Excavation with old facades preservation

Critical analysis of the Jasmin Noir building

Freke De Roeck

Thesis to obtain the Master of Science Degree in

Civil Engineering Technology

Supervisor: Prof. Alexandre da Luz Pinto

Examination Committee:

Chairperson: Profª Maria Rafaela Pinheiro Cardoso

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Declaration

I declare that this document is an original work of my own authorship and that it fulfills all the requirements of the Code of Conduct and Good Practices of the Universidade de Lisboa.

Acknowledgement

First, I would like to thank my supervisor Prof. Alexandre Pinto for his continuous support and enthusiasm in all stages of this research. His guidance, observation and availability to discuss all details of this work are highly appreciated and, undoubtedly, made this milestone in my academic journey much easier and enjoyable! In addition, I would like to thank the mobility coordinators of civil engineering at Técnico, Prof. Ana Paula Pinto and Ms. Cristina Ventura, as well as the Mobility Coordinator in Belgium, Prof. Anthony Tetaert, for their assistance during this Erasmus experience that greatly contributed for the present research.

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Finally, I want to thank my Swedish Erasmus friend Linnea for being the motivation to wake up and join her to the library of the civil engineering building at Técnico, without which this endeavour would never be accomplished.

Thanks for all the encouragement.

Abstract

The increasing occupation of the urban underground space has consequences for existing buildings, which often are not properly assessed when projects involving excavations and peripheral walls are carried out. Thus, it is necessary to make studies and analysis addressing this issue, in order to provide a scientific basis for the essentially empirical construction methods, which depend heavily on the geological and geotechnical conditions. Chapter two of this thesis gives a general historical overview of Lisbon and its rebuilding actions after the Great Lisbon earthquake. In chapter three a few main earth retaining systems are outlined, such as temporary and permanent King Post walls and the use of ground anchors. Also a brief description of micropiling and jet grouting underpinning techniques is presented.

This thesis is centred on a construction work called the 'Jasmin Noir' building, located at the *Príncipe Real* square in Lisbon. An underground car parking was performed using Munich type walls. More detailed information about this case study can be found in chapter four of this work. An analysis of the displacements was made through the finite element program PLAXIS 2D in chapter five.

Finally, two alternative solutions have been studied in chapter six to examine if the solution performed was possible to become optimized. In order to compare their viability a technical and economic study was made.

KEYWORDS: Great Lisbon earthquake; Earth retaining systems; Underpinning; Munich type walls; PLAXIS 2D; Alternative solutions.

Resumo

A crescente ocupação do espaço subterrâneo em zonas urbanas tem tido consequências no edificado existente, em particular quando as obras subterrâneas, como as escavações para a construção de pisos enterrados, são executadas junto ao mesmo edificado. Este cenário, determina a importância dos trabalhos de escavação serem desenvolvidos através de projetos, baseados numa correta avaliação de todos os condicionamentos, em particular os de natureza geológica e geotécnica. Capítulo um da presente Tese é abordada a reconstrução da cidade de Lisboa após o terramoto de 1755. Capítulo tres são descritas algumas das principais soluções de contenção periférica, tais com as soluções de contenção provisória tipo Berlim e definitiva tipo Munique, assim como a utilização de ancoragens seladas no terreno. É igualmente efetuada referência às técnicas de recalçamento com recurso a microestacas e a colunas de jet grouting.

A Tese é baseada no caso de obra de demolição, com contenção de fachadas, escavação e contenção periférica do edifício "Jasmin Noir", localizado na *Praça do Príncipe Real*, em Lisboa, descrito nos quarto e quinto capítulos do trabalho. Os trabalhos de escavação, para a construção dos pisos enterrados para fins de estacionamento automóvel, foram realizados ao abrigo de uma contenção periférica tipo Munique. A estimativa das deformações da parede de contenção foram obtidas através de uma modelação numérica através do software PLAXIS 2D, apresentada no quinto capítulo. No sexto capítulo são descritas duas possíveis soluções construtivas alternativas, incluindo as consequências das mesmas na modelação através do software PLAXIS 2D. Por último, no sétimo capítulo é efetuada uma análise comparativa, técnica e económica, das soluções estudadas.

PALAVRAS CHAVE: Terramoto de 1755; Soluções de contenção periférica; Técnicas de recalçamento; Contenção definitiva tipo Munique; PLAXIS 2D; Análise comparative.

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Abbreviations

σ	Stress
ε	Strain
E	Young's modulus
ν	Poisson's ratio
τ	Shear strength
c	Cohesion
ϕ	Angle of internal friction
ψ	Angle of dilatancy
E_{50}	Triaxial loading stiffness
E_{ur}	Triaxial unloading stiffness
E_{oed}	Oedometer loading stiffness
a_{gR}	Maximum reference acceleration values
K	Coefficient of permeability
γ	Specific weight;
ΣM_{stage}	Total multiplier associated with the staged construction process.

Appendices

1. Introduction

As a Belgian Erasmus student, the opportunity arose to study an excavation work in Lisbon city, where the techniques used fit the content of the dissertation and ranged from King Post walls, concrete slab bands, micropiles to retaining and underpinning facades. When King Post walls are structures which consist of metal profiles with between them, profiles of wood or precast concrete panels (Patrício, A., & Teixeira, R., 2006), these types of retaining walls are temporary and they are called Berliner walls. Although, when the execution of the walls is a permanent solution that uses reinforced concrete poured in site, supported by micropiles staled at the ground vertically, this technique is called Munich walls. These walls can be used as the final wall of an underground floor, like in this case study. The 'Jasmin Noir' building was built more than hundred years ago and is not habitable anymore. The facades must be preserved as a request of the Lisbon Municipality and there will be an excavation because the basement will become a place for two parking lots. This construction site and the performed work was studied during the spring semester, 11 site visits are made.

Figure 1: Jasmin Noir building

The case-study was analysed using a Finite Element model, PLAXIS 2D, which gave an overview of the expected deformation of the soils and could be compared with the alert and alarm criteria. Several companies who design and develop geotechnical engineering solutions are asked to propose other possible solutions for the Jasmin Noir project, to investigate if it's possible to optimize the techniques. Belgian and Portuguese companies have different experience gathered through numerous geotechnical projects, so they give different solutions. These solutions are examined technically, practically and economically for this case study. The frequency of use of any engineering technique depends mainly on the technical feasibility and economics of the system. In geotechnical engineering, the more problems a construction technique can solve, and the more soil types in which it is effective, the more applications will be available for the system's use.

2. Lisbon historical overview

2.1 General outline of Lisbon

About 180 million years ago, in the Early Jurassic period, Europe and Africa started to separate from America creating the Atlantic Ocean. This separation caused a major reshape on the Iberian plate, resulting in the creation of several sedimentation basins. The Lower Tagus Basin was one of the most important, occupying a large area in Portugal. The city of Lisbon is located in the centre-south western coast of Portugal, with an official county presenting an area of about 84 km². (Pedro, A., 2013)

When we want to know more about the geological formations of Lisbon throughout the years, we have to take a closer look at the geologic time scale. The GTS is a system of chronological dating that relates geological stratigraphy to time. It is used to describe the timing and relationships of events that have occurred during earth's history. The primary defined divisions of time are eons, divided into eras, which are in turn divided into periods, epochs and ages. The city of Lisbon is rich in different subsurface geological formations, dating from the Cretaceous to Miocene, which are covered by formations of the Pleistocene and Holocene. (Matildes, R., Taborda, R., Almeida, I. M., et. al. ., 2010).

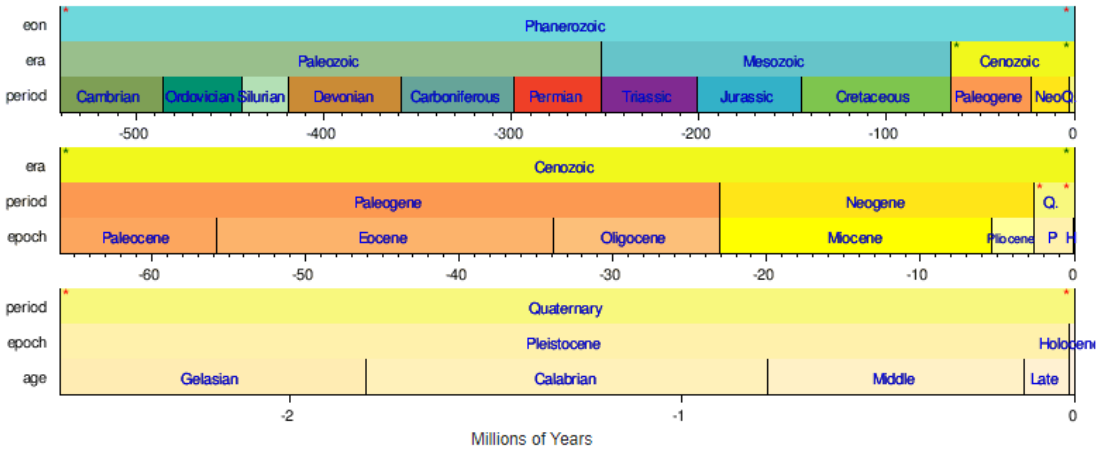

Figure 2: Geologic time scale (https://en.wikipedia.org/wiki/Geologic_time_scale)

The Miocene deposits of Lisbon have been intensively studied through the years by several authors, since the majority of the city of Lisbon is founded upon these soils. The oldest superficial soils discovered in the city are limestone and marls that were deposited in a marine environment following a large transgression of the sea that occurred in the Cretaceous period (95 Ma). These deposits, only visible due to intense tectonic activity, do not surpass 100m in thickness in Lisbon. Except for the Volcanic Complex of Lisbon (CVL) formation, the subsurface geology of Lisbon consists of sedimentary formations, including Cretaceous limestones and marly limestones, Eocene sandstones, clays and conglomerate rocks. The Miocene is characterized by clayey, sandy and silty soils, calcareous sandstones and limestones. There are important lateral and vertical facies variations, registry of the alternans of sea and continent environments, originating 15 stratigraphic units within the Miocene. The basaltic volcanic formation is characterized by important lateral variations of thickness (Almeida, 1991), and of structure with lava flows, interbedded pyroclastic layers and, in some locations, sedimentary

layers within the volcanic formation (Pais J. et al., 2006). The set of geological formations to model and the geological cartography of Lisbon can be found in Figure 3. The “Prazeres Clays” unit of Lisbon’s marine Miocenic constitutes one of the most identified geological formations by means of boreholes and geological surveys. (Marques et al., 2006)

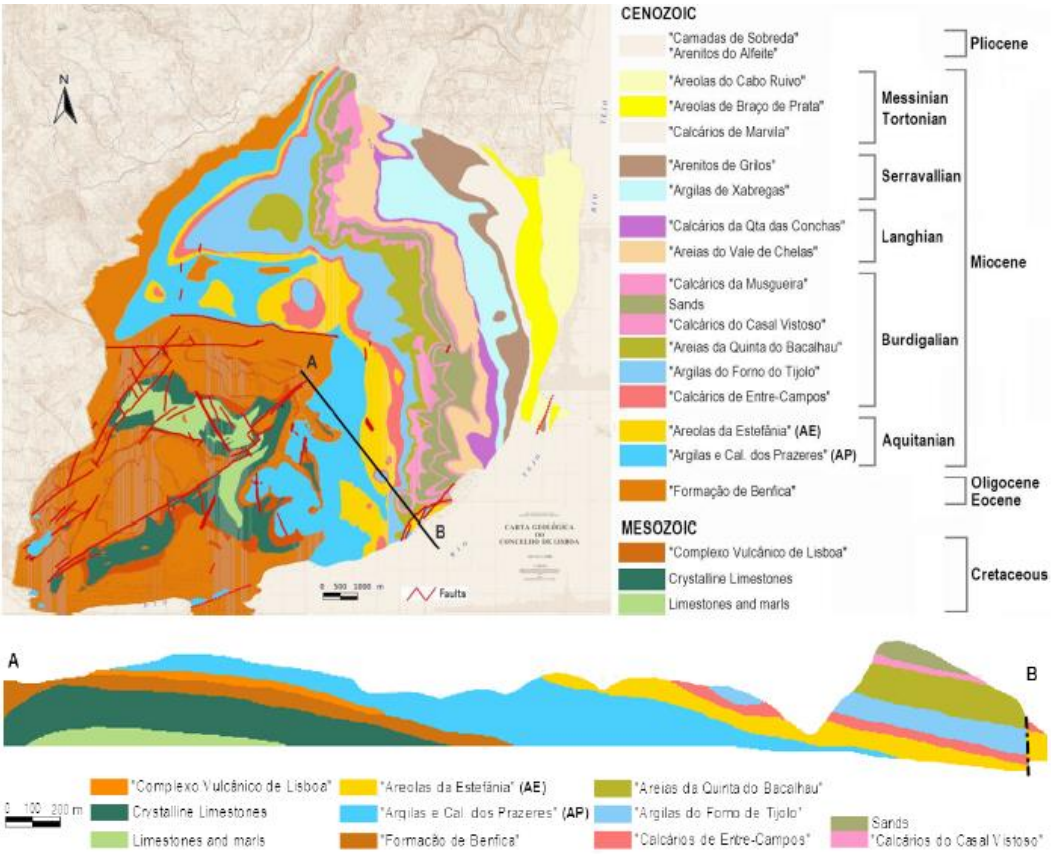

Figure 3: Geological cartography Lisbon (Matildes, R., Taborda, R., Almeida, I. M., et.al.,2010.)

