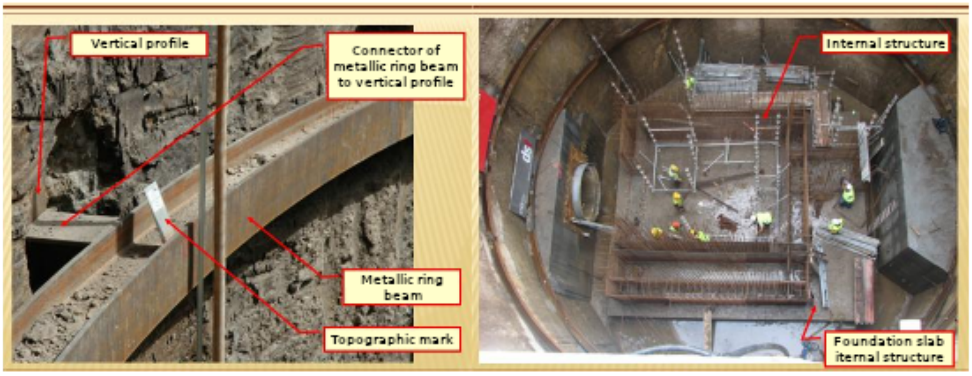


View of the interior of the right bank well after excavation

Still within the scope of the CSM technique, the equipment's capacity for strict execution control deserves highlighting, as it allows the monitoring and correction, in real time, of parameters such as the advance of the cut-off wheels, the injection of the mortar and the verticality of the panel, in the event of any deviation during execution. This monitoring is possible at any stage of the cut-off wheel progress, as the operator of the equipment has a complete instrument panel, which allows automatic control of all the drilling, fluid injection and mixing parameters.

As a way of controlling the quality of execution, in particular the confirmation of the resistance and deformability parameters of the soil-cement panels, samples of the panels were taken immediately after their execution and drilling cores during the interior excavation of the wells, which were used to perform tests that confirmed, in general, the achievement of the parameters defined in the design phase.

The foundations of the hydraulic bodies built inside the wells were materialised by a reinforced concrete foundation slab, with a thickness of 0.35m, connected at the edges to the peripheral wall of reinforced soil-cement panels, and which also worked as a foundation element for the microtunneling support systems. The eventual need to nail this slab to the base was not necessary, due to the reduced inflow of water into the excavation, controlled by conventional pumping devices.



Connection of ring beams to the vertical profiles and view of the construction of the internal structure

Based on the construction context, as well as the innovative nature of the type of retaining solutions adopted, it was considered very important to implement an adequate Instrumentation and Observational Plan, in order to ensure that the works were carried out in safe conditions for the construction and for the neighbouring structures and infrastructures, as well as to allow the confirmation of the assumptions made in the Project phase. In the described context, 12 topographic marks were installed in each of the wells, on the topping and bracing beams and 3 inclinometers on the back of the temporary retaining in CSM panels. The installed devices were read on a weekly basis during the excavation and construction works of the internal structure. From the results of the calculations performed for the design of the solutions, warning and alarm thresholds were defined, as well as reinforcement measures, in case these thresholds were reached. However, according to the results of the campaigns carried out, none of the mentioned thresholds were reached, proving the adequacy of the proposed and implemented solutions. The maximum value of displacements recorded, in the horizontal direction, did not exceed 10 mm, having occurred during the final phase of excavation at the bottom of the excavation and was practically coincident with that estimated in the design phase, through the modelling performed in axisymmetric mode. The instrumentation and observation of the behaviour of the construction were complemented by the collection of cores to verify the homogeneity and strength of the soil-cement panels, the latter measured through unconfined uniaxial compression tests with strain gauging, in order to determine the value of the compressive strength and the modulus of deformability. The results of the tests confirmed that the strength and deformability values stipulated in the project were obtained. After the drilling of the cores, all the holes were properly sealed.

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2007

TERREIRO DO PAÇO METRO STATION. LISBOA

General characteristics

Municipality: Lisboa – Year of completion: 2007 - Purpose: Metro Station
Type: reinforced concrete and bentonite-cement secant piles and prestressed struts in the peripheral curtain
Maximum height: 26 m – Lenght: 144 m – Width: 24 e 29 m
- Excavation volume: 100 000 m^3
Ground: sandy and soft clay alluvium underlying compact marls
Owner: Metropolitano de Lisboa - Designer: Cenor (today TPF)-Ferconsult
- Technical assistance: Cenor-Ferconsult - Contractor: Teixeira Duarte, Mota-Engil and OPCA consortium.

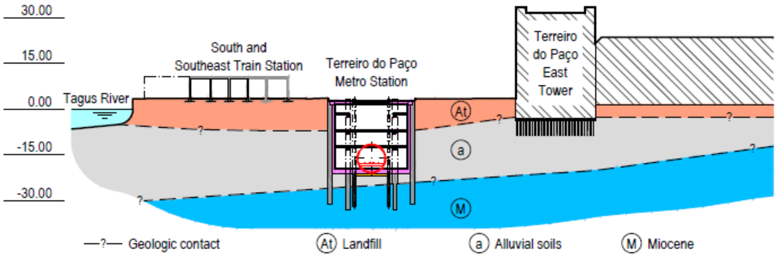
Description of the works

- Main objectives

Terreiro do Paço Metro Station is part of the Blue Line of the Lisbon Metro, located in the zone that connects Baixa-Chiado Metro Station to Santa Apolónia Metro Station. Is located between the east building of Terreiro do Paço Square, occupied by the Ministry of Finance, and the south and southeast maritime station, a ground floor building already built in the 20th Century. A large part of the station's implantation area was reclaimed from the river with fills, placed before and after the 1755 earthquake.

- Geological and geotechnical conditions

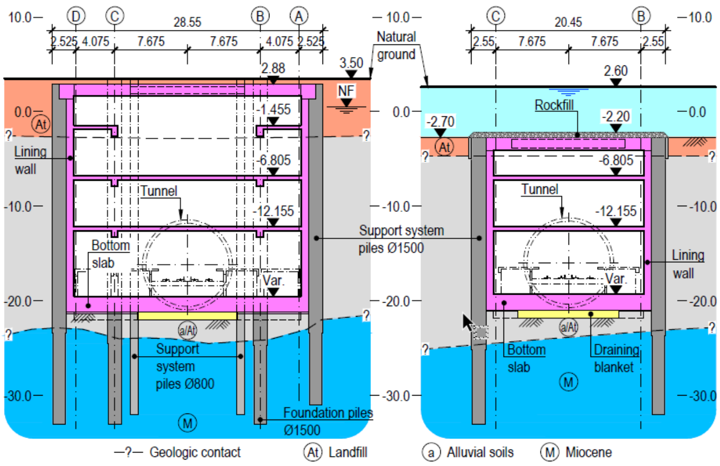
The substratum is composed of Miocene formations, covered by soft alluvial deposits and embankments. Overall, the surface of the substrate descends progressively towards the river with a slight inclination towards S-SW. Landfill soils with variable thickness occur in depth, sometimes mixed with alluvial deposits, containing stones and obstacles, sometimes of large dimensions. This is followed by the predominantly clay-muddy alluvial deposits, ranging from soft clays to sands (some very clean). However, there is a very significant predominance of clean sands at the base of the alluvial deposits. Underlying the alluvial deposits are the Miocene formations, consisting of clays from Forno do Tijolo, with a hard consistency, interspersed with layers of dense sands with artesianism.



Schematic section across the Tagus River according to the station and the east tower

- General description of the solution

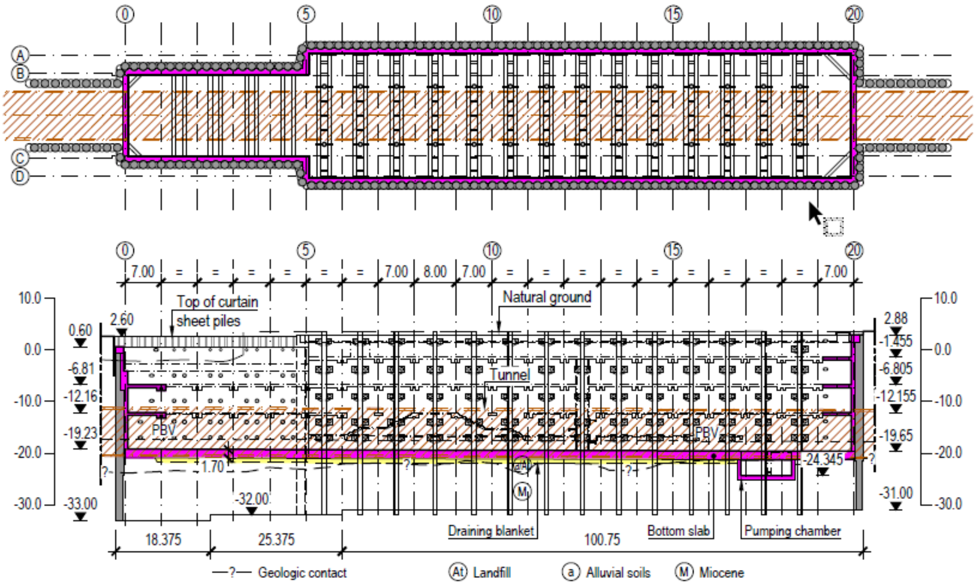
In simplified terms, the structure of the station corresponds to a large reinforced concrete box, built from the surface, connected to the tunnel in the two portals, at a distance of about 140 m and with a width of 16 m in the narrow part and 24 m in the wide part. The top of the substratum occurs slightly below the base of the station bottom slab.



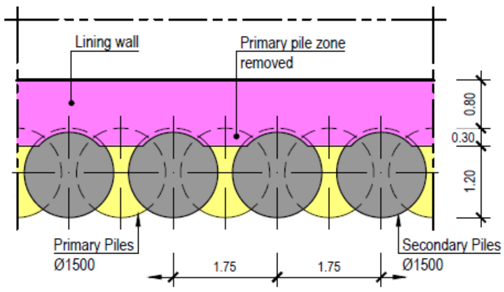
Cross-sections of the wide and narrow areas of the station

- Main design aspects

Reinforced concrete and bentonite-cement secant piles with 1.50 m diameter was chosen for the peripheral curtain. Bentonite-cement piles of 1.75 m spacing were previously executed. The reinforced concrete piles were then built, alternately with the first ones and partially sectioning them, also with a 1.75 m spacing. All piles penetrated at least 8 m into the Miocene substrate.

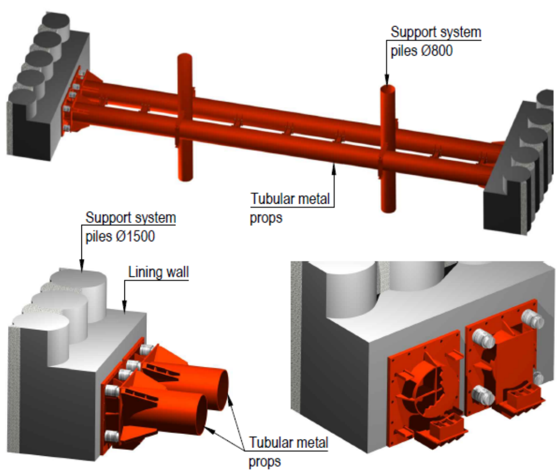


Plan and northern elevation of the peripheral curtain of secant piles and the shoring system



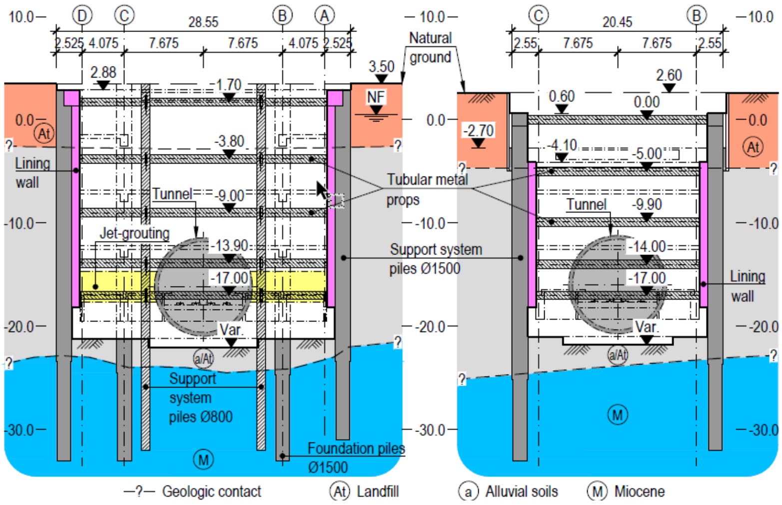
Plan of the theoretical position of the piles and the lining

For the temporary shoring system five levels of horizontal steel strut pairs were used, between the longitudinal faces (north and south) of the curtain, consisting of tubular profiles of large diameter (ϕ711 mm) and thickness from 16 to 25 mm with an average horizontal spacing of 3.50 m. The struts were strongly prestressed during the installation, with a uniform prestress load of 3 500 kN per strut, introduced by 4 hydraulic jacks. In the east (wide) area of the station, the struts were provided with bracing elements in the vertical plane at two points supported on metal piles ϕ800 filled with concrete, embedded in the subsoil and installed prior to excavation.



Struts system of the station wide area and the device for the application of prestress

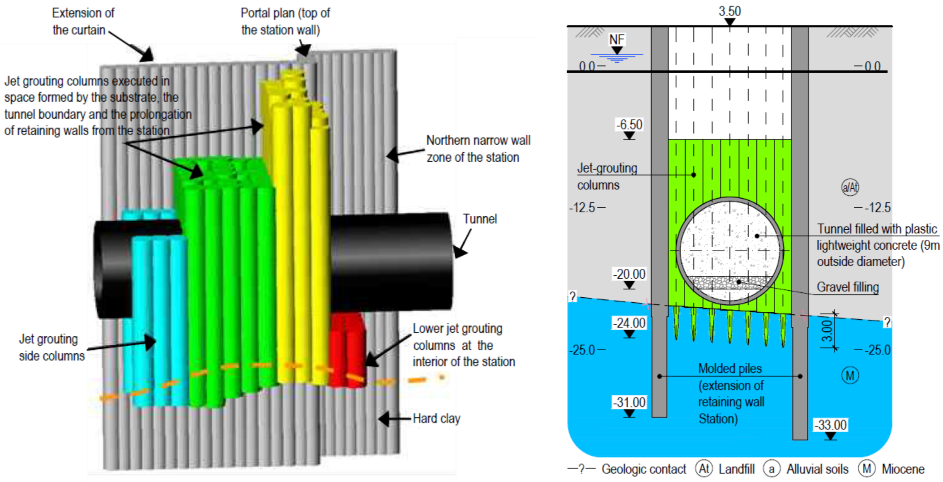
In the wide area of the station this temporary shoring system has been accompanied by a 3 m thick jet grouting slab, placed between the tunnel and the longitudinal curtains, with its median plane coinciding with the tunnel "equator". This slab, combined with the tunnel itself and the corresponding filling material, provided a particularly suitable support to the curtain at a depth of about 18-21 m. This has significantly reduced the displacement of the curtain compared to a solution method by using only conventional struts.



Cross sections of the wide and narrow areas of the station after completion of the last excavation phase


View of the curtain and shoring system from the Terreiro do Paço east tower

On the two extreme transversal walls of the station adjacent to the portals, the piles could not pass through the tunnel. It was then necessary to complement the pile curtain on these sites with a system which would allow the excavation to be carried out safely, as well as the construction of the final internal structure of the station, adequately connected to the tunnel lining. The extreme importance of the performance of such a system can be evaluated taking into account that the soft alluvial soils in contact with the clay substrate, located near the base of the tunnel, 25 m below the groundwater level, were at certain points made of clean sand.
The solution consisted of surrounding the tunnel sections adjacent to the station, with a mass of soil treated with jet grouting which would have to fulfil two essential conditions: be practically impermeable and be resistant to water and soil pressures on its outer face. In order to meet these conditions, it was essential, on one hand, to ensure good penetration of the jet grouting mass into the Miocene substrate and, on the other hand, to make a good connection between itself, the tunnel lining and the station structure. To realize this last requirement, the peripheral pile wall of the station was extended about 16 m beyond the plane of the portals, in order to confine the tunnel and the jet grouting.



3D view of the jet grouting treatment on the west portal and cross section of the west portal plane, showing the area treated with jet grouting

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2004

ALCÂNTARA STORM WATER DRAINAGE TUNNEL. REPAIR OF A SECTION OF THE STRUCTURE. RETAINING STRUCTURES. LISBON

Project's main features

Location: Lisbon - Conclusion year: 2004
Purpose: Reconstruction/Repair of an underground sewer tunnel following its collapse
Type: Anchored diaphragm wall with underpinning system
Maximum height: 14 m - Perimeter: 2 x 60 m - Surface: 1.800 m^2
Excavation volume: 21.000 m^3
Soil: landfill and alluvial deposits over limestone formations
Client: City Council of Lisbon - Design: Teixeira Duarte E.C
Technical Assistance: Teixeira Duarte E.C - Contractor: Teixeira Duarte E.C

Main project's features description

- Main goals

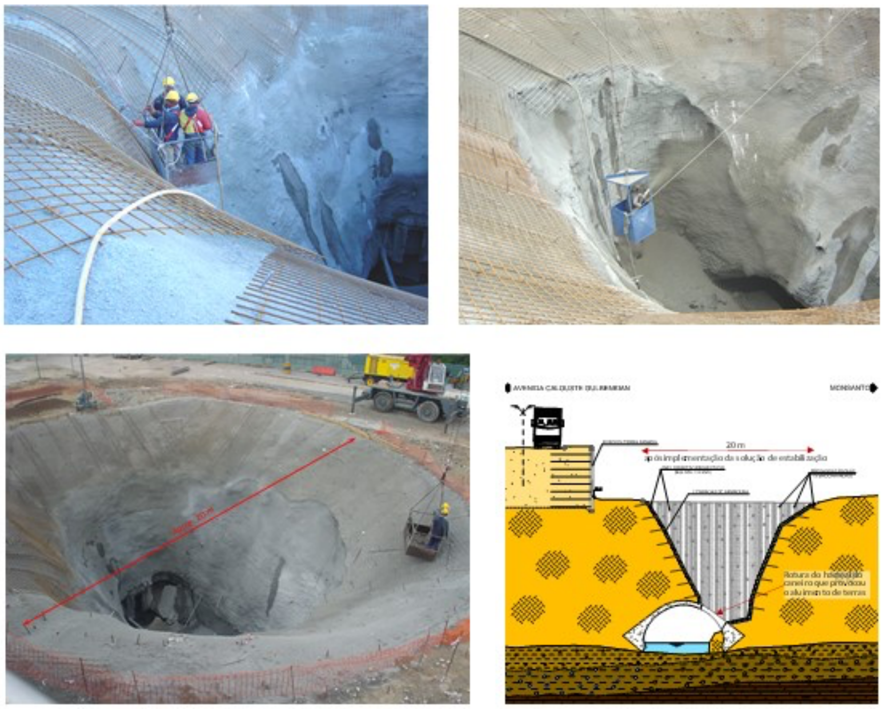
Intervention carried out, with urgency, for the City Council of Lisbon, to repair a section of the "Alcântara sewer tunnel", after the collapse of its structure which occurred in November 2003. This collapse occurred in a location where the section of this underground channel is at a depth of about 14m, which led to the formation of a crater that reached the surface, opening a large hole in the road pavement and sidewalk, near the "Campolide" train station.

Post collapse images illustrating its consequences on the road pavement causing a bus to fall into the formed crater

In addition to the initial damages, the continuous fall of the instable bordering soil into the crater and its consequent transport by the effluent stream further intensified the extreme complexity of the intervention, providing no safe conditions for the construction site. This events favoured a progressive collapse phenomena, fostering the growth of the surface crater's diameter (which initially measured approximately 14m), and providing no safety conditions for the execution of the stabilization measures of the affected area and repair operations of the tunnel itself.

- Preliminary stabilisation measures

At first, the intervention aimed to ensure the stability of the crater's surface preventing its volume growth.
In order to avoid placing equipment near the surroundings of the affected areas, the stabilization operations were carried out using suspension structures, operated remotely, which allowed the access of personnel and equipment to install geotechnical nailing bars, assemble steel reinforcing elements and apply a 25cm layer of shotcrete. These operations allowed the execution of a diametral temporary shaft retaining structure, which was essential to ensure the necessary safety conditions to allow the development of the following procedures in order to execute a complementary retaining wall compatible with the area and with the 14m excavation needed to carry out the tunnel's repair works.