In terms of population, Lisbon has always been the largest Portuguese town. At the end of the 13th century Lisbon was the largest town of Portugal, with the highest population density as well. In this time it had an estimated population of 240 740 inhabitants. (Baptista, L. & Rodrigues, T., 1996) According to a 2015 census, there are 506 892 people within the city of Lisbon. When talking about the population of Lisbon it is important to consider the fact that the city limits are different than the outlying urban areas. Nowadays, when the outlying urban areas are considered (Lisbon Metropolitan Area or LMA), the population grows dramatically. It is estimated that in the areas surrounding Lisbon, there are 2 800 000 people. Lisbon is the largest urban area in the EU and continues to grow with each passing year. The city houses the administration of Portugal and is therefore a hub for both residents of the city as well as international guests. (Baptista, A., 2014)

2.2 The 1755 Lisbon’s earthquake

Through its history, Portugal mainland has experienced the effects of various moderate to strong earthquakes, thus presenting a moderate seismic risk, more important in the Lisbon and Lower Tagus Valley, and Algarve regions. (Teves-Costa, P. & Almeida, I. M., 2004) In 1755, a great earthquake

devastated Lisbon, destroying or rendering uninhabitable most of the wealthy city's buildings. This earthquake, also known as the “Great Lisbon Earthquake” and “the Disaster at Lisbon”, occurred in the Kingdom of Portugal on Saturday, 1 November 1755, the Catholic holiday of All Saints’ Day, at around 9:40 a.m. On this day the deeply religious Portuguese packed Lisbon's churches and cathedrals to celebrate the important feast day. As part of the religious celebrations every possible candle was lit and the churches were decorated with flowers and flammable decorations. There were three distinct quake shocks over a ten minute period. The first shock was followed by an even more powerful second shock and a third shock which was less powerful. Seismologists today estimate the Lisbon earthquake had a magnitude in the range 8.5–9.0 on the moment magnitude scale. The strength was enough to bring down the solid stone walls of the *Ribeira Palace* and roofs of dozens of churches across Lisbon. As the tremors rocked the churches the candles tumbled and ignited the flowers. These fires ravaged Lisbon for five further days after the earthquake. Approximately 40 minutes after the earthquake, a tsunami engulfed the harbour and downtown area, rushing up the Tagus river. This first tsunami wave was followed by two more waves which hit the shore. (Teves-Costa, P., & Almeida, I. M., 2004) Eighty-five percent of Lisbon's buildings were destroyed, including famous palaces and libraries, as well as most examples of Portugal's distinctive 16th-century Manueline architecture. Several buildings that had suffered little earthquake damage were destroyed by the subsequent fire. Because most of the effects of the offshore quake were caused by the massive tsunami and widespread fires that followed, rather than by ground shaking, it was believed that were a similar event to occur today, modern tsunami warning systems and disaster response practices, as well as superior building construction, would moderate the scale of damage and casualty. The geotechnical soil characterization is of the utmost importance for seismic risk assessment, being used, in particular, for site effect assessment. They depend mainly on the geological, geotechnical and topographic site characteristics. The large number of old masonry buildings present the most significant potential for large loss earthquakes in Lisbon. (Franco, G., & Shen-Tu, B., 2009)

Figure 4: 1755 Earthquake location (https://en.wikipedia.org/wiki/1755_Lisbon_earthquake)

2.3 Lisbon rebuilding actions and plans

Three typologies of masonry buildings are usually recognized in the Lisbon County: *Pombalino* buildings built after the 1755 earthquake, *Gaioleiro* buildings built between 1870 and 1930 and *Placa* buildings, a short-term structural solution which precedes the reinforced concrete buildings. The date of construction might be an indicator of their seismic resistance. For instance, the first anti-seismic construction in Portugal appeared with the *Pombalino* buildings built after the 1755 earthquake and systematically imposed during the whole reconstruction program. During the nineteenth century, these construction methods were gradually abandoned resulting on the design of buildings with inferior constructive quality, known as *Gaioleiro* buildings. The introduction of reinforced concrete solutions (ring beams and columns) between the 1930-1950 decades represented, in most cases, an improvement to the resistance of the buildings. (Simões, A., Lopes, M., Bento, R., & Gago, A., 2012)

2.3.1 Before the earthquake

The buildings that remain after the 1755 Earthquake belong to a very heterogeneous group. Actually, it is not possible to define a specific typology of buildings, as they emerged from several centuries of history without a proper urban planning. The fast-growing population in that time brought with it the need of building more and more houses, as well as the necessity to create all the infrastructures required in a city. Thus, and as the city rapidly increased, a typical Muslim city (heritage left by the occupation of Muslim forces during the Middle Ages and Renaissance) was being raised in a chaotic and poorly organized environment. (Mascarenhas, 2005; Oliveira, V., & Pinho, P., 2010) The buildings were constructed in the insecurity of alluvial soils additionally to a complete disregard or any attention for circulation, sanitation and safety against natural phenomena (like earthquakes) or others. The constructed buildings, until 1755, had many flaws, mostly of them structural, that represented an imminent risk for the population in case of catastrophe. First of all, the balconies protruding the facade with poorly connections to the structure as well as the chimneys in the roof tops, signify a great risk in case they will detach from the main structure and fall apart in the streets striking any person escaping. Other example was the construction of buildings on top of arcades. This solution was intended to give a wider and spacious area for the improvement of all commercial trades. However, if it was a good solution for the trade, it was not the best solution for this kind of buildings, considering that an open space at the ground floor would collapse more easily than a standard floor with walls. Additionally, the use of unprotected timber represented a key-factor to an easy fire propagation throughout the city, a catastrophe that should be prevented at any cost. Furthermore, the flooded soils where the city was being constructed, represented a major threat to all citizens. The flat and low altitude of Lisbon's downtown terrains (taking the level of sea waters as a reference) facilitate the sea entrance in the city and possibly, its destruction in case of a tsunami occurrence. To sum up, several weaknesses could be pointed out to the Lisbon layout and their constructions before the year of 1755. (Nunes, R. D. D. S. F., 2017)

2.3.2 Pombalino buildings

The *Pombalino* construction represents the first time in history that a city was entirely built making use of solutions designed to withstand future earthquakes. The new downtown design placed the buildings in rectangular quarters with similar dimension following an orthogonal grid of streets. According to Mascarenhas (2005), the structural regularity of the buildings provided a similar performance of the construction within the compound, which besides reinforcing the group effect also gave them superior structural stability. The seismic resistance of a structure depends on the elements capacity of transferring the inertia forces, imposed by the dynamic actions, directly to the foundation system without collapse of the building (Lopes, M., 2008). The operation of the structure altogether is essential for the good seismic performance of the buildings and to prevent the overturning of the facade walls, which is the most common collapse mechanism of old masonry buildings. This *Pombalino* typology of buildings can be identified by timber reinforcement of the masonry. Vertical and horizontal timber elements were added to the facade walls, stiffening the masonry structure around the window openings. The interior structure was composed by timber-masonry walls, timber floors and roof, linked to the exterior walls by timber connecters partially embedded on the masonry and reinforced by metal straps. The wood structure results on the buildings strength and energy dissipation capacity, essential to support the seismic actions in any direction (Lopes, M., 2010). The existence of the three-dimensional timber structure is named 'gaiola pombalina'. The principal structural material is not only masonry but also wood elements that exist wrapped up in it like a cage made of vertical and horizontal elements braced with diagonals, enclosed on the walls above the first storey. These diagonal elements form Saint Andrew's Crosses, which allow forces redistribution from horizontal actions. The wooden cage is the main earthquake resisting system, eventually leaving masonry to a secondary role. It is known that the mass of a building plays an important role regarding the seismic effects, thus, a timber structure would drastically reduce the weight of the building which combined with the cross timber members conferred an increased resistance that could not possibly be achieved with a simple masonry wall (Nunes, R. D. D. S. F., 2017). It is usual to find several timber species at the same building, what indicates that elements were recycled from ruined constructions during 1755 Lisbon's earthquake, and placed with no care, aiming to speed up the town re-construction process. From the strength point of view, the influence of the wood structure must be the major point due to the improvement that it may represent. (Cardoso, R., Lopes, M., & Bento, R., 2005) There is no record at all when this improved solution started to be implemented, as well as in all the urbanistic laws published at the time, none of them made any reference to this brilliant invention. However, and possibly due to the complexity of the system to be solely by one person, it is common to attribute the implementation of this solution to *Casa do Risco* and their engineers at the time. Other documents refer to Carlos Mardel as the inventor, mainly due to his experimental tests done in *Terreiro do Paço* to a similar structure there constructed and tested by a military battalion (França, J.-A., 1989).

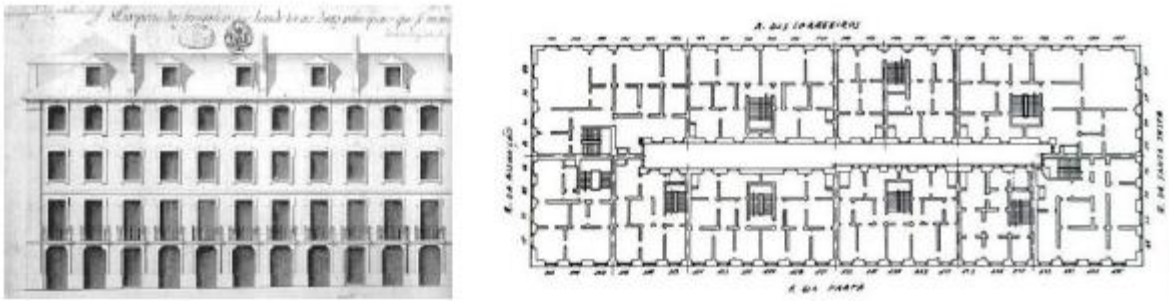

Figure 5: Pombalino structure (Cardoso, R., Lopes, M., & Bento, R., 2005)

Figure 6: Gaiola pombalina wall (Cardoso, R., Lopes, M., & Bento, R., 2005)

2.3.3 Gaioleiro buildings

During the first half of the nineteenth century there were few changes on the urban landscape as the city continued to grow accordingly with the *Pombalino* reconstruction plan. In 1864, a commission was nominated by the Ministry of Public Works to deal with a program of urban improvements and expansion of the city to the north upland. In 1888, the engineer Ressano Garcia developed a new plan regarding the connection between Liberdade Avenue and Campo Grande. The *Gaioleiro* buildings were aggregated in quarters with interior yards and surrounded by a grid of secondary streets, wider than the streets of the *Pombalino* downtown. There were no standards for buildings height or depth, neither for the architectural design of the facade walls. The construction was carried out by private entities, and therefore the quality of the buildings is very variable. This typology of buildings is related with the buildings built to be sold or to be rented by flats aiming to sustain the development of the city and the housing needs of an increasing population (Frazão, T., 2013)

Figure 7: Gaioleiro structure Cardoso, R., Lopes, M., & Bento, R., 2005)

During the nineteenth century, the cage structure characteristic of the *Pombalino* buildings was progressively simplified. The diagonal elements started to be removed, conditioning the bracing of the timber structure and the rubble infill was then replaced by brick masonry, solid on the lower floors and hollow on the upper, or by '*tabique*' walls, originally used on *Pombalino* buildings as partition walls. A *tabique* building component as a wall is formed of a timber structure made up of vertical boards or studs connected by laths trough metal nails. This structure in then coated with an earth based material (Cardoso, R., Pinto, J., Paiva, A., & Lanzinha, J. C., 2015).

2.3.4 Placa buildings

In 1938, a new urbanization plan was commissioned by engineer Duarte Pacheco. The first buildings were built with exterior masonry walls and timber floors strengthened by peripheral concrete beams.

The buildings from these new neighbourhoods have a characteristic shape in plan known as '*Rabo de Bacalhau'* originated by the expansion of the lateral light-shafts characteristic of the *Gaioleiro* buildings into the back yard of the quarter. The concrete slabs were extended to the whole floor, supporting the name '*Placa*' (meaning concrete slab) given to this typology of buildings (Simões, A., Lopes, M., Bento, R., & Gago, A., 2012).

3. Main earth retaining systems

3.1 Temporary King Post walls: Berliner walls

3.1.1 Introduction

Urban construction often involves the execution of underground floors. Due to often existing surrounding construction, vertical excavations have to be executed, supported by a retaining wall. In such context, Berlin-type walls are one of the most suitable techniques to the execution of retaining walls. This technique takes advantage of the construction phasing in order to minimize walls and back soil displacements. (Rodrigues, J. N. S. S., 2011) The Berliner wall is a temporary earth retention system with two basic components. One component, carrying the full earth pressure load and moment resistance, is the vertical so-called soldier pile. These are most of the time steel beams, I or H section profiles. The piles support the lagging wall or horizontal sheeting that spans the distance between the vertical elements. Lagging can be made of wood, steel sections or (reinforced) precast concrete.

3.1.2 Construction sequence

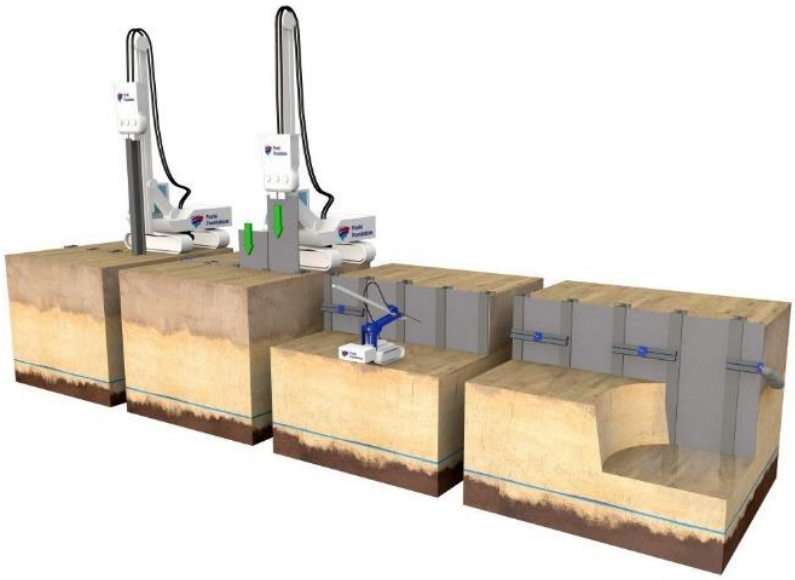

Figure 8: Berliner walls construction sequence (http://www.ffgb.be/Business-Units/Retaining-Walls---Utilities/Berlijnse-wand-en-paroi-parisienne.aspx?lang=en-US)

First of all the steel sections are positioned verticality and can be driven, vibrated or drilled into the ground. Steel sections are in most cases H type, so the flanges of the profiles have to be placed parallel to the longitudinal axis of the planned excavation (Brito, 2001). Alternatively, a continuous flight auger, large diameter or minipiling rig is used to create a bore hole which is filled with concrete to form a base for a H pile. The profiles must be placed at least one meter below the maximum excavation level to transfer the soil pressure from the lagging to the piles. The beams should be placed at regular spacing, the distance depends on the strength of the used laggings. When these soldiers are in place, excavation proceeds in stages while installing the lagging. The excavation can be one to a few meters deep,

depending on the stand-up time of the soil. This process is repeated level after level until the desired depth is achieved (Rodrigues, J. N. S. S., 2011). In general, the width of the panels is a little bit shorter than the distance between two consecutive steel sections, so they can slide between the steel piles and be clamped between the flanges and the ground with the aid of wedges.

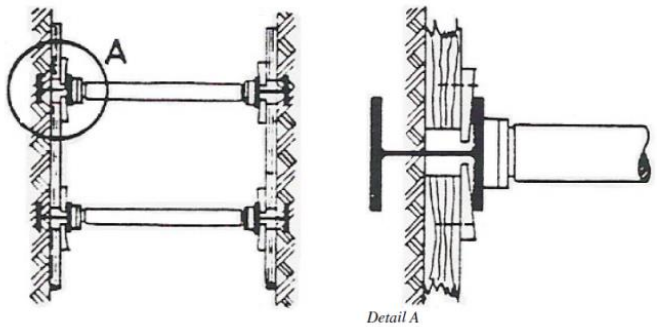

Figure 9: Wedges (Wylaers M., 2016, Bouwtechniek 1 – Berlin walls)

Another method is the use of a hook, attached to the outer flange of the piles. The hook is placed between the horizontal sheeting and a U-shaped iron is then placed over the hook to cover at least 3/4 of the width of the planks. The system then gets clamped by a steel wedge which fits through the opening of the hook. The joint between 2 series can be executed as shown in Figure 11.

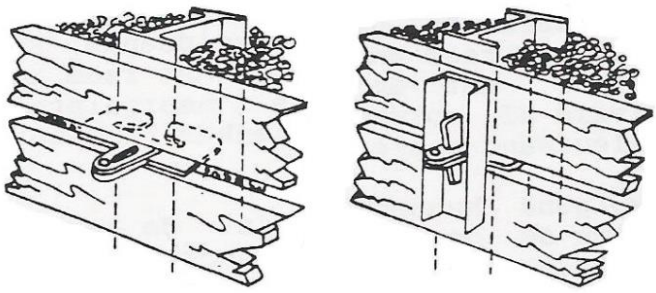

Figure 10: U-shaped iron and hook (Wylaers M., 2016, Bouwtechniek 1 – Berlin walls)

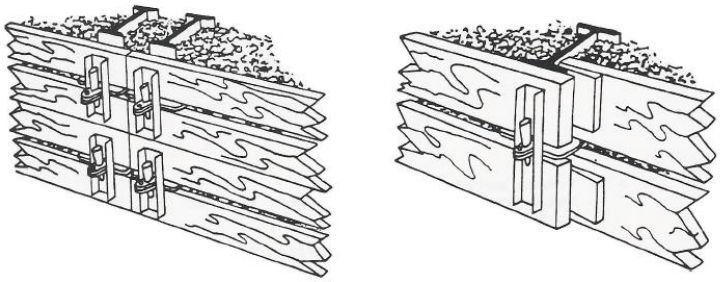

Figure 11: Joint between lagging (Wylaers M., 2016, Bouwtechniek 1 – Berlin walls)

Optionally, the Berliner wall can be stamped or anchored with ground anchors, immediately installed on the soldiers previously built.

3.1.3 Advantages and disadvantages

The main advantage of Berliner walls is their versatility. Adjustments can be made in the field easily to accommodate changes. The other major advantages of soldier pile walls are:

- ✓ Soldier piles are fast to construct;
- ✓ Berliner-type walls are cheaper when compared to other systems;
- ✓ Lagging construction can be very quick;
- ✓ The materials are reusable;
- ✓ Suitable for use near cables and pipes;
- ✓ Construction of soldier pile and lagging walls does not require very advanced construction techniques.

The major disadvantages of Berliner walls are:

- They are primarily limited to temporary construction;
- Cannot be used in high water table conditions without extensive dewatering;
- Poor backfilling and associated ground losses can result in significant surface settlements;
- They are not as stiff as other retaining systems;
- Because only the flange of a soldier pile is embedded beneath subgrade, it is very difficult to control basal soil movements.

3.2 Permanent King Post walls: Munich walls

3.2.1 Introduction

When King Post walls are structures which consist of metal profiles with between them, profiles of wood or precast concrete panels (Patrício, A., & Teixeira, R., 2006), these types of retaining walls are temporary and they are called Berliner walls. Although, when the execution of the walls is a permanent solution that uses reinforced concrete poured in site, supported by micropiles staled at the ground vertically, this technique is called Munich walls. These walls can be used as the final wall of an underground floor. The name of these techniques has already led to some controversy, because they are similar but definitely not the same. Depending on the height of the containment structure, the function to which it is intended or the type of soil to retain, it may be necessary to perform one or more levels of ground anchors (Martinho, F. C., 2013). The use of Munich-type walls is a solution widely used nowadays in buildings where is expected to maintain its facade, since it presents several advantages over other solutions (Cravinho, A., Brito, J., Branco, F., Vaz Paulo, P., & Correia, J. , n.d.).

3.2.2 Construction sequence

For the execution of Munich-type walls, first a general excavation has to be executed, just up to the bottom of the crown beam. This should be as low as possible, dependent on the conditions of the project. Then, the micropiles can be installed vertically and the crown beam can be made. The crown beam

makes sure that the remaining loads of the building can be transferred to the micropiles. Therefore, the crown beam connects the micropiles to the remaining structure of the building.

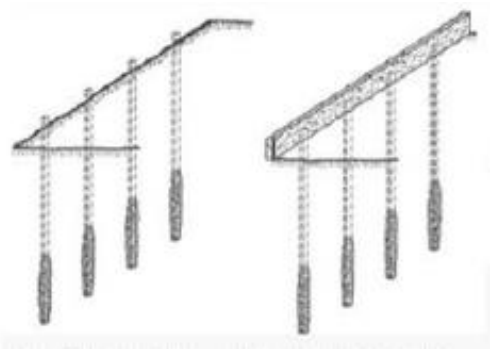

Figure 12: Micropiles and crown beam (Pinto A., 2010, Retaining walls and structures. Discipline of Excavations and Underground Works. Instituto Superior Técnico)

Just like the execution of the Berliner wall, the execution of the Munich-type wall is done in vertical stages. But here the use of horizontal staging is also important, because of the 'Soil-arching effect' discussed in 3.2.3. Horizontal stages usually have a width of 1m to 1,5m. In this alternate panel method, primary panels shall be cast first, leaving suitable gaps in between. These gaps are excavations made in a slope for an optimal soil-arching effect. Secondary panels shall then be cast, resulting in a continuous Munich wall. Each stage consists of the placement of the reinforcement and formwork, followed by pouring the concrete. The crown beam has the objective of joining all the profiles so they can work together.