Images and schematics of the solution for the provisional stabilisation of the crater created by the rupture of the tunnel

- Retaining wall structure

Given the poor geomechanical properties of the 14m landfill layer, aggravated by the consequences of the accident, and the imminent possibility of new ruptures of the tunnel structure, the repair intervention required the use of a retaining wall structure that would allow a 14m depth excavation, using a technology that would not involve the approaching of heavy equipment near the unstable areas. These constraints determined the execution of an anchored diaphragm wall solution, using a cable *benne*, operated remotely. Nevertheless, the existence of rockfill layer, not detected in the geotechnical survey campaigns, functioning as a drainage element, caused the loss of the stabilising fluids, imposed by the execution procedures of this technology, putting at risk the stability of the excavation trenches during the diaphragm wall execution. This problem forced the limitation of the height of the diaphragm panels to a 12m depth, in order to avoid intersecting this layer, which led to the adoption of underpinning systems to make it possible to reach the necessary depth to carry out the repair of the tunnel structure. The underpinning was accomplished by using steel HEB400 profiles, driven from a platform located about 1.5m above the level of the diaphragm wall base, which were connected to the retaining wall so they would function as vertical stability elements, capable to absorb the loadings resultant from the vertical component of the anchors and the self-weight of the diaphragm wall, and as horizontal stabilisation elements, through the mobilisation of the soil's passive earth pressure in their embedment.

Imagens and schematics of the implemented solution for the underpinning of the diaphragm wall

- Methodology used to repair – reconstruct the tunnel structure

The described underpinning solutions, performed according with a detailed constructive phasing, based on the execution of primary and secondary reinforced concrete underpinning panels, made it possible to reach the required excavation depth, allowing the subsequent installation of a piping by-pass system which permitted the drying of the areas along the extent of damage observed in the tunnel structure.

Images and schemes of the flow by-pass solution executed along the extent of the damages detected in the tunnel structure

The previous procedures allowed the execution of the repair/reconstruction operations of the strengthened reinforced concrete tunnel structure, in dry conditions, resetting its safety conditions and permitting the execution of a well graded and compacted landfill, followed by the reconstruction of the road pavement and sidewalk.

Images showing the main sequential phasing used for the reconstruction of the tunnel's structure

2004

SOTTO MAYOR PALACE. TEMPORARY SHORING SYSTEM AND EXCAVATION. FOUNDATIONS. LISBON

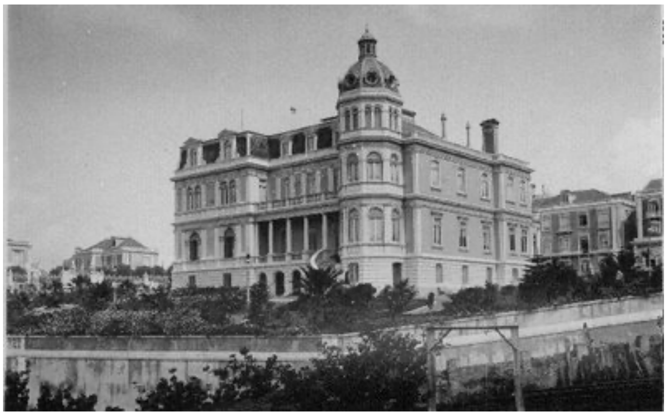
General site specifications

Local: Lisbon, Portugal - Conclusion date: 2004
Propose: reassessment of the Palace to build an underground gallery
Type: Retaining wall and palace reassessment
Terrain: clayey sands
Designer: Tecnasol - Contractor: Tecnasol

Description of the site characteristics

- Main goals

The project, Palace Sotto Mayor Business Center, consisted of the design and construction of the retaining walls and underlayment of the Palace with the aim of building an underground gallery under the aforementioned Palace and an excavation for the construction of 8 basements surrounding the gallery. The main objective of the project was to keep the building unaltered, in the center of the excavation, ensuring that there were no restrictions on the preservation of the Palace, maintaining the historical heritage or the surrounding services.



1915 photo of Sotto Mayor Palace

- Geological and geotechnical conditions

The lithology of the site concerned by the excavation is made up of a sandy-clay surface layer, based on the Miocene "Argils and Limestones of Prazeres". Its consistency is medium to hard with average N_{SPT} values between 30 and 40 strokes. At the base of the excavation was found the Benfica Formation, Oligocene.

- Other conditionings

The most important constraints were of a patrimonial nature (architectural and structural) as it was a building built between 1902 and 1906. In the 1990s the building was the target of a fire that damaged the roof and caused some structural damage. The excavation was confined to the west by Av. Fontes Pereira de Melo, where, about 10 m from the containment, there is the Lisbon subway tunnel.

- Design and general description of the solutions adopted

Shoring system

Three different constructive techniques were used in the retaining wall: Av. Pereira de Melo fountains, diaphragm wall 1.0 m thick and maximum depth of 27 m; Rua Martens Ferrão, diaphragm wall 0.6 m thick and maximum depth of 27 m, Rua Sousa Martins: Ø800 mm piles spaced curtain, 1.0 m apart coated with shotcrete, reinforced with mesh, maximum depth of 23 m Largo das Palmeiras and Andaluz: "Berlin" type wall, 350 mm thick, supported by micro piles and a maximum height of 18 m. All these retaining structures were provisionally locked with 1000 and 600 kN prestressing anchors, sealed in the ground using the RSI injection system (repetitive and selective injection). The anchors were built with lengths varying between 42 and 15 m. Simple or detailed reception tests were performed on all anchorages.

Interior renovation of the palace

To carry out this reassessment and considering the lack of interior space in the building, a solution was chosen using micro piles N80, Ø127 mm and 9 mm thick with 12 m in total length and 6 m of sealing, with a service load of 600 kN These micro piles were topped with prestressed beams that were joined to the existing walls with Ø32 mm Gewi bars. Regarding the applied prestress, 50% was installed prior to the external excavation works.

Palace retaining wall

Considering the spatial constraints, a Ø800 mm piles curtain spaced 1 m, coated with sprayed concrete over sun mesh, was used. Its locking used horizontal strapping beams in pre-stressed reinforced concrete due to the incompatibility of anchoring with the excavation and micro piles inside the palace. Six levels of beams were executed along a maximum height of 24 m.

Most relevant aspects of the project

The valorization of the historical and architectural heritage was achieved through a mostly geotechnical work that involved the excavation of about 150,000 m^3, in a block of 95 m X 85 m, with a depth of 27 m where the integrity of the palace was preserved in the area of excavation intervention.

View of the Palace's support and shoring system in the final stage of excavation

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Available @: http://www2.dec.fct.unl.pt/UNIC_ENG/palestras/Ciclo_Palestras_2003/SottoMayor/SOTTOMAYOR.pdf

2018

PIT AND DISCHARGE AND TRANSPORT TRENCH OF BAUXITE. KUMSAR. CONAKRI GUINEA

General site specifications

Location: Kamsar, Republic of Guinea Conakri – Concluding date: 2018
Purpose: Construction and design of a retainment structure in diaphragm wall of a pit and a trench for the discharge and transport of bauxite.
Type: Peripheral retaining wall using diaphragm wall technic
Terrain: clayey-sandy surface layer of recent age
Owner: Compagnie des Bauxites of Guinea – Designer: Tecnasol FGE
- Contractor: Tecnasol FGE
Ricardo Esquível Teixeira Duarte Award 2020. Portuguese Geotechnical Society

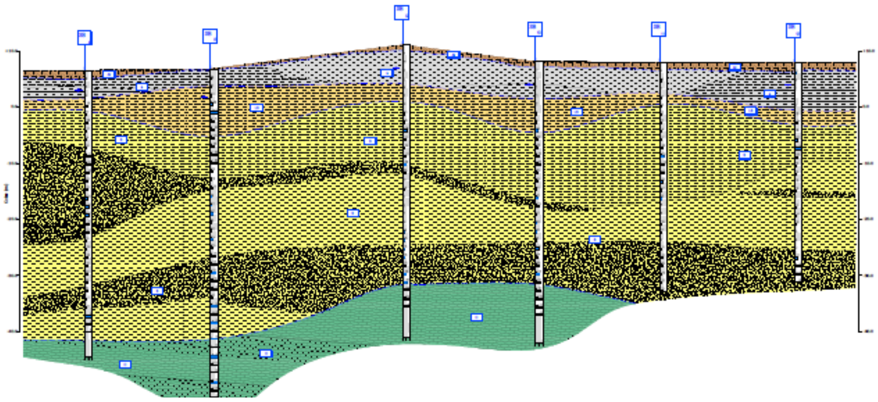
Description of the site characteristics

- Main goals

The main objective of the project was the execution of a structure to support the tipping mechanism of wagons with ore and the respective structure for a conveyor belt. It was part of the global project to expand the factory's production capacity.

- Geological and geotechnical conditions

From the survey carried out, it was possible to identify a recent-age clayey-sandy surface layer. Its consistency is medium to hard with average NSPT values between 30 and 40 strokes, except between depths of 15 m to 30 m depth where the consistency is soft to medium with average NSPT values between 8 and 15 strokes. At the base of the excavation, sandstone soil from Pita, Ordovician, was found. The water table was about 3 m deep.



Geological section

- Other conditionings

The most important constraints were the time schedule, as the work to be carried out could not interfere with work activity, namely with the existing railway line in the area surrounding the execution of the well and trench. The quality of the concrete produced, and the respective parameters obtained in the initial tests were not those prescribed by Tecnasol. In terms of work safety, it was also a challenge, as there were enough constraints to avoid incidents and/or accidents as much as possible. The excavation work will be carried out before the start of the rainy season.

- Design and general description of the solutions adopted

Circular pit

Tecnasol proposed the construction of a circular pit of Ø53 m as an alternative to the initial project that contemplated the construction of a rectangular structure. The following construction techniques were used: well structure, diaphragm wall 1.0 m thick and maximum depth of 46 m, 4 containment rings with 1.0 mx 0.8 m, wall joints reinforced in columns of Ø800 mm in jet-grouting and 0.7 m thick bottom slab, anchored in a 2.0 mx2.0 m mesh on TITAN 73/53 self-drilling micro piles with depths of about 30 m and a load capacity of 900 kN.

Diaphragm wall and retaining beams in the final excavation phase and after completion of the bottom slab

Trench

The trench was constituted by 2 lateral structures in diaphragm wall with 1.0 m of thickness, 9.0 m of width and maximum depth of 46 m next to the well, these two walls were supported laterally by horizontal levels of locking elements, with variable horizontal spacing from 2.0 m to 4.0 m between vertical lines and bottom slab, anchored in a mesh of self-drilling micro piles TITAN 73/53 load capacity of 900 kN.

Trench in the execution phase of micro piles

- Monitoring

Given the complexity and risks inherent to this type of construction, any displacements and deformations in the diaphragm walls were monitored, and a monitoring plan was implemented, including topographic measurement on the diaphragm walls and the railway line adjacent to the well and trench (topographic targets), measurement of horizontal and rotational deformations (inclinometers) and measurement of water level variation (piezometers).

Most relevant aspects of the project

The execution of a work of this complexity involved several areas of the company, namely Production and Design, as well as the entire Logistics and Supply chain. The main amounts of work were: 55 000 m^3 of excavation; 19 000 m self-drilling micro piles; 236 000 kg of A500 steel; 19 000m^3 concrete and 115 000 kg of cement.

2012

MAY FLOWER BUILDING. EXCAVATION AND RETAINING WALL. ANTIBES. FRANCE

General characteristics of the site

Location: Antibes, Côte d'Azur, France - Year of conclusion: 2012 –
Purpose: Residential Building
Maximum depth: 11 m - Perimeter: 70 m: CSM retaining wall - Area: 317 m^2
Client: SARL Foch - Contractor: Geo-Rumo - Project: Geo-Rumo

Description of the work

- Main goals

The construction of four underground floors of a residential building. The building also consists of ground floor, seven raised floors. The retaining wall was designed to allow the excavations required for the execution of the underground floors of the new building. The aim was to design a retaining solution that would minimise interference and disturbance in the area surrounding the work, ensuring that it would function safely during and after the execution of the works and limiting the influx of water into the excavation area. The main challenge encountered in this project consisted in excavating to an average depth of 11 m in a densely populated area, in the presence of soils with very variable mechanical characteristics, generally presenting low strength values and high deformability to depths below the excavation base, and in the presence of a water table close to the surface.

- Geological and geotechnical conditions

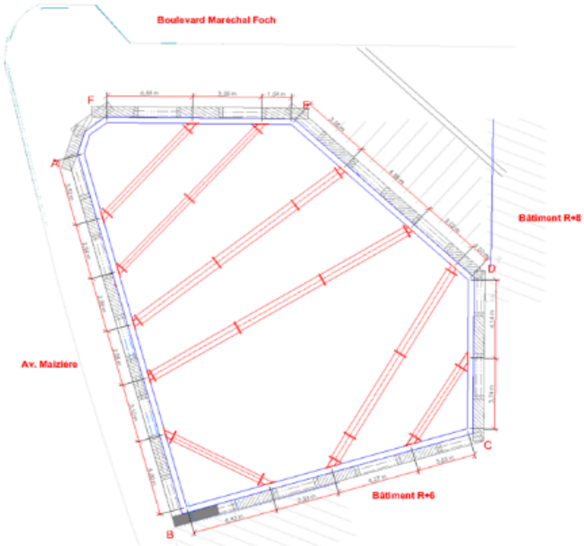
The characterisation of the existing soils at the construction site was carried out based on two geotechnical studies, carried out in different phases. Before the demolition of the existing building, a study involving a destructive borehole with pressuremeter tests (PMT) and a borehole with continuous sampling (core drilling) was carried out, where a piezometer was installed to measure the groundwater table. In a second phase a complementary geotechnical study was carried out, which included a destructive drilling with the execution of PMT tests and another continuous sampling drilling with the collection of intact samples for laboratory tests. The laboratory tests aimed to identify the type of soil and its mechanical characteristics, in particular triaxial tests. In this last borehole permeability tests were also performed at various depths. The results collected in the two surveying campaigns revealed the presence of three distinct layers: heterogeneous deposits and cover soils detected from the surface to a depth of about 2 m, predominantly fine-grained alluvial deposits with sand and gravel passages to depths of about 12 m, alluvial deposits with layers alternating between gravelly sand and silty sand. With regard to the water table, its occurrence was detected at a depth of approximately 2.2 m.

- Surrounding conditions

The excavation site is located in an urban area with tall buildings, which meant that one of the main concerns during the design of the solution and the execution of the work was to ensure the safe operation of the buildings and roads located near the site.

- Description of the solution adopted

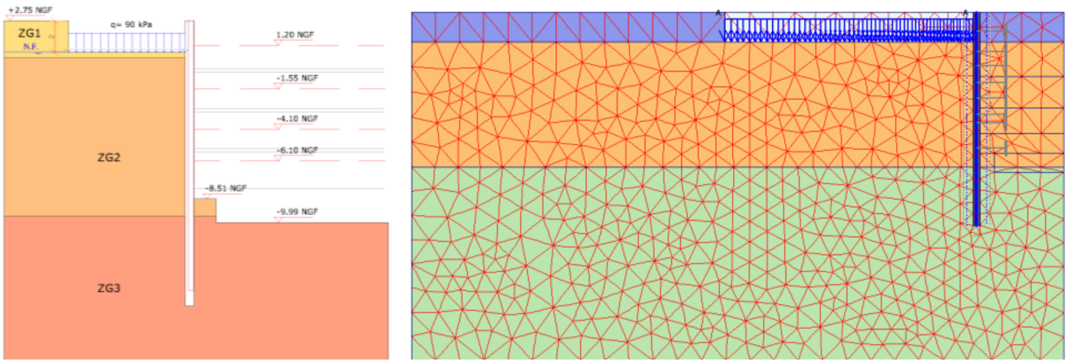
Based on the existing conditions, a peripheral retaining wall solution was chosen, consisting of a continuous wall of CSM panels, provisionally supported by four levels of metal struts, being the three lower levels pre-stressed. The retaining structure was executed sequentially by primary and secondary soil-cement panels, with rectangular transversal section of dimensions 2.40 m x 0.55 m and with minimum overlap of 0.20 m in order to guarantee an effective connection along the whole excavation height. Limiting the inflow of water into the excavation was one of the concerns during the design of the project. Therefore, a minimum embedded length of 6 m from the bottom of the excavation was defined, resulting in panels with an average length of 17 m. To accommodate all impulses, each CSM panel was vertically reinforced with two IPE 450 metal profiles. From the base of the excavation, the entire building was constructed in reinforced concrete, dimensioned to resist all the forces in the definitive phase.



Plan of the retaining wall and horizontal struts

- Key aspects of the project

The analysis of the retaining structure was performed using a finite element numerical model and the analysis included the study of the representative sections of all elevations of the retaining structure. The soils were modelled with Hardening Soil behaviour. In the following figure it can be seen, as an example, the numerical model of one of the sections studied: Section 5 (representative of the southeast elevation, building with 9 floors).

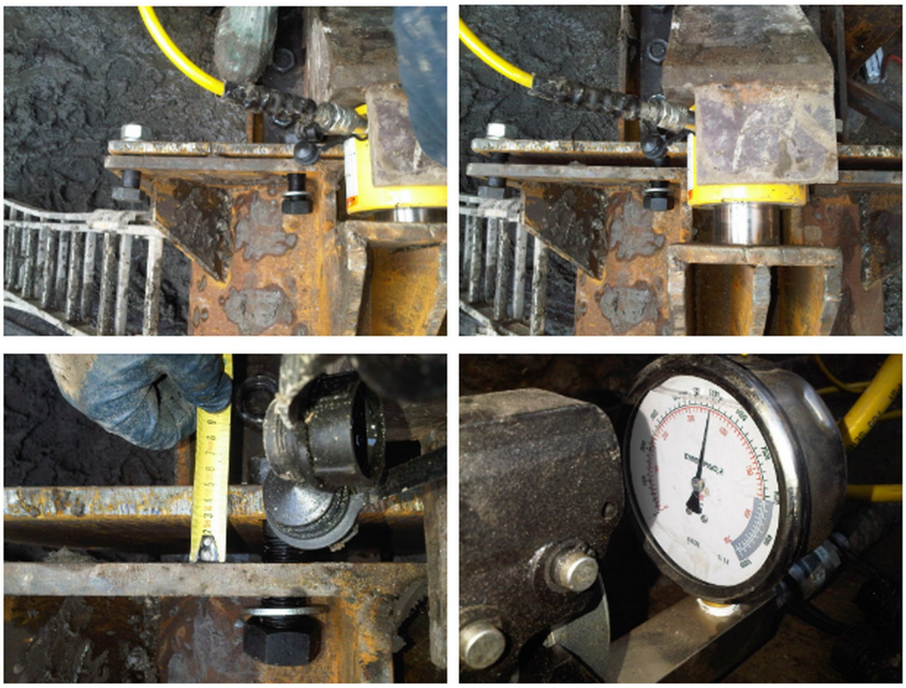


Section 5: Cross section (left) and numerical model (right)

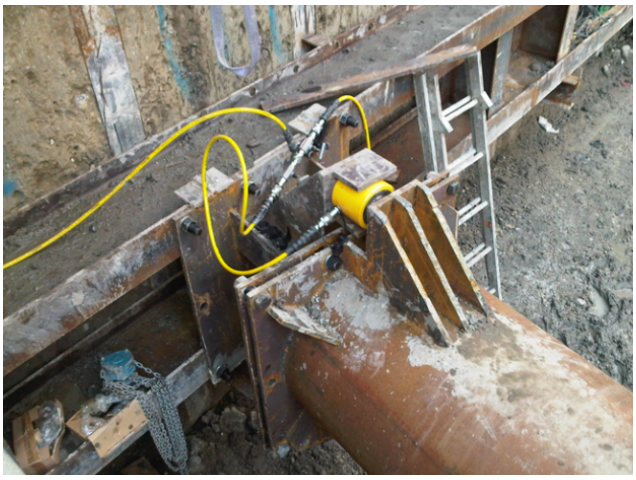
To simulate the construction process of the retaining structure and excavation, the calculation phases corresponding to the various phases of excavation and strut placement were considered. Given the characteristics of the intervention area and its surroundings, acceptable horizontal displacement limits of 5 mm at the top of the curtain and 40 mm along the height of the curtain were defined. In the numerical modelling of all the sections studied, values lower than these were predicted. The length of the CSM panels was defined from the safety verification of the rupture by hydraulic lifting, through the methodology proposed by Mandel and adapted by Cassan (Cassan, 1994), being then verified the safety of the hydraulic rupture by internal erosion ("piping").

- Key aspects of the work

The process of execution of the retaining structure began with the execution, from the surface, of the reinforced soil-cement panels, before any excavation work. The vertical profiles were installed immediately after the execution of each CSM panel, before they hardened, and positioned close to the inner face of the panels to facilitate the later joining of the metal distribution beams to support the internal struts. After the construction of the capping beam, excavation was carried out until approximately 0.50 m below the level of the first level of struts, followed by the execution of the metallic distribution beam and the installation of the metallic struts. The excavation for the following struts levels was carried out in the same way as for the first level. Starting from the second strut level, post tensioning was applied to the struts. The tensioning was applied with two hydraulic jacks supported between the tubular strut and the plate that allows the formation of the angle of the strut relative to the retaining wall. In order to make the application of the tension uniform, a single hydraulic pump connected to the two jacks through a "T" adaptor was used.