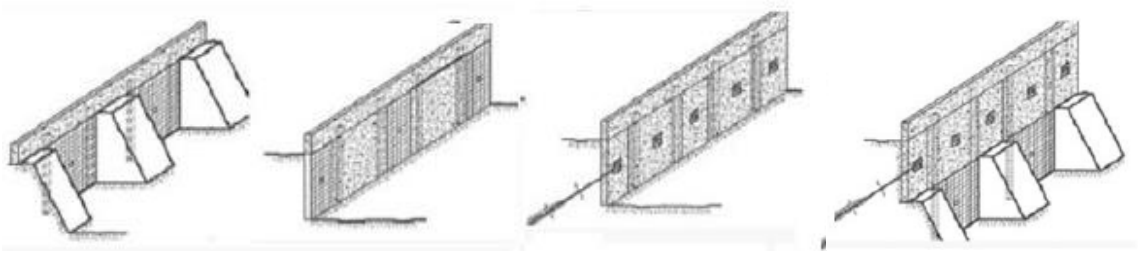

Figure 13: Munich walls construction sequence (Pinto A., 2010, Retaining walls and structures. Discipline of Excavations and Underground Works. Instituto Superior Técnico)

Afterwards, anchorages and shores are placed if required. Finally, the foundation of the Munich walls is executed and the new structure is responsible for the stability of the facade walls.

3.2.3 Soil-arching effect

It has been well recognized for a long time that one of major mechanisms for stabilizing soil is the soil-arching effect, which is a phenomenon of transfer of stresses from a yielding mass of soil onto the adjoining stationary part of soil. This effect occurs when there is a difference in the stiffness between the installed structure and the surrounding soil. From studies of Vanel and Howell (1999) it is stated that changes of soil strength as well as elasticity modules have an effect in the formation mechanism of the arch. A simple example of arching is what occurs in a large box of soil with a panel at the base. When

this panel is lowered, the soil immediately above it will tend to move down with it. If the yielding part moves downward, the shear resistance will act upward and reduce the stress at the base of the yielding mass. On the contrary, if the yielding part moves upward, the shear resistance will act downward to impede its movement and cause increase of stress at the support of the yielding part. However, if the shear strength of the soil is sufficiently large, what will happen instead is that the weight of the column of soil immediately above the moving panel will be partially transferred to the surrounding soil. Naturally, the vertical stresses in the soil around the moving panel will increase, while those immediately above it will decrease. (Terzaghi, 1943)

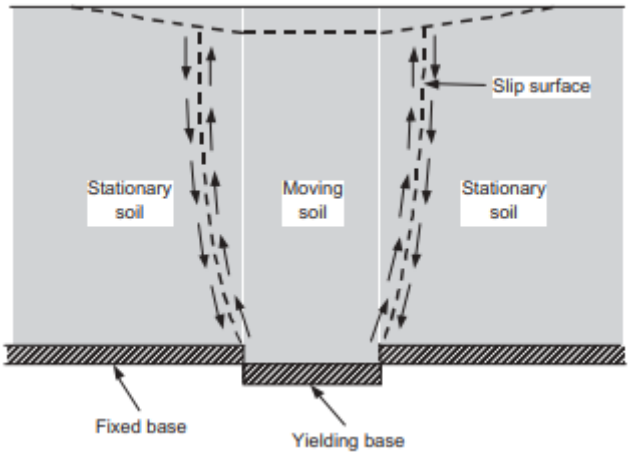

Figure 14: Soil-arching effect (https://ascelibrary.org/doi/abs/10.1061/%28ASCE%291532-3641%282008%298%3A6%28347%29)

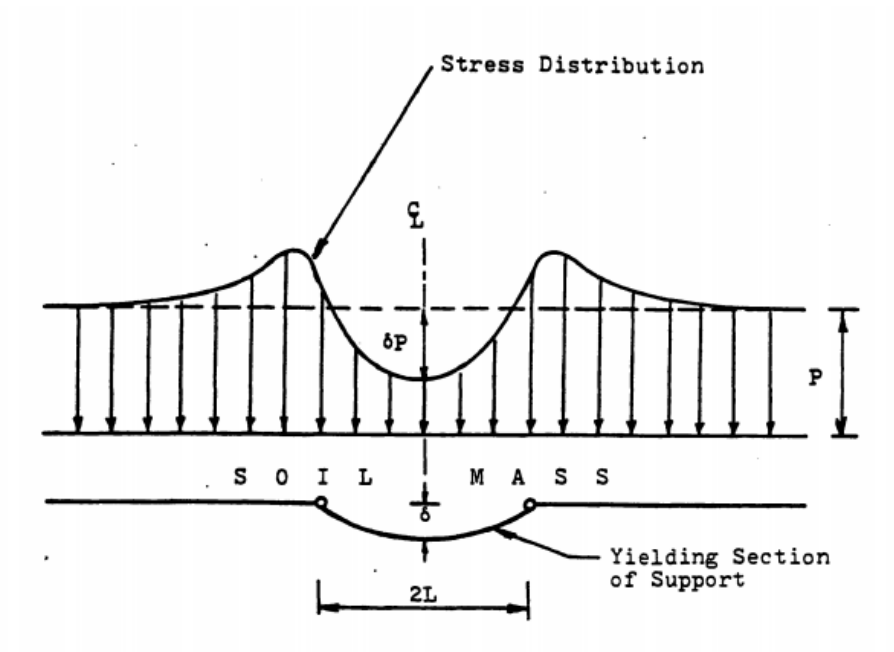

Figure 15: Stress distribution in the soil above a yielding base (Bjerrum et. al., 1972)

Underground openings can be built utilizing the arching action to account for the reduction in the overburden pressure. The soil medium adjacent to the underground opening can increase the structure's

load-carrying ability compared to an identical unburied structure. The Munich walls are excavated in a primary and secondary phase to get a significant reduction of the stresses in the excavated zone and an increase of the stress in the laterally adjacent soil. The gaps between the primary panels are executed as a slope to support the increase of stress resulting from the decompression of the affected ground, taking advantage of the soil-arching effect. In the upper zone of the primary panels the soil tends to distribute its tensions horizontally, increasing the soil stresses laterally. The soil located in the bottom zone transfers most of its load in the vertical direction, to the soil at the base of the excavation, distributing less in the horizontal direction.

3.2.4 Advantages and disadvantages

The main advantage of Munich walls are:

- ✓ It's one of the cheapest permanent containment systems;
- ✓ The technique requires no large work area, no specialized employees or technology;
- ✓ It saves a lot of space because the walls are executed against the soil.

The major disadvantages of Munich walls are:

- It's a slow process because of the horizontal and vertical staging;
- Cannot be used in high water table conditions without extensive dewatering;
- Require soils with some consistency;

3.3 Ground anchors

3.3.1 Introduction

The idea of installing retaining systems without ground anchoring is almost inconceivable nowadays. Excavation pits with no obstructive strutting have been the standard ever since the ground anchor for loose soils was invented in 1958 by Karl Heinz Bauer. Today ground anchors are used to secure pile walls, sheet piles, Mixed-in-Place walls or slurry walls, as well as steep slopes, support embankments and quay walls. Grouted anchors consist of the three main parts: steel tendon, anchor head and grout body. The steel tendon is flexible between the front edge of the grout body and the anchor head and is called free steel length. It acts like a spring with which the part to be tied is prestressed against the construction soil. The load is only transferred into the subsoil in the area of the grout body, due to the bracing of the grout in the ground. When anchorages are installed, the phenomenon of the arching-soil effect occurs both vertically and horizontally. The effect reduces pressures in the deformable area (anchors) and concentrates them in the area around them.

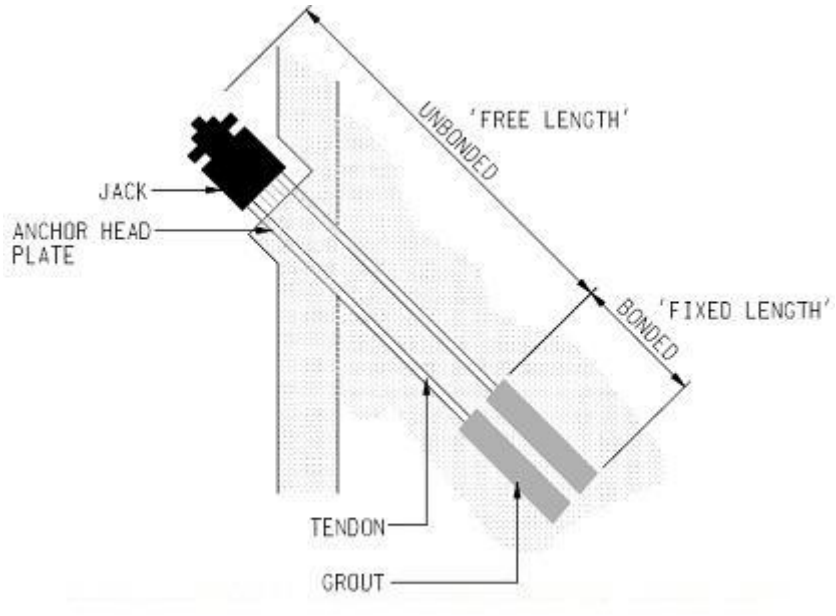

Figure 16: Ground anchor (https://www.engineeringcivil.com/what-is-the-significance-of-free-length-and-fixed-length-in-tiebacks-in-anchored-excavation.html)

The anchors can be executed as temporary or permanent measures. Short-term anchors may usually only be in use for a maximum of two years, while permanent anchors have a useful life of at least 100 years. Therefore, they must have appropriate corrosion protection, as spoken of in part 3.3.3. Depending on the base element there's a difference between single bar, multiple bar and stranded anchor.

3.3.2 Construction sequence

The anchors are usually installed at an angle of 15 to 45 degrees. First of all, a borehole is drilled using a drilling method relevant for the existing site conditions. Various drilling methods are available to install anchors, depending on the existing soil, ground water and post treatment. Especially in cohesive soil, the selection of the best method is decisive for gaining the necessary anchor bearing capacity. These are the most common methods: rotary drilling or rotary percussion with single rod, overburden drilling, double head drilling, auger drilling and driving. The drill rods can be withdrawn after or during the filling/grouting with cement mortar. The anchor is installed and the grout is pumped under pressure into the ground anchor holes to increase soil resistance and thereby prevent ground anchors from pulling out, reducing the risk for wall destabilization. If necessary there can be multiple post grouting.

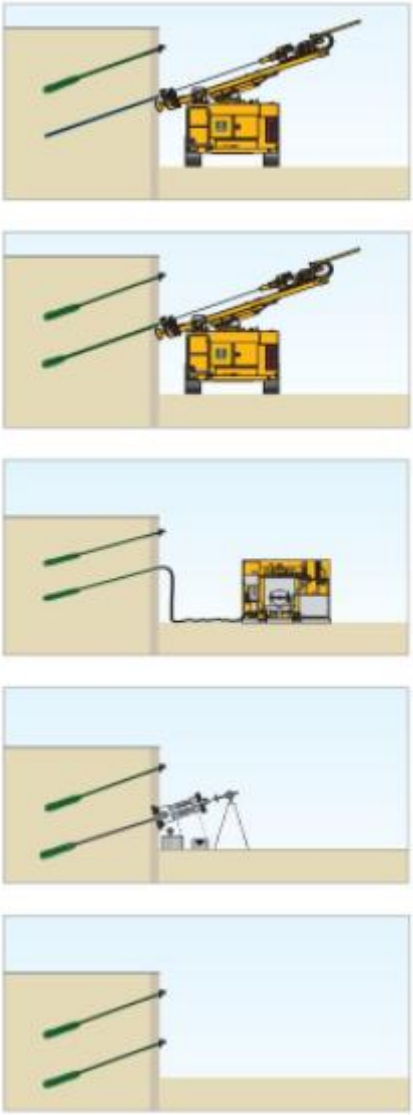

Figure 17: Ground anchors construction sequence
(http://www.bauerfoundations.com/en/competences/anker.html)

Permanent anchors always need an acceptance test after hardening of the grout body. This is a local load test at each anchor to check the adherence of the design criteria.

3.3.3 Temporary or permanent ground anchors

As said in the introduction, temporary anchors have a planned service life of up to two years. They can be installed as single rod, multiple rod or wire strands. If required they can be removed partly or completely. Sometimes, temporary anchors also have to undergo a local load test (acceptance test) to prove that they adhere with the design criteria. Permanent anchors are designed for a service life of more than two years. The main difference between the permanent anchor and the temporary one is the additional corrosion protection. The anchors have a complete and permanent corrosion protection and are fabricated ready to be installed. The tendon rod has double corrosion protection provided first by a thin hard coating of corrosion resistant plastic throughout its length and second by concrete grout over

the anchor zone of the rod and by a heat shrunk plastic tube tightly encapsulating the unbonded zone of the rod (Weatherby, D. E., 1978).

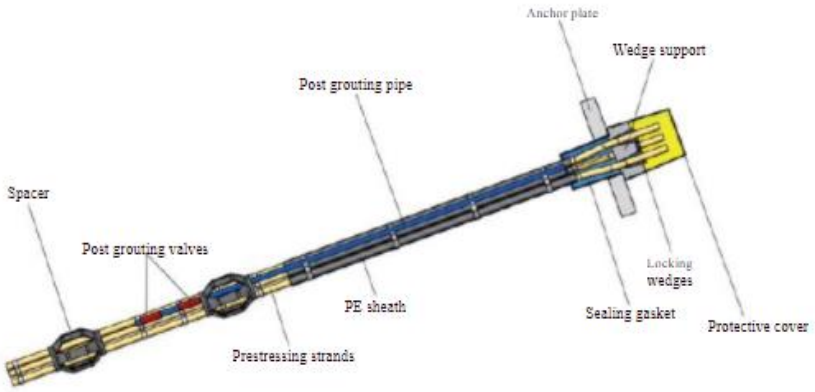

Figure 18: Temporary ground anchor (http://www.bauerfoundations.com/en/competences/anker.html)

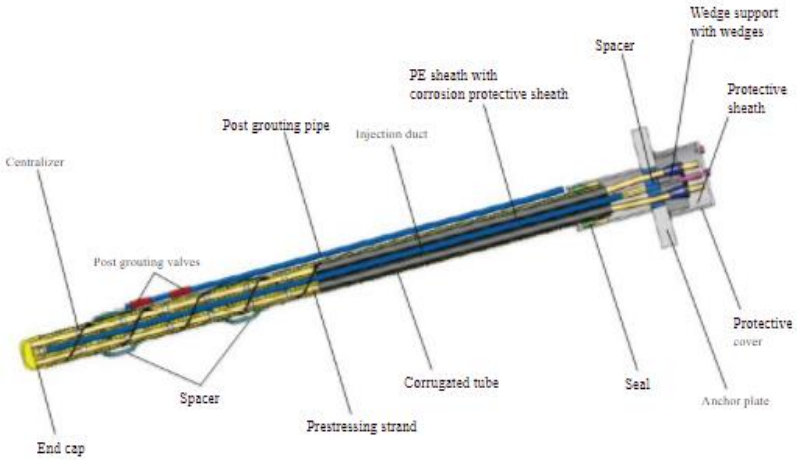

Figure 19: Permanent ground anchor (http://www.bauerfoundations.com/en/competences/anker.html)

3.3.4 Advantages and disadvantages

The major advantage of ground anchors is the ability to support a temporary construction excavation without the need for cumbersome bracing that obstructs workspace. Ground anchors are cheaper than conventional bracing in cuts of more than 4,6m to 6,1m and/or widths of greater than 18,3m, and construction is not impeded by cross-bracing. Disadvantages of ground anchors for tiedowns include potentially large variations in ground anchor load resulting from settlement and heave of the structure. The difficulty in constructing watertight connections at the anchor-structural slab interface is particularly important for hydrostatic applications.

3.4 Underpinning

3.4.1 Introduction

Underpinning is the installation of temporary or permanent support to an existing foundation to provide either additional depth or an increase in bearing capacity. There are several existing conditions which may lead to the need for underpinning (Nemati, K. M., 2005).

- Construction of a new project with a deeper foundation adjacent to an existing building;
- Settlement of an existing structure;
- Change in use of a structure;
- Addition of a basement below an existing structure.

3.4.2 Preparation

A key aspect to consider is how much disturbance to the building the underpinning method will cause. Studies and calculations must be done by qualified structural engineers, as the operation can cause serious structural damage or total collapse if not done correctly. Information about the following subjects is very important (Chudley, R., & Greeno, R., 2006).

- Total length of wall to be underpinned;
- Width of existing foundation;
- General condition of existing substructure;
- Superimposed loading of existing foundation;
- Estimated spanning ability of existing foundation;
- Subsoil conditions encountered.

The risks associated with excavations are so great that general shoring can never be regarded as an unnecessary luxury because there are always unforeseen circumstances. The shores must be chosen in function of the expected movements. Horizontal shores at high altitudes can counteract the general rotation of adjacent structures. Oblique shores on the bottom of the wall can be useful to prevent subsidence. When the separation walls are no longer anchored to neighbouring buildings, they could suddenly start to move and simply fall over. Therefore, these walls must be anchored to a neighbouring structure.

3.4.3 Main risks

The damage that occurs during replacement works is generally due to one of the following causes.

3.4.3.1 *A change in loads and tensions*

When the excavation is completed, the load of the separating wall is transferred to a deeper layer of soil. This allows the soil layers to further compress, with a settlement of the wall as a result. Consideration should also be given to the shrinkage of the materials used. The connection between the old and new foundation plays the most important role in this problem.

3.4.3.2 *Movements during the execution of work*

Precautionary measures have to be taken to prevent the movement of the ground on the other side of the wall, especially when working in loose sandy soil which is often the case along the coast. The possibility that the sand mass suddenly starts to move and slips into the construction pit has to be taken into account. A small shift is enough to cause serious damage to the floors in the neighbouring building. The risk of collapse persists when the barrier wall has already been partially completed. At that time, ground pressures are exerted on the inscribed wall.

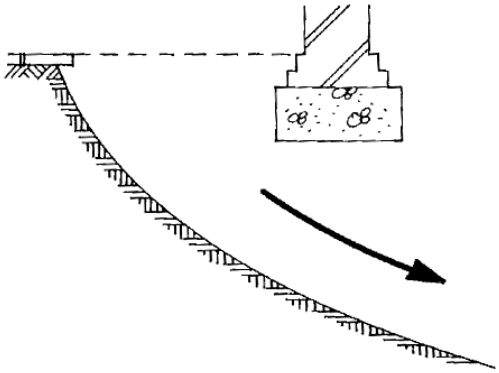

Figure 20: Soil movements during the execution of the work (Wylaers M., 2016, Bouwtechniek 1 – Underpinning)

3.4.3.3 *Hollow spaces in the ground*

There is a considerable chance that, due to underpinning, hollow spaces are created in the ground. The most dangerous zone is located behind the underpinning. Zone 2 in the figure must be supplemented very thoroughly during the execution of the work, for example with stabilized sand. The problem of this hollow space is much smaller when the overlap is carried out with concrete poured in situ. In such a case, the liquid concrete will automatically fill all voids.

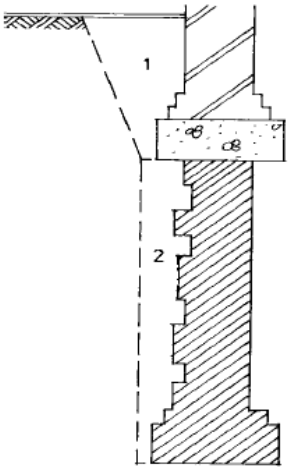

Figure 21: Hollow spaces (Wylaers M., 2016, Bouwtechniek 1 – Underpinning)

3.4.3.4 *Time factor*

In order to limit the size of the excavation, one generally tries to excavate the ground vertically under the wall to be covered. The result, however, is highly dependent on the cohesion of the soil. Since this

decreases over time, there is a risk of dehydration and settling of the soil with a landslide as a result. All this has to be prevented by a quick execution of the work. Therefore, materials will be used that can be processed quickly so that the slots must only be kept open for a short time. Even if the wells are properly covered over the entire circumference, then the duration of the work must be limited as much as possible.

3.4.4 Methods

Nowadays, numerous underpinning methods are available to provide safe, fast and practical solutions to nearly any geotechnical problem related to the foundations of a structure. (Kordahi, R. Z., 2004) The means and methods of supporting a structure foundation depends on many factors including:

- Foundation loads: static and dynamic, permanent and temporary;
- State of existing foundations;
- Type and magnitude of allowable structural movement i.e. deformations;
- Subsurface soil conditions;
- Subsurface groundwater conditions;
- Condition of the structure;
- Access and mobility to the foundations;
- Potential for environmental hazards;
- Seismic loading.

This list is by no means exhaustive and each of these factors must be considered in making the evaluation of which underpinning method can best satisfy the project needs. The development of new underpinning techniques has allowed the adoption of a wide number of solutions, progressively more adapted to the singularities and restraints of each scenario, especially when sensitive, old or historic, constructions founded on weak soils have to be underpinned. In this context, the solutions comprising micropiling and jet grouting techniques should be pointed out due to their versatility and advantages related to the limitation of vibrations, as well as the possibility to be adopted in small spaces with low head rooms and restricted access. These techniques also allow the soil improvement, minimising the soil disturbance due to the boreholes small diameter, drilled with suitable equipment (Bullivant, R. A., & Bradbury, H. W., 1996).

3.4.4.1 *Jet grouting*

Jet grouting technology has initially been developed in Japan, the UK and Italy. In Portugal the technology has been applied in the last 6 years, mainly on Lisbon Metro extension works. (Falcão et al., 2000; Greenwood, 1987). Jet grouting or high pressure grouting is a load transferring system for underpinning. It's an erosion-based grouting technique using a high-pressure jet of grout to break up the soil structure and simultaneously mix the loosened soil with a cement grout to form so-called '*soilcrete*' (grouted soil - cement soil) column-shaped bodies, panels or half-columns. The distinct advantage of jet grouting is to significantly increase the strength of the soil treated and / or reduce its permeability. This technology requires specialized equipment and experience to construct the *soilcrete*. Work is accomplished safely above grade, and sequenced so that little or no structural deformation occurs.