Post tensioning: a) Initial position; b) Final position; c) Deformation measurement; d) Hydraulic pump pressure check



Picture of the post tensioning system

After reaching the maximum excavation depth, the bottom slab and the erection of the whole reinforced concrete building structure were executed, accompanied by the process of deactivation of the supporting structure as successive slabs of the building's interior were built. The variability of the strength and deformability parameters of the soil-cement resulting from the application of the CSM technique is directly related to the degree of homogeneity of the mixture and it is influenced by several factors such as, for example, the type of soil involved, the presence of water, the way the binder distribution is processed in the disaggregated soil mass, the presence of air as a component and the chemical reactions that take place during the mixing process, among others. For these reasons, it is necessary to take special care and carry out effective control, both during execution and afterwards in the evaluation of the quality of the mixture in accordance with the design requirements. In addition to the control carried out by the equipment operator during the execution of the CSM panels, control is also carried out through laboratory tests complemented by any field tests, and on test panels executed for this purpose, which allow the execution parameters to be calibrated. The control during excavation was carried out through the Observation and Instrumentation Plan implemented, which included the placement of topographic marks on the retaining wall, inclinometers on the back of the retaining wall and strain gauges installed on the steel struts. The readings in the inclinometers showed deformations higher than those predicted by the numerical modelling, and only in one of the inclinometers, corresponding to the confrontation of the street to the northwest, the deformations exceeded the warning values. This situation led to an increase in the frequency of readings of all monitoring instruments. Despite the deformation of the curtain wall above the expected ones, the marks placed in the neighbouring buildings did not exhibited displacements. The maximum deformations in each phase of the works were below the excavation base and were therefore difficult to correct. However, a slight recovery of the deformations was always achieved with post tensioning of the struts.

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2012

CAR PARK. LARGO DO AMBIENTE. EXCAVATION AND RETAINING WALL. LUANDA. ANGOLA

Características gerais de obra

Location: Luanda, Angola – Year of conclution: 2012 - Purpose: car park
Maximum depth – 17,4 m - Perimeter: 350 m. CSM retaining wall, excavation by top-down - Area: 4 735 m^2
Client: Centro Cerro Angola - Contractor: Terzaghi/Geo-Rumo - Project: A400

Description of the work

- Main goals

The intention was to build an underground car park in the city of Luanda, Angola. The car park is designed as a slope of two to five underground levels, resulting in a maximum excavation height of 17 m. The excavation area faces directly against existing roads and has a roughly rectangular geometry with an area of approximately 4 735 m2. The geological and geotechnical conditions found at the site, namely the Luanda Formation (p1), composed of clays, siltstones, heterometric sands, sometimes alternated with fossiliferous limestones, highly heterogeneous, make its characterisation very difficult and lead to inaccuracies in the geotechnical models.

Aerial view of the site

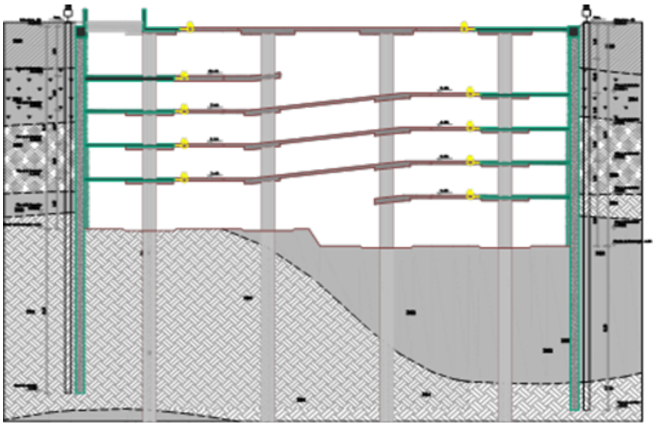
- Geological and geotechnical conditions

The geological and geotechnical study consisted of 8 mechanical drilling with a hollow auger up to 44.5 m and standard penetration tests - SPT. According to the study, the soils occurring in the intervention area are: Holocene soils of anthropic genesis and/or landfill, dune beach sands, sandy soils of variable granulometry, with SPT values between 1 and 31 blows; Pliocene formation, consisting of heterogeneous sands sometimes with silt underlying the Holocene soils, with SPT values between 0 and 60 blows. The water table was about 12.0m in the highest part of the site and 3.50m in the lowest part.

- Description of the solution adopted

The retaining wall for the excavations required for the underground floors of the car park consisted of a continuous wall of CSM panels, with a rectangular cross section of 2.40 m x 0.60 m, executed sequentially by primary and secondary panels, with an overlapping length between them of 0.20 m in order to ensure a good connection between adjacent panels in depth. Based on the efforts resulting from the hydrostatic pressure, the ground pressure and the surface overloads, each CSM panel was reinforced with two vertical profiles IPE 400 to IPE 270, depending on the excavation height. In order to control the influx of water into the interior of the excavation area and to guarantee its safety in terms of hydraulic stability and overall stability, a total length of the panels of 30.0m was proposed for the highest part and 20.0m for the lowest part. The CSM panels and their vertical profiles were supported at the top by a reinforced concrete capping beam. The retaining wall was materialized by two to five levels of slab bands, executed in a manner compatible with the excavation works and later incorporated into the structure of the floors, constituting rigid supporting frames for the four elevations. The provisional support of the slab bands was provided by 800 mm diameter foundation piles. A 0.20 m thick reinforced concrete coating wall was also built as the excavation progressed.

It should be noted that, in the particular situation of the work presented, if the retaining wall were to be supported by metal struts, the dimensions of the excavation area would require the use of struts of great length and diameter and, given the existence of direct confrontations of buildings, the use of anchors would not be viable. Thus, the use of slab bands to support the retaining wall was considered the most technically and economically advantageous option, with the main advantage being the incorporation of elements from the definitive structure to support it in the provisional phase.



Cross section of the solution

- Key aspects of the project

In the analysis of the retaining wall, seven sections representatives of the four elevations of the building site were selected for the design of the peripheral retaining walls. The behaviour of the sections studied was analysed using finite element software, where all phases of the construction process were analysed, in order to maximise the accuracy between the numerical modelling and the real conditions, having analysed the behaviour of the rock mass and the retaining wall in terms of stresses and deformations. The design of the retaining wall support was carried out taking into account the interaction between the CSM wall and the slab bands of the support. Thus, elastic supports were introduced in the analysis model of the peripheral retaining wall, the rigidity of such supports is intended to simulate the rigidity of the closed frame constituted by the sections of the supporting slab. The rigidity introduced in the finite element model of the peripheral containment was determined through the analysis of the closed frame model. It should be noted that, since this is a simulation in a plane state of deformation, the longitudinal construction phasing of the work is not considered. A study of the water flow that would occur inside the excavation was also performed, with a lowering of the water level by about 5.35 m in the most conditioning section, resulting in an estimate of the flow that would occur inside the excavation of 0.755 m3/s (2.716 m3/h). In the percolation analysis, the phenomena of hydraulic rupture by internal erosion, also known as "piping", was also evaluated. This rupture phenomena is associated with high hydraulic gradients near the base of the excavation.

- Key aspects of the work

The execution of the works began with the construction of the CSM panels, with the simultaneous execution of test panels, in order to allow the calibration of the execution parameters with the collection of soil-cement samples for uniaxial compressive strength tests with measurement of the modulus of deformability. After the initial equipment calibration phase, the CSM panels of the retaining wall were constructed. Immediately after the execution of each panel and before the hardening of the soil-cement, IPE profiles were introduced. The inclinometers were also installed in this phase to control the horizontal movements at the back of the retaining wall. The next phase consisted in the construction of the foundation piles that also served as provisional support of the slab bands.

Picture of the beginning of the execution of the roof slab

Due to the great thickness of sands, some difficulties were encountered in stabilizing them during the execution of the CSM panels, and Cutter was stuck in the sands twice. In both situations, the Cutter was recovered with difficulty, and in one of the cases, the Cutter was stuck for about a month. To guarantee the solidity of the CSM panels and the respective vertical profiles, a capping beam was built before the excavation work started. The excavation work began and the execution of the slab strips were constructed in order to obtain rigid support frames for the retaining wall and to install the planned topographic marks, continuing the phasing to the bottom of the excavation. It should be noted that the CSM panels were executed to a depth that minimised the inflow of water into the excavation area, which facilitated the whole excavation process. Once the maximum excavation depth was reached, the bottom slab and the definitive structure of the car park were constructed, and the temporary supporting piles were demolished. Quality control at the construction site was performed at different aspects. During the execution of the CSM panels, through the control of the execution parameters performed in real time by the equipment operator and through laboratory tests on samples collected from the panels of the retaining wall. At the same time, the construction site was monitored during the excavation and readings were taken from the monitoring devices installed according to the planned Observation and Instrumentation Plan. In general, the results obtained through the monitoring were consistent with the values predicted through the numerical modelling.

Partial view of the site during the excavation phase

Final aspect of the project

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2011

LUANDA TOWERS. TEMPORARY SHORING SYSTEM AND EXCAVATION. FOUNDATIONS. ANGOLA

General site specifications

Local: Luanda, Angola – Conclusion date: 2011
Purpose: design and execution of the retaining walls and foundations for 3 towers with a maximum of 28 floors, two of them for housing and one for offices, with excavation heights between 8.7 m and 16.8 m
Type: Peripheral retaining walls and foundations
Terrain: surface layer of landfill, over medium to coarse-grained sand, with fine-grained levels interleaved
Designer: Tecnasol FGE - Supervision: Cenor (today TPF) - Contractor: Tecnasol FGE

Description of the site characteristics

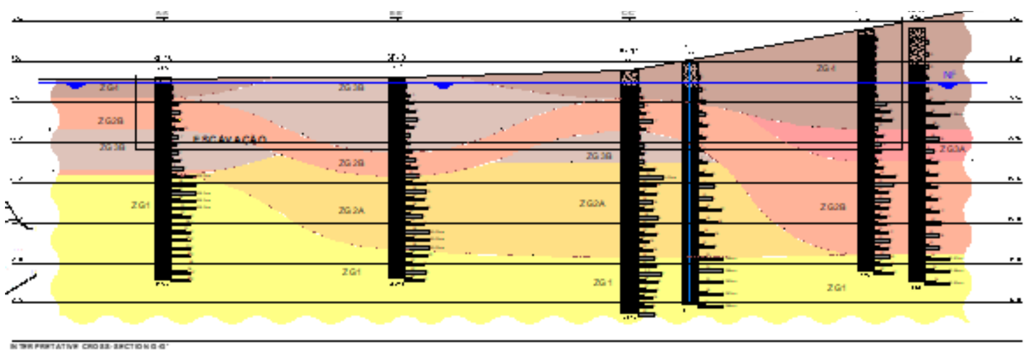
- Main goals

The Owner intended that the foundations, excavation and superstructure works were completely independent and out of phase in time, thus preventing solutions involving the construction of basement slabs (or bands of these same slabs) as shoring of the retaining wall in the excavation phase. The phasing of the excavation was also imposed by the Owner, in order to proceed first with the excavation of the area of one of the housing towers, thus enabling the start of work on the superstructure of this tower before the completion of all the excavation and foundation works.

Plan location

- Geological and geotechnical conditions

From the survey carried out, it was possible to identify a surface layer of landfill composed of loose sands, from fine to coarse granulometry, under which a recent Holocene formation develops, characterized by medium-compact to very compact sands, from medium to coarse-grained, with interspersed levels. fine granulometry. The results of the prospecting campaigns showed a great heterogeneity of resistance values that is verified between soundings, having been defined an exhaustive geotechnical zoning, to optimize the adopted solutions. The water levels recorded in the piezometers show elevation variations, obtaining a maximum of 9.65 m of water column at the bottom of the excavation.



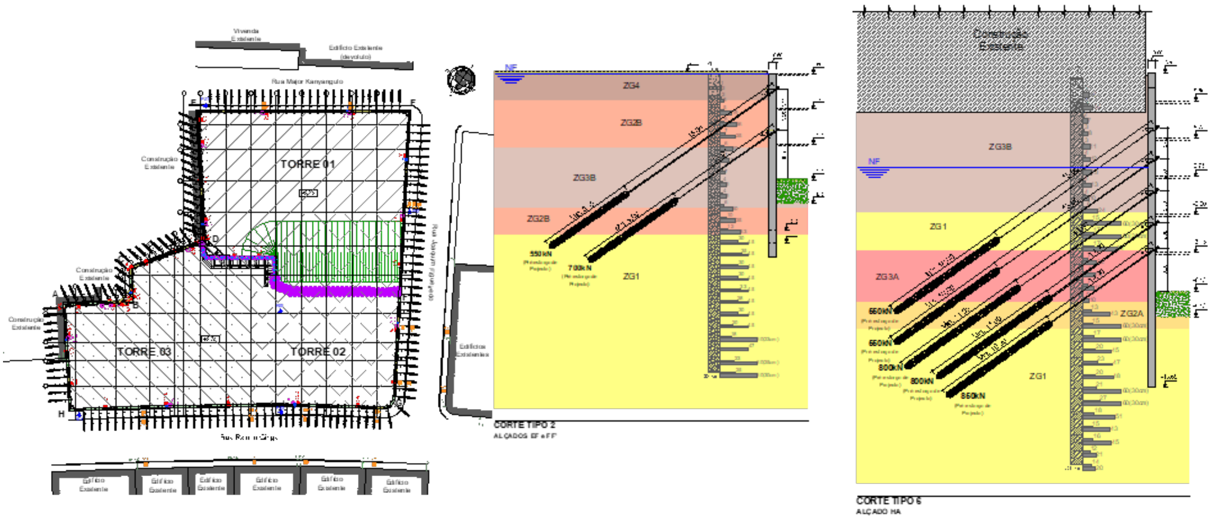
Geological profile and geotechnical section

- Other conditionings

The site area was occupied to the northwest, northeast and southeast by streets with small buildings and buildings with 5, 7 and 15 floors. On the southwest side, the work bordered on small, very dilapidated buildings, as well as a 12-storey building with a basement. The existing rainwater network had a drainage capacity limited to around 300 m³/hour, divided into two discharge points, one with ≈200 m3/hour and the other with ≈100 m³/hour.

- Design and general description of solutions

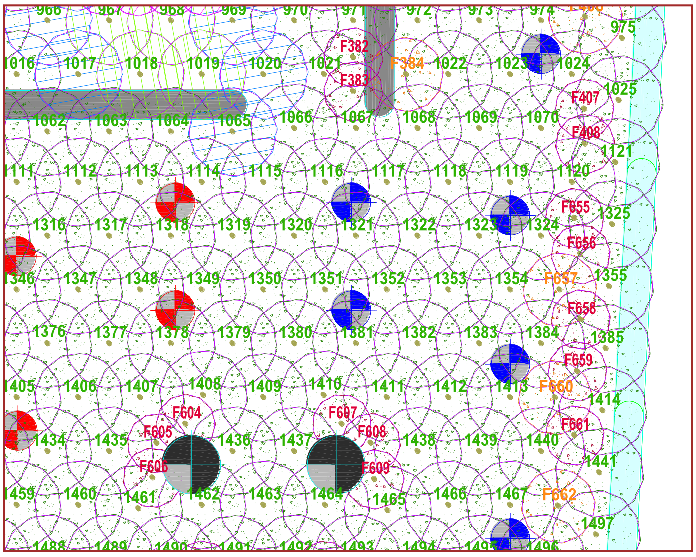
Given the data from the geotechnical survey and the various constraints identified, the following challenges were taken into account: watertightness; minimization of impacts on the surroundings; adequate rigidity; high strength; integration into the definitive structure; maximization of the useful area and independence from the superstructure contract. As a result, the adopted solution consisted of a diaphragm wall (e=0.60 m, temporary anchors) 2 to 5 levels, 550 to 950 kN // 2.5 m - with an exhaustive campaign of previous and suitability tests, reinforced crowning beams and strapping beams in the convex corners.



Plan and type cross-sections

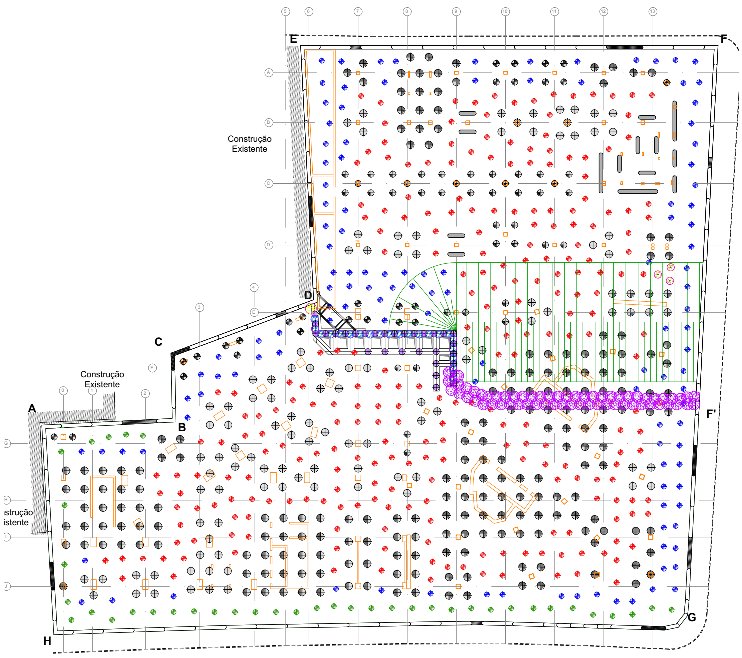
Retaining wall

The technical challenges and requirements of excavation and retaining wall, given the existing constraints, were: the removal and control of water within the excavation area (given the sandy nature of the terrain, it was impossible to reach strata of impermeable soil with the foot of the retaining wall, so the excavation would have to be assisted by a solution that allowed the flow or control of the high water table, in order to allow the progress of the works), the demanding deadline for the completion of the excavation, namely in Tower 01 , the minimization of the risk of negative impacts on the surrounding constructions, the phased excavation for the execution of anchors in the wall (including under water table) and the existing rainwater network with limited drainage capacity. The solutions adopted were the following: bottom plug in jet-grouting columns (e=2.0 m, general mesh columns Ø1800 mm and Ø1600 mm, 9.65 m of non-injected drilling): plug anchorage through definitive indirect foundations and of additional temporary pilesØ800 mm (maximum area of influence 3.6x3.6 m2), both previously executed and cofferdam system (inside) for excavation of the first tower consisting of a curtain of reinforced concrete piles (Ø1000 mm) with columns of jet-grouting, secured at the top by a reinforced concrete beam and excavation slope waterproofed by a jet-grouting curtain.



Detail of the floor plan of the jet grouting columns

The technical challenges and requirements of the foundations, in view of the existing constraints, were: high load capacity and maximum settlement of 2.5 cm, ability to mobilize competent soils in depth, work platform approximately at the level of the water table, ability to withstand horizontal impulses not balanced by the diaphragm walls (pulse imbalance scenario), ability to withstand temporary horizontal forces, by deformation of the bottom plug close to the diaphragm walls, compatibility with the initial design of the superstructure and drainage, including connection of the structure to the foundations through massive and beam grids and permanent pumping system over the plug, for reduced flow rates. The solution adopted included: groups of piles or barrels of reinforced concrete (8.65 m unconcreted hole, 12 m to 17.5 m of concrete length) made up of piles Ø1000 mm (N_{max} = 4 310 kN in service), piles Ø1200 mm (N_{max} = 6 180 kN in service), braces 0.60x2.80 m2 (N_{max} = 9740 kN in service) and braces 0.60x6.50 m^2 (N_{max} = 19 245 kN in service), performance of dynamic load test (pile Ø800 test, concreted to the work platform), integrity verification tests (cross-hole and sonic), the structural design of the foundations/structure connection was changed during the execution of the foundations and foundation slab (replaces the bottom cap in the final phase) with final mobilization of the plug anchor piles.



Localização das barretas, estacas de fundação e estacas de ancoragem

Most relevant aspects of the project

Given the size, constraints and geotechnical complexity of the work, it was necessary to carry out in-depth considerations and analysis in the project design phase. At this stage it was essential to understand all the requirements and impositions of the Site Owner since they made some of the possible solutions unfeasible. The performance of the 2 m thick jet-grouting bottom cap stands out, particularly due to the high constructive demands associated with its materialization in a large area. This type of solution was key to the feasibility of the work, responding to the main constraints, limitations and requirements involved. It was also possible, through the data obtained from the installed instrumentation, to confirm the good behavior of the diaphragm wall and the surrounding to the excavation (with regard to the aspect of deformation and safety of the existing constructions).