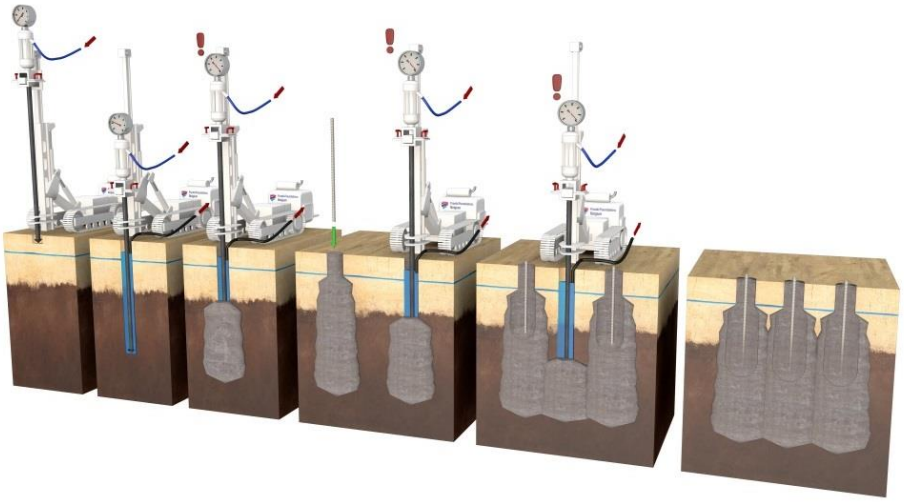

Figure 22: Jet grouting (http://www.ffgb.be/Business-Units/Bored---Micro-Piles/Jet-grouting.aspx)

The process starts with the drilling phase, where a jet grouting drill sting with a special drill bit at its bottom is used to drill a small hole to the design depth of the treatment through injection of water or cement grout. In the jetting phase, cement grout is pumped at very high pressure (400 bar) above the bottom of the drill tube where it emerges through very small diameter orifices or injection 'nozzles' (diameters from 1,5 to 4 mm) into the soil, converting the energy from high pressure to very high velocity and disintegrating the soil structure over / across a specific distance. The jet erodes the soil as the drilling rod and drill bit are kept rotating and slowly pulled up at a controlled rate. A homogeneous mixture of injected cement grout and soil formes a cylindrical '*soilcrete*' column in the soil. Upon reaching the desired column height, jetting is stopped and the tube is withdrawn. A central reinforcement bar, a reinforcing cage of limited dimensions or a steel profile can be inserted into the freshly formed grout column.

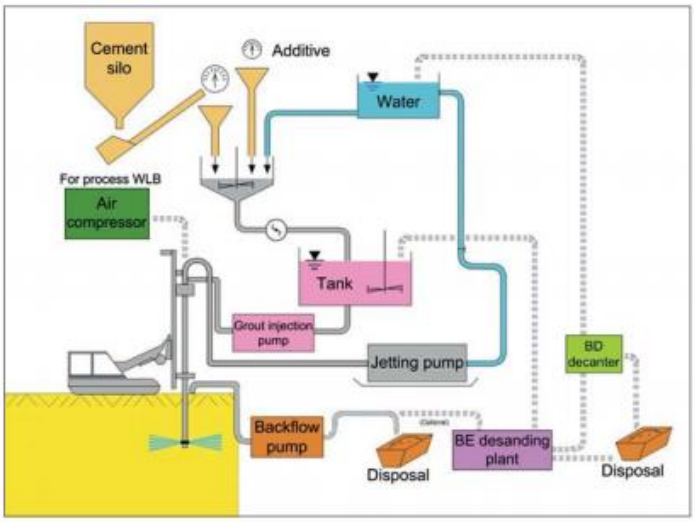

Figure 23: Site installation jet grouting (http://www.bauer.de/export/shared/documents/pdf/bma/datenblatter/HDI_Bauer_JetGrouting_EN_905.760.2.pdf)

Depending on the prevailing soil conditions, different jet grouting methods are employed.

- 1-Phase System: Binder cutting, used in granular soils for small to medium column diameters;
- 2-Phase System (suspension and air): Binder cutting with air shrouding, used in granular soils for medium to large column diameters;
- 2-Phase System (water and suspension): Water cutting and filling the soil with binder, used in cohesive soils for small to medium column diameters;
- 3-Phase System: Water cutting with air shrouding and filling the soil with binder, used in cohesive soils for medium to large column diameters.

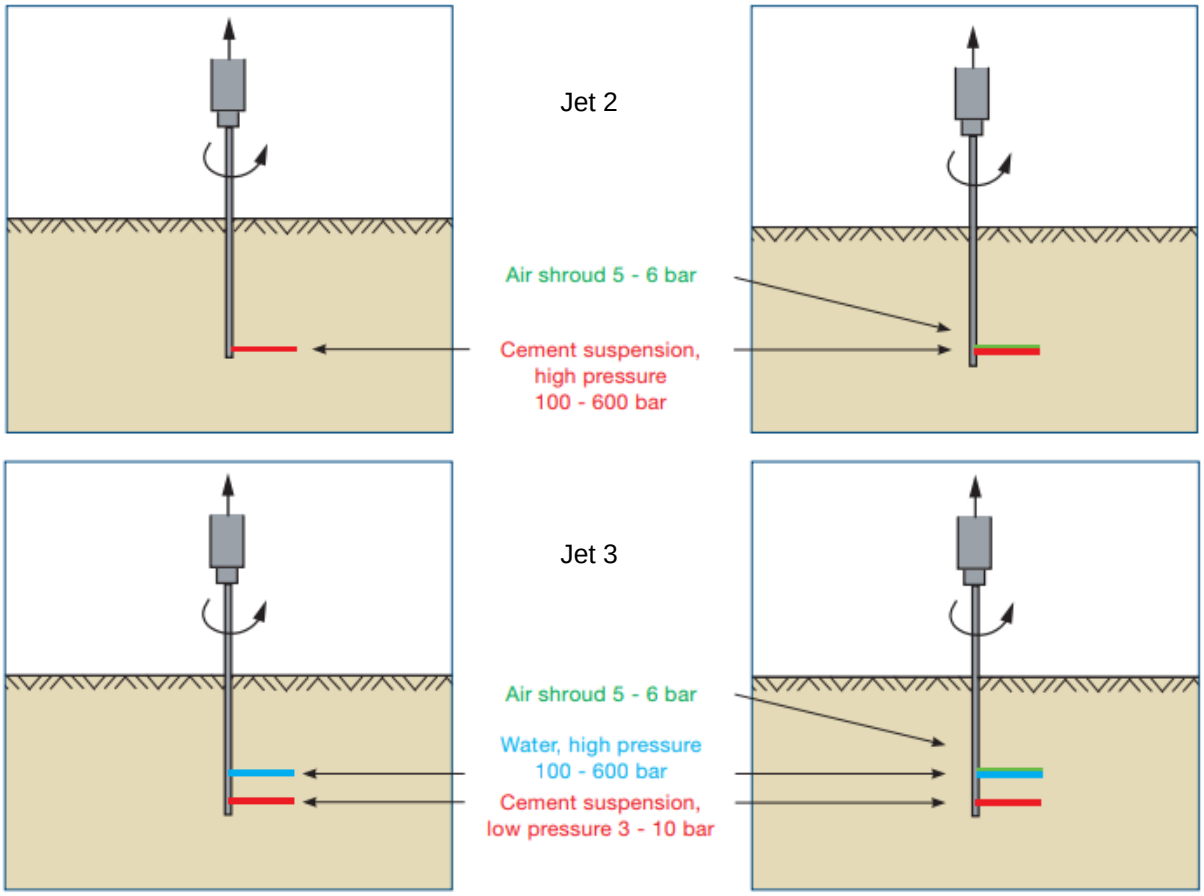

Figure 24: Different jet grouting methods (http://www.bauer.de/export/shared/documents/pdf/bma/datenblatter/HDI_Bauer_JetGrouting_EN_905.760.2.pdf)

3.4.4.2 *Micropiles*

Micropiles were developed in Italy in the early 1950's in response to the demand for innovative techniques for underpinning historic buildings and monuments that have endured damage with time. The micropile, also known as root pile, is a small diameter cast-in-situ bored pile formed by cement grout injection and subsequently equipped with lost, steel reinforcement element (bar, steel tube, H-type profile,…). The boring is performed with compressed air or by means of a supporting drilling fluid (water, cement grout, bentonite). Some of the advantages of micropiles are high carrying capacity, less site constraint problems, low noise and vibration and self-sustained operation. Furthermore, a major advantage when using micropiles for underpinning is that the system can be designed to have very low settlements. It is common for these piles to develop settlements on the order of a few millimeters or less under working loads. Under these conditions, its bearing capacity is not fully mobilized (Ellis, 1985). This

piling system is therefore attractive to both the client and the foundation designer. The only disadvantage of micropiles is the relatively higher cost as compared to other piling systems.

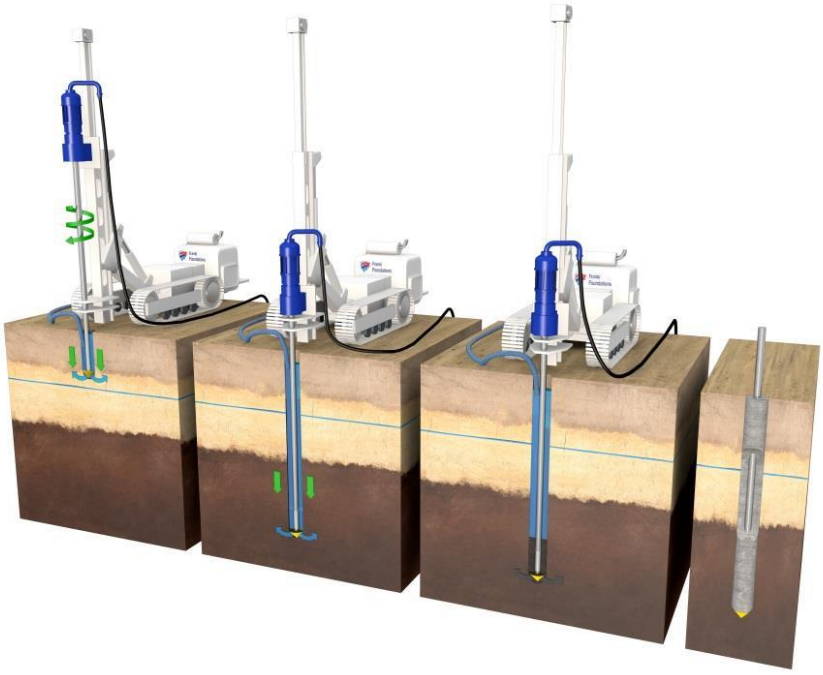

Figure 25: Micropiles (http://www.ffgb.be/Business-Units/Bored---Micro-Piles/Micropaal.aspx?lang=en-US)

The first section of the steel drill casing is placed, equipped with the drill bit fixed on a rod. After this installation, the boring process can start and fluid is flushed inside the drill casing. The boring is continued until the required depth has been reached, while the additional casing segments are coupled. After reaching the design depth, the drilling fluid is replaced by the primary grout injection of cement grout under pressure. The micropile is formed by single stage grouting under the so-called “unitary and global” grouting under low pressure or I.G.U. mode (« Injection Globale et Unitaire ») or the micropile is formed by multi-step grouting under the so-called “repetitive and selective” grouting under high pressure or I.R.S. mode (“Injection Répétitive et Sélective”).

4. Case study: Jasmin Noir building

4.1 Introduction

This case study is about an excavation site with the preservation of the old facades. The ‘Jasmin Noir’ building was built more than hundred years ago and is not liveable anymore. The facades must be preserved as a request of the Lisbon Municipality and there will be an excavation because the basement will become a place for two parking lots. This construction site and the performed work will be studied during the spring semester, 11 site visits are made.

4.2 Contract

The most relevant aspects for the classification of the construction site are:

- Type: Demolition, facade containment, peripheral containment, excavation and reconstruction;
- Employee Entity: Private Work;
- Competition: Limited (by invitation);
- Use: Apartments (one for each floor);
- Deadline for completion: 24 months;
- Under Article 78 (1) of Decree-Law No 555/99 of 16 December, amended and republished by Decree-Law No 136/2014 of 9 September, it becomes public that the Lisbon City Council issued on 10/12/2016 Licensing permit for works of alteration No 80 / 0D-CML / 2016. Building described in the Building Registry of Lisbon, under number 401, and inscribed in the matrix under article 638, of the parish of incarnation (extinct).