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2010

RESIDENTIAL AND OFFICES BUILDING. RETAINING WALLS FOR 6 BURIED BASEMENTS. LUANDA. ANGOLA

Project’s main features

Country: Angola - Location: Luanda- Conclusion year: 2010
Purpose: residential and office building
Type: anchored diaphragm walls - Maximum height: 25 m - Perimeter: 200 m
Area: 2 500 m^2 - Excavation volume: 67 500 m^3
Soil: sandy soils from Luanda’s formation
Client: ESCOM - Design: Teixeira Duarte - Technical assistance: Teixeira Duarte - Contractor: Teixeira Duarte

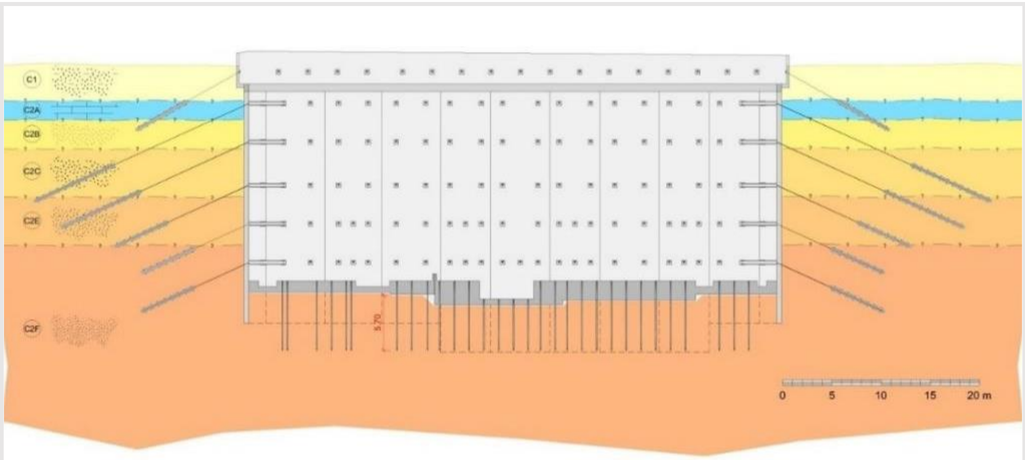
General description

- Geological and geotechnical constraints

The GES office and residential building is an Escom, Espírito Santo Real Estate Company property, built in 2010. Its architectural layout, with a square plan of approximately 50 m edge, features twenty-five storeys, six underground floors and a partially buried ground-floor. The characterisation of the geologic formations was based on a geotechnical survey which involved the execution of three boreholes to a depth of 40m. Standard penetration tests (SPT) were carried out in the boreholes at each 1.50m. The soil occurring at the site is characterized by the Tertiary complex known as “Luanda formation”, represented mostly by sandy, sometimes clayey, soils; the SPT tests, ranging from 6 to 60 blows, showed an increasing N_{SPT} blows in depth, being the average SPT over 40. At a depth of approximatively 5m, a limestone formation ranging from 1 m to 3 m thickness was detected. The occurrence of underground water was not detected by any of the piezometric devices installed in the boreholes.

- General approach and description of the implemented solutions

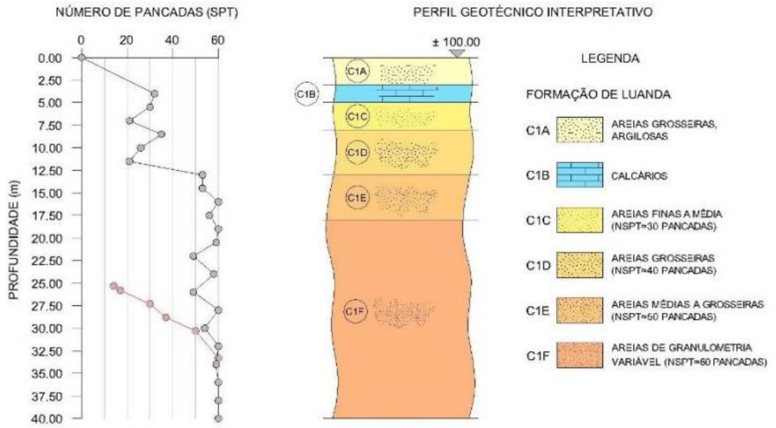
Considering the results of the geotechnical survey, the design determined the adoption of a raft foundation, executed at variable depths ranging from 23.50 m to 26.00 m, where the existing thin to coarse sand formations accomplish $N_{SPT's}$ of approximately 60 blows. The raft, 2 m thick with 2.5 m thickening zones on the core wall areas, imposes significant soil stresses which were analysed



in elastoplastic models considering a linear Winkler modulus of 100 000 kN/m^3 and soil plastification to a 600 kN/m^2 stress, corresponding to the soil’s estimated allowable stress. Given the significant excavation depth and the proximity of the site to existing roads and buildings in all it´s 4 confrontations, the solution adopted corresponds to a 0.50 m width cast in situ diaphragm wall with variable height, ranging from 25.00 m to 28.0 0m. To balance earth pressure throughout the excavation works, the retaining wall was stabilised by 328 temporary 600kN pre-stressed ground anchors, vertically distributed in five levels and horizontally spaced from 1.50 m to 3.00 m. Ground anchors length varied from 12 m to 24 m, with bond lengths from 6m to 9m, according to the geomechanical resistances of the Tertiary formations, in which N_{SPT} values vary from 30 to 60 blows. These resistances may be correlated with Menard pressiometer limit pressure with values between 1500 kN/m^2 to 3000 kN/m^2, and correspond to unitary lateral friction values from 150 kN/m^2 to 350 kN/m^2. The stability in the corner areas of the diaphragm wall was achieved with HEB200 metallic struts of 2.00 m and 5.00 m length, working in pairs. Due to the 25 m depth limit of the diaphragm wall equipment available in Luanda at the time, it was necessary to lower the working platform to ensure that the peripheral wall achieved the required depth. The first excavation phase was, therefore, sustained by a 3.5 m depth, 0.25 m thick reinforced concrete retaining wall, stabilised by 300 kN prestressed anchors.

- Diaphragm wall with 6 geotechnical anchor levels

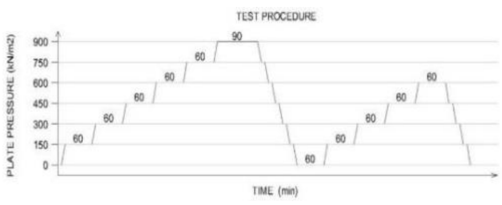
The design calculations of the retaining structure during the excavation works were performed using an elasto-plastic beam model, where the soil structure interaction was simulated by non-linear springs. In order to consider the non-linearity, the applied earth pressure loads were imposed according to an incremental method, corresponding with the established construction phasing. The monitorization/observation plan was composed by topographic marks, judiciously distributed along the retaining wall surface, where it was possible to register maximal horizontal displacement measurements of about 30 mm. Considering the sandy nature of the intersected soils and the significant excavation depth, these values, slightly above the calculation deformations but within the H/800 limit normally imposed for this type of retaining structures, have been considered as acceptable.



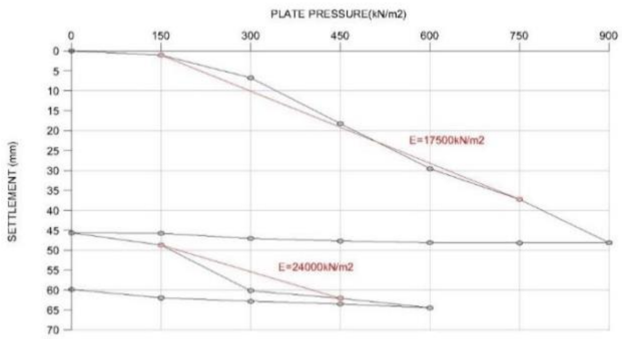
Representative range of the SPT test results showing is variation in depth

- Main construction singularities

The evaluation of the decompression effect on the post excavation sandy foundation soils was accomplished by comparative analysis of the N_{SPT} test results on 4 additional boreholes, executed after reaching the base excavation level. The complementary SPT testing results showed that the soil was significantly affected by the decompression phenomena which transformed the pre-excavation 60 N_{SPT} blows layers into a soil presenting an average resistance of 25 N_{SPT} blows, in a depth up to 6.0 m below the excavation base. The considerable geomechanical reduction of the 6 m layer of soil underneath de raft foundation slab, resulting from the significant decompression phenomena due to vertical stress reduction, would have direct implications in the behaviour of building foundation system, since the significant vertical loads, where the columns are loaded with axial stresses of approximately 20 MN, would result in vertical settlements of about 54 mm, which would not be compatible with the demanding serviceability limit states of this important building. This phenomenon was confirmed by a plate bearing test, carried with a ϕ1.50 m circle metal plate incrementally loaded by 4 prestressed ground anchors. This test was performed using two load cycles with maximum stresses of 900 kN/m^2 and 600 kN/m^2, applied with load intervals of 60 to 90 minutes and stress increments of 150 kN/m^2. This evaluation method resulted in settlements of 45mm for the first loading cycle and in 20 mm settlements in the reload procedure, which confirmed the settlement values previously achieved in the numerical model analysis.



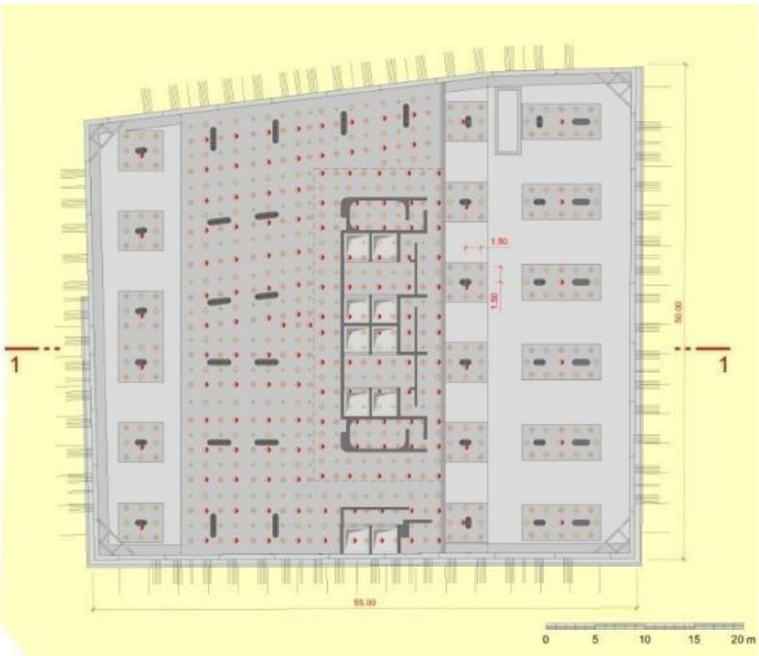
Cycles and load intervals considered in the test



Settlement evolution results along the two load cycles

Load test with a circular ϕ1.5 m plate with loading imposed by prestressed ground anchors

The need to control/decrease vertical settlements led to the design of a soil improvement solution that would restore the initial geomechanical resistance of the soil, before the decompression phenomena due to the excavation. This soil improvement was achieved by the execution of cement grout injections, which were carried out through 715 segments of steel circular hollow sections (CHS), previously installed in the raft slab structure in a square grid with 1.5 m spacing. The injection process was synchronised with the critical activities of the project phasing, avoiding delays in the strict construction schedule. The improvement procedures started with the pre-drilling operations in depths that ranged from 5 m to 6 m, followed by the injection operations carried out by gradual 50 kPa pressure increments until the maximum total pressure value of 500 kPa was reached. These proceedings were coordinated with the construction of the building's structure, by adjusting injection pressures to the increasing vertical stress conveyed by the growing structure. The measurements, made throughout a period approximatively 10 months using topographic marks distributed over the walls and columns of the ground floor level, registered vertical absolute settlements inferior to 5mm. This value is rather acceptable and is correspondent to approximatively 10% of the estimated value before the soil injections, proving the effectiveness of this soil improvement technique in sandy soils.



Distribution of the injection grid


Slab reinforcement cage with incorporation of the steel CHS pipes through which the injection was executed


Conclusion of the raft foundation slab enlightening the injection pipe location

2010

HOTEL MONTAIGNE. EXCAVATION AND RETAINING WALL. CANNES. FRANCE

General characteristics of the site

Location: Cannes, Côte d'Azur, France – Year of conclusion: 2010 - Purpose: Hotel
Client: Locavim - Contractor: Geo-Rumo - Project: JETsj
Maximum depth – 9 m - Perimeter: 48 m - CSM retaining wall. Area: 290 m^2

Description of the work

- Main goals

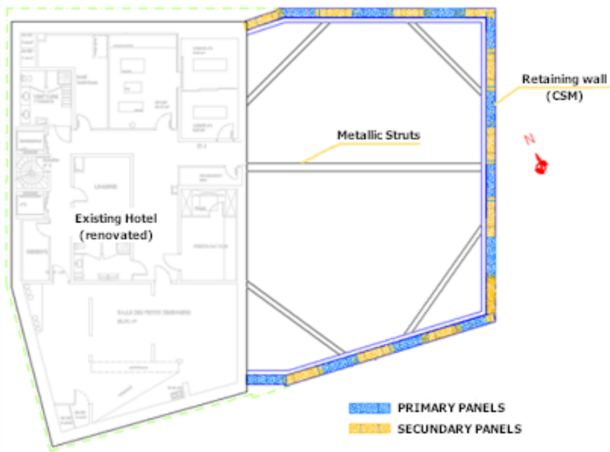
The existing hotel was renovated and expanded, gaining an additional floor and a new building with seven elevated floors and three underground floors. The retaining structure was designed to allow the excavation required for the execution of the underground floors of the new hotel building. The aim was to design a retaining solution that would minimise interference and disturbance in the area surrounding the work, ensuring that it would operate safely during and after the execution of the works and, at the same time, limiting the influx of water into the excavation area. The proposed solution consisted of a continuous wall of soil-cement panels built using Cutter Soil Mixing (CSM) technique, vertically reinforced with rolled steel profiles, associated with two levels of horizontal metallic struts. The excavation area, with a trapezoidal shape, has an area of approximately 290 m2 and is surrounded by roads and existing buildings. The main challenge encountered consisted in the execution of the excavation with an average depth of 9 m in a densely urbanized area, in the presence of soils with very variable mechanical characteristics, generally presenting low resistance values and high deformability to depths below the excavation base, in the presence of a water table close to the surface.

- Geological and geotechnical conditions

The geological and geotechnical survey consisted of two phases, in a total of three boreholes with pressuremeter tests (PMT), installation of two piezometers and collection of intact samples for laboratory tests, including triaxial tests. The results collected in the surveying campaigns carried out on the site revealed the presence of three distinct layers, namely, heterogeneous clay and silty embankments detected from the surface to a depth of about 5 m, sandy clays detected to depths of about 12 m and the rocky substratum (sandstone) with altered, less compact layers. The water table was detected at a depth of approximately 3.5 m.

- Description of the solution adopted

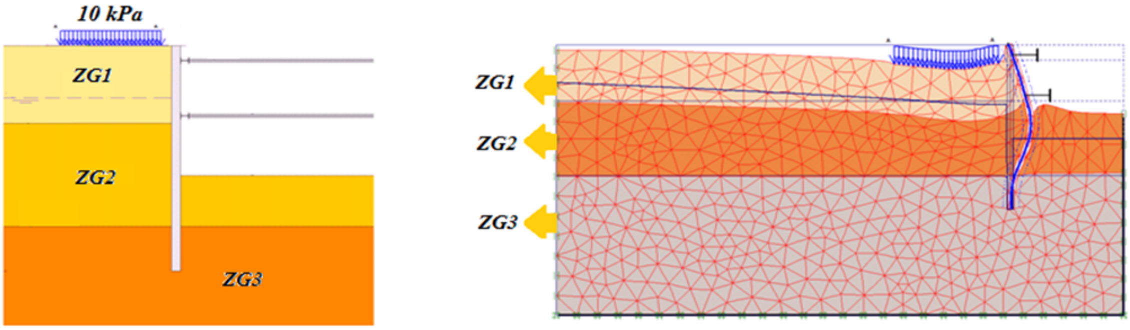
A peripheral retaining wall was designed and built, consisting of a continuous wall of CSM panels, temporarily restrained by two levels of metallic struts. The retaining structure was executed sequentially by primary and secondary soil-cement panels, with rectangular transversal section of dimensions 2.40 m x 0.55 m and a minimum overlap length of 0.20 m between adjacent panels in order to guarantee an effective connection between them along the whole excavation height. Limiting the influx of water into the excavation was one of the concerns during the design of the project, and to this end a minimum embedment length of 3 m in the rock substrate was defined. Thus, the CSM panels were executed with an average length of 15 m. To accommodate all the forces caused by the soil and by the overloads of the areas located behind the retaining wall, the hydrostatic impulse and the vertical loads transmitted in the definitive phase, each CSM panel was vertically reinforced with two IPE 450 profiles made of S 275 JR steel. The placement of the profiles inside the CSM panels results in a protection of these elements against possible buckling phenomena due to the confinement provided, while minimising corrosion phenomena caused by the aggressiveness of the surrounding terrain. A 0.15 m thick reinforced concrete coating wall was also built from the excavation base, connected to the IPE profiles of the containment structure through horizontal HEA metal profiles. The retaining structure was designed to, together with the micropiles executed below the bottom slab inside the excavation, accommodate the vertical forces transmitted by the final structure.



Plan of the retaining wall and the horizontal support

- Key aspects of the project

The retaining wall was analysed using a finite element numerical model. The soils were modelled with the Hardening Soil behaviour model. The analysis included the study of the representative sections of all elevations of the retaining wall.



Section A (representative of the north elevation): Cross section (left) and numerical model with deformed mesh of the last excavation phase (right)

Given the characteristics of the intervention area and its surroundings, the following maximum acceptable horizontal displacements were defined: 5 mm at the top of the wall and 20 mm along the height of the wall. In the numerical modelling of all sections studied, values lower than these were predicted, and it was verified during the execution of the work that the values of horizontal displacements obtained were still slightly lower than the predicted values. The CSM panels were designed for a compressive stress not exceeding 2 MPa and for a value of the modulus of deformability higher or equal to 1 GPa. In the case of the compressive strength, a safety factor of 2 was considered, thus imposing that the soil-cement samples collected and submitted to uniaxial compressive strength tests in the laboratory had a minimum value of rupture stress in compression of 4 MPa.

- Key aspects of the work

During the installation phase of the CSM equipment on site, demolition work had not yet started on the building directly facing the work on the east elevation, which made installation of the equipment difficult.

Installation of the equipment

Equipment and cutting tool (left) and execution of the CSM panel (right)

The continuous retaining wall was created by the sequential execution of primary and secondary CSM panels, with an overlapping length of 0.20 m between them. The secondary panels can be executed immediately after the execution of the primary panels or after the hardening of the primary panels. These two possibilities are possible, given the versatility of the cutting tool for making the CSM panels, allowing the application of the technology to all soil types, but not achieving the same effectiveness in all soil types. According to experience, it can be stated that this technique presents better results when applied to sandy soils, obtaining in clayey and silty soils lower resistances for the same quantities of cement. The next phase consisted of the execution of a small excavation to allow for the construction of the capping beam. Once the capping beam was executed, excavation took place until approximately 0.50 m below the first strut level, followed by the execution of the metallic distribution beam and the installation of the metallic struts. The excavation for the 2nd supporting structure was carried out in the same way.

Detail of the steel distribution beam directly connected to the vertical profiles of the retaining wall

After the maximum excavation depth was reached, the bottom slab and the reinforced concrete coating wall were executed, followed by the process of deactivation of the struts as successive supports materialized by the slabs of the building's interior structure were built. The variability of the strength and deformability parameters of the soil-cement resulting from the application of the CSM technology is directly related to the degree of homogeneity of the mixture and is influenced by several factors such as, for example, the type of soil involved, the presence of water, the way the binder distribution is processed in the disaggregated soil mass, the presence of air as a component, the chemical reactions that take place during the mixing process, among others. For these reasons, it is necessary to take special care and carry out an effective control, both during execution and afterwards in the evaluation of the quality of the mixture according to the project requirements. One of the main advantages of applying CSM technology compared to other alternative techniques, in addition to the economic and environmental aspects (use of soil in situ as a construction material), is the possibility of real-time control of execution parameters by the equipment operator. During the execution of a CSM panel, the operator has an instrument panel that allows him to monitor and correct, in real time, parameters such as the speed of advance of the cutter wheels, the amount of binder added, the density of the fluids involved (A/C ratio) and the verticality of the panel, among others. In addition to the control performed by the equipment operator during the execution of the CSM panels, the control is also performed through laboratory tests complemented by any field tests on test panels executed for this purpose, which allow the calibration of the execution parameters. Once the execution parameters are calibrated, the production of the CSM panels of the retaining wall begins, from which samples are also collected for laboratory tests. The control during the execution was also performed through the Observation and Instrumentation Plan implemented, which included the implementation of topographic marks in the retaining wall and in the slab bands, inclinometers on the ground located at the back of the retaining structure and strain gauges installed in the steel struts. The readings taken in the monitoring devices throughout the execution of the excavation made it possible to verify that the displacements were slightly lower than predicted, being below the values established as warning limits for each of the excavation phases.

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