The organization of companies in the rehabilitation work of the ‘Jasmin Noir’ building is as follows:

- *Attitude* - General Contractor
  Engº Ricardo Sério - Project Manager
- *FCM* - Subcontractor that will carry out the work
  Engª Margarida Cardoso - Project Manager
  Engª Liliana Mendes - Responsible for hygiene and safety in the construction site
  Mr. Agostinho Martins - Foreman

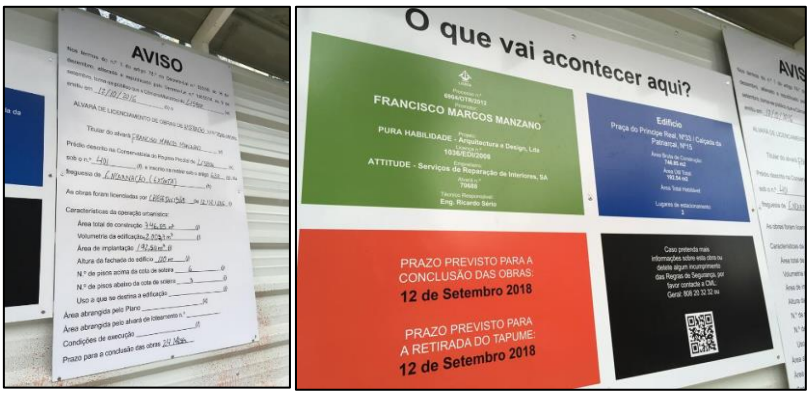

Figure 26: Construction site information

4.3 Location

The ‘Jasmin Noir’ building is located at the attractive neighbourhood of *Príncipe Real*, which extends north of *Bairro Alto*. It remains essentially a residential district, but it's now slowly becoming a serious shopping area. The building is situated in front of a tranquil park, the *Jardim do Príncipe Real*. From here you can see people grabbing a coffee from one of the Jardim’s two kiosks, or sitting down to lunch at the outdoor café under two massive mangrove-type trees. Although it’s almost invisible from the surrounding streets, you can find the enchanting botanical garden covering ten acres of subtropical vegetation, just 100m from the site. The ‘*Miradouro de São Pedro de Alcântara’* is only a five minutes’ walk, but from the balcony of the ‘Jasmin Noir’ building you can have your own viewpoint.

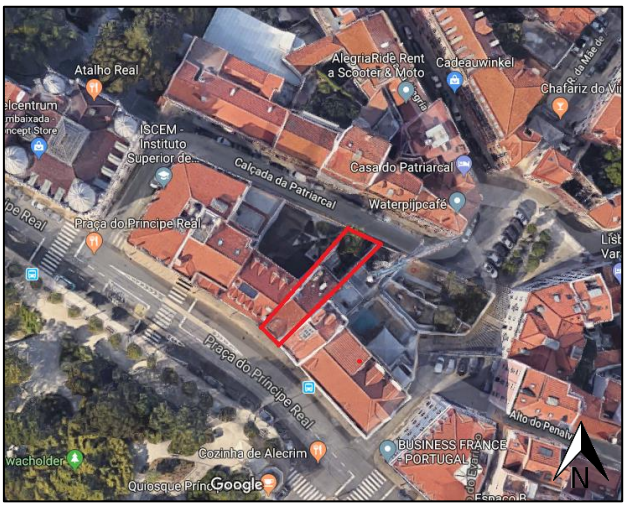

Figure 27: Jasmin Noir location (a) (Google maps)

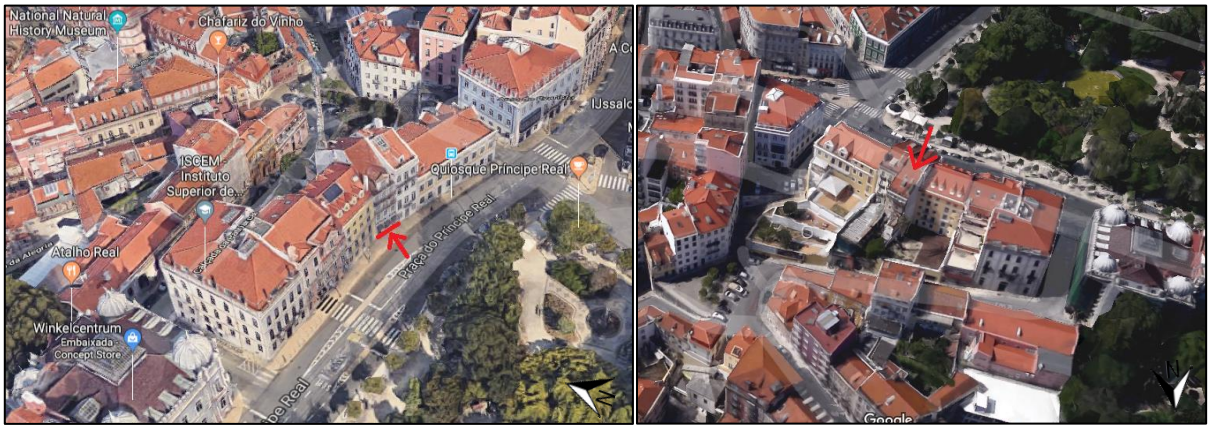

Figure 28: Jasmin Noir location (b) (Google maps)

4.4 Urban planning

Characteristics of the urban operation:

- Total construction area: 746.85 m²
- Building volumetry: 2003,7 m³
- Implantation area: 192,54m²
- Elevation of the facade of the building: 12m
- Number of floors above threshold: 6
- Number of floors below threshold: 3

4.5 The building's typology

The building is built after the 1755 Lisbon Earthquake, and has characteristics of both *Pombalino* as *Gaioleiro* typologies. *Gaioleiro* buildings were aggregated in quarters with interior yards and are characterised by a certain architectural freedom. This freedom can be seen in the varied window and ashlar designs, in the decoration of facades and in the introduction of new elements such as terraces and metal staircases at the rear of the buildings. We can see some of these elements in the Jasmin Noir building which indicates the building was built between 1870 and 1930. As picture 31 shows, the back facade of the building has a metallic structure of balconies, but we can't find the common characteristic of light shafts in the building. This could be explained by the small width of the building.

Figure 29: Metallic structure of balconies

The foundations of *Gaioleiro* buildings can be divided into two types: foundation footings and vaults or arches, while *Pombalino* foundations are generally composed of timber piles. (Mascarenhas, J., 2005). The Jasmin Noir building has the foundation footings in masonry, as shown in the inspection shafts in part 4.6.3. This indicates again that the building is built following the *Gaioleiro* typology.

All exterior walls and some of the interior walls in *Gaioleiro* buildings are made of stone or brick masonry. Usually the exterior walls are made of rubble stone masonry and the interior ones are made of brick

(Appleton, J. G., 2005). But in the Jasmin Noir building the interior walls are made of masonry reinforced with an internal wooden cage. This indicates the building is a response of urban reconstruction following the 1755 quake, based on a three-dimensional grid typically of *Pombalino* buildings. Horizontally, the structure of both *Pombalino* as *Gaioleiro* buildings consists of timber flooring where the beamwork runs perpendicular to the facades. The ceilings were finished by wooden boards covered with wooden strips at the floor beams, or with laths covered with plaster and interesting stucco details (Simões, A., Lopes, M., Bento, R., & Gago, A., 2012).

Figure 30: Internal wooden cage

These elements show that the Jasmin Noir building is an example of the *Gaioleiro* typology buildings. However, there was no construction obligation. It was noticeable that structures with the typical *Pombalino* timber reinforcements did not had significant damages after an earthquake and studies proved that this technique was one of the best structural anti-seismic solutions in that time. Landlords were free to apply the technique of '*gaiola pombalina*' to their buildings.

4.6 Seismic, geological-geotechnical and hydrogeological information

4.6.1 Seismic information

The distribution of seismic events allows the realization of zoning in the form of issosists, these are curves that delimit around an epicentre, areas where there were identical seismic intensities during the same earthquake. According to the chart of issossists of maximum intensity, of the Institute of Meteorology, the building is located in an area of intensity 9.

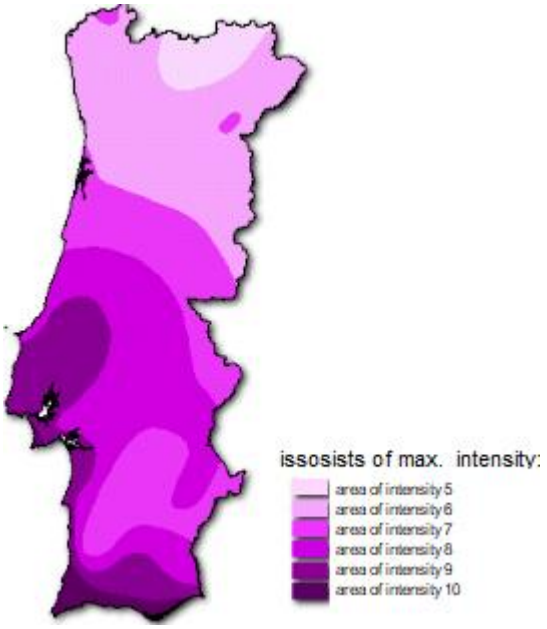

Figure 31: Chart of issosists of maximum intensity (TECNASOL FGE.,2017)

The seismicity occurring in Portugal is not uniform, generally increasing from north to south. The Eurocode 8 divided the continental territory into five types of foundation land, designated A, B, C, D and E. For the zoning of the territory, differentiation is foreseen depending on the nature and seismic intensity of each region. Thus, two types of seismic action were considered, due to the fact that there are two types of earthquake which affect Portugal:

- a scenario designated as "remote" referring generally to earthquakes with an epicentre in the Atlantic region and corresponding to Type 1 Seismic Action;
- a so-called "near" scenario, generally referring to earthquakes with an epicentre in the Continental Territory or in the Azores Archipelago, corresponding to Type 2 Seismic Action.

Seismic zoning for Continental Portugal is established by municipalities, as shown in Figure 32.

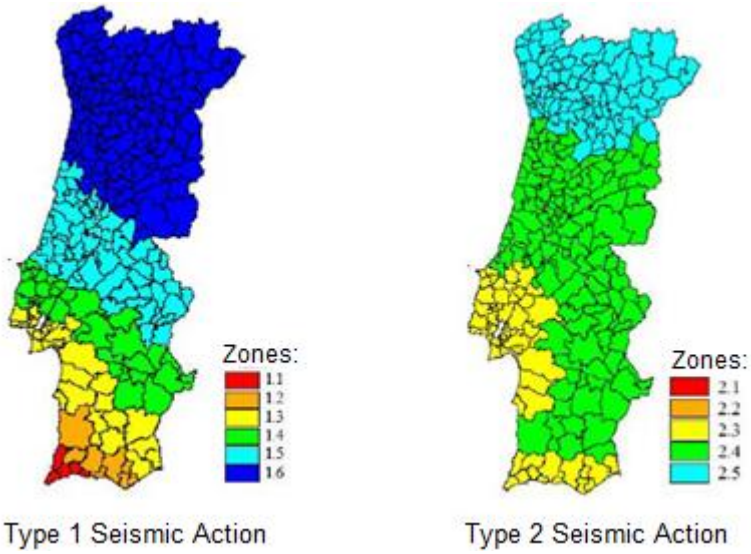

Figure 32: Seismic zoning of Portugal (TECNASOL FGE.,2017)

According to European NP EN 1998-1: 2010 (Eurocode 8), the following maximum reference seismic accelerations, a_{gR}, are considered, depending on the type of seismic action and the seismic zone of the site under study:

- Seismic action type 1 - Seismic zone 1.3: a_{gR} = 1.5 m/s^2
- Seismic action type 2 - Seismic zone 2.3: a_{gR} = 1.7 m/s^2

4.6.2 Geological information

The study site is located in *Praça do Príncipe Real, n ° 33*, extending to *n ° 15 of Calçada da Patriarcal*, in the parish of *Santo António*, an area where, according to the geological chart of the municipality of Lisbon there are lithologies belonging to *Areola da Estefânia* (M^1_{II}) characterized by clays with silt and silts with clay, with brown and gray tones, sometimes with carbonate intercalations. Covering part of the study area, there are brown clays with fragments of varied nature, dispersed.

Figure 33: Geological chart of the municipality of Lisbon (in 1:10 000 scale) (TECNASOL FGE.,2017)

4.6.3 Geotechnical information

ATTITUDE, S.A. requested Tecnasol FGE to do geotechnical tests of the building's area to recognize its characteristics. They did this using two drill holes that allowed to identify the soils that occurred, followed by dynamic SPT penetration tests (Standard Penetration Test). For the identification of the base and geometry of the lateral wall foundations in the backyard of the building, two inspection wells P1 and P2 were made.

Figure 34: Geological-geotechnical and hydrogeological tests location (TECNASOL FGE., 2017)

4.6.3.1 *Test boring*

As mentioned, the test boring consisted of two polling stations, S1 and S2, with depths of 15.0 m (S1) and 12.0 m (S2), in a total of 27.0 m of drilling. The drillings were carried out using a rotary probe driven by a hydraulic motor with continuous drilling. The drilling diameters used were 86mm, with the respective coating tubes 113mm and 98mm and the circulating fluid used was water. The following lithostratigraphic units were identified as indicated in Table 1. The individual survey charts are shown in the appendix number 1.

Table 1: Identified lithostratigrapigh units

Age	Formation	Lithology
Recent	Floors	Ceramic or limestone
	Landfill	Clay with silts, brown in color with fragments of varied nature, dispersed.
Miocene	Areolas da Estefânia (M^1_{II})	Clay with silts, with brown and gray colors, sometimes with small carbonate intercalations dispersed
		Silt with clays, with brown and gray colors

Figure 35: Test boring

Table 2 shows the geotechnical parameters found from the test boring. The soil is defined in geotechnical zones ZG1 to ZG4, visualised in appendix 2.

Table 2: Geotechnical parameters

Zone	Description	Specific weight γ (kN/m³)	Angle of internal friction Ø (°)	Cohesion C (kPa)	Stiffness E (MPa)
ZG4	Landfills	10-14	15-20	0	2,5*
					3,5**
ZG3	Clays with silt	19-21	30-33	22-50	16,5-18,7*
					23-26**
ZG2	Clays with silt, sometimes with marls	20-21	34-36	50-110	24-34*
					33-48**
ZG1	Clays with silt and silt with clay	21-22	35-40	100-150	45*
					60**

* Assymetric loading

** Flat deformation

4.6.3.2 *Standard Penetration Test (SPT)*

The Standard Penetration test is a common in situ testing method used to determine the geotechnical engineering properties of subsurface soils. SPT involves driving a standard thick-walled sample tube into the ground at the bottom of a borehole by blows from a slide hammer with a mass of 63.5 kg falling through a distance of 760 mm. The sample tube is driven 15 mm into the ground and then the number of blows needed for the tube to penetrate each 150mm up to a depth of 450mm is recorded. The sum of the number of blows required for the second and third 150mm of penetration is reported as SPT blow-count value, commonly termed "standard penetration resistance" or the "N-value".

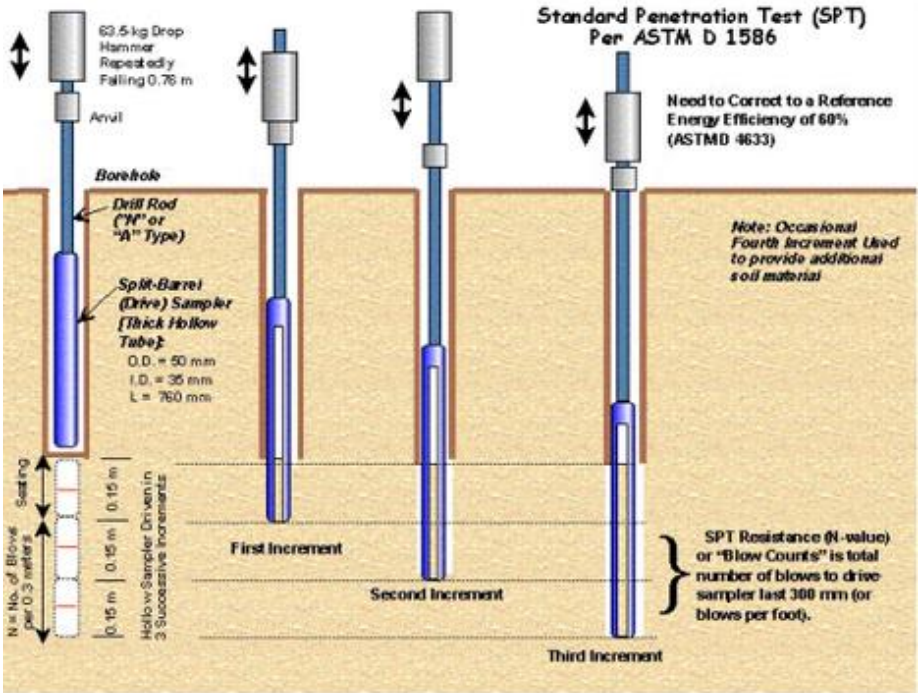

Figure 36: Standard Penetration Test

The N-value provides an indication of the relative density of the subsurface soil, and it is used in empirical geotechnical correlation to estimate the approximate shear strength properties of the soils. The N_{SPT} values obtained are shown in Table three, which proves that the resistance increases with depth.

Table 3: N_{SPT} values obtained

Depth (mm)	N_{SPT} for S1	N_{SPT} for S2
150	7	15
300	5	22
450	7	36
600	25	23
750	46	32
900	60	60
1050	60	60
1200	60	60
1350	60	-
1500	60	-

$5 \leq N_{SPT} \leq 15$	**$22 \leq N_{SPT} \leq 46$**	**$N_{SPT} \geq 60$**

4.6.3.3 *Inspection shafts*

Two inspection shafts (P1 and P2) were carried out, with the objective of assessing the geometry of the foundations of the side walls at the end of the building, *Calçada da Patriarcal*, n ° 33. Its location is represented in picture 36. In inspection shaft P1, the foundation element of the wall consists of blocks (D_{max} = 40/50cm) surrounded by compact cement mortar. At a depth of 0,80m there is a foundation beam. The excavation of the well was interrupted at 2,15m depth, without having reached the foundation of the wall, due to safety reasons. The inspection shaft P2 also presents blocks (D_{max}= 30cm) surrounded by compact cement mortar. This foundation element rests on blocks of limestone with lengths varying between 0,50m and 1,05m. Due to the existence of these structures and also for safety reasons, it was not possible to continue the excavation beyond 1,82m depth. More details about the inspection shafts can be found in appendix 3.

Figure 37: Inspection shafts (TECNASOL FGE.,2017)

4.6.4 Hydrogeological information

After the geotechnical tests, two piezometers were installed in the holes of the drillings and two Lefranc type permeability test and Slug type tests were done to see the hydrogeological characterization of the area.

4.6.4.1 *Piezometers*

A piezometer is a small diameter plastic pipe with a porous section at the bottom designed to measure static pressures. The pipe is installed inside a borehole and the porous section is positioned at the depth where the pore water pressure is to be measured. The annulus between the porous filter and the borehole is filled with sand, the top and bottom surfaces of the sand are sealed with bentonite and the rest of the borehole is filled with a cement/bentonite grout. The pressure of the ground water pushes water into and up the standpipe until the level of water inside the standpipe is equivalent to the pore water pressure in the ground at the elevation of the porous filter. Water level meters are set to measure water levels in piezometers, monitoring wells, and bore holes. These meters then emit an audio or visual signal to denote where the water level is.

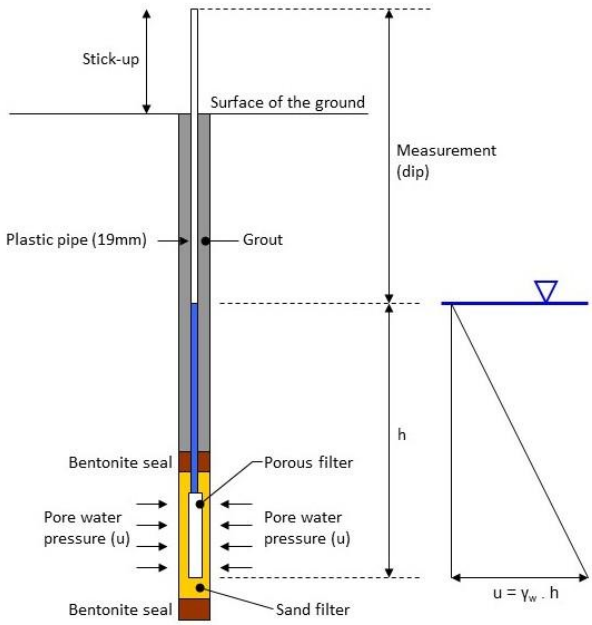

Figure 38: Piezometer (http://www.geo-observations.com/piezometers/)

Figure 39: Water level meter (https://www.geosense.co.uk/products/details/water-level-meter)

The analysis of the water level readings lead to the conclusion that there should be a stabilized water table in the area of the Jasmin Noir building, about 12,50m in S1 and at 10,60m in S2. This can be visualised in appendix 2.

4.6.4.2 *Lefranc test*

Site measurement of subsoil permeability is mainly performed within a borehole by recording the variation of the water level. During the Lefranc type test, the filled-up borehole is kept constant and the water supply rate is recorded to get a steady water level. Based on those records, the permeability coefficient is obtained.

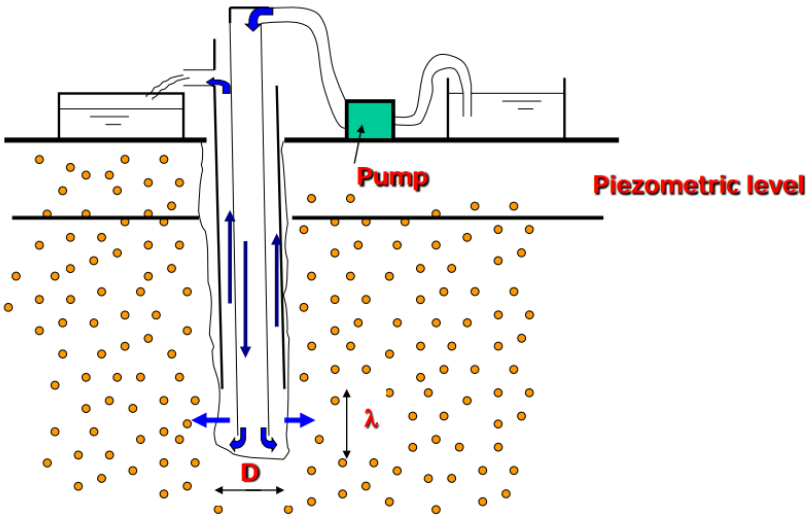

Figure 40: Lefranc test (ftp://ceres.udc.es/International%20Master%20in%20Water%20Engineering/1.7%20Groundwater%20Engineering/2016_2017/clase%2011%20hydraulic%20testing%20fractured%20rocks.pdf)

Table 5 shows the depth of the tests and its results, being the coefficient of permeability. The very low K-values are related to the high percentage of clays in the soil.

Table 4: Lefranc test results

Borehole	Depth (m)	Permeability K (m/s)
S1	3,0-6,0	2.10^{-6}
S2	1,5-3,0	$4{,}5.10^{-8}$

4.6.4.3 *Slug test*

A slug test is a particular type of aquifer test to estimate the local hydraulic conductivity of the material surrounding a well. Water is quickly added or removed and the time of return to static water levels is measured, to determine the near-well aquifer characteristics.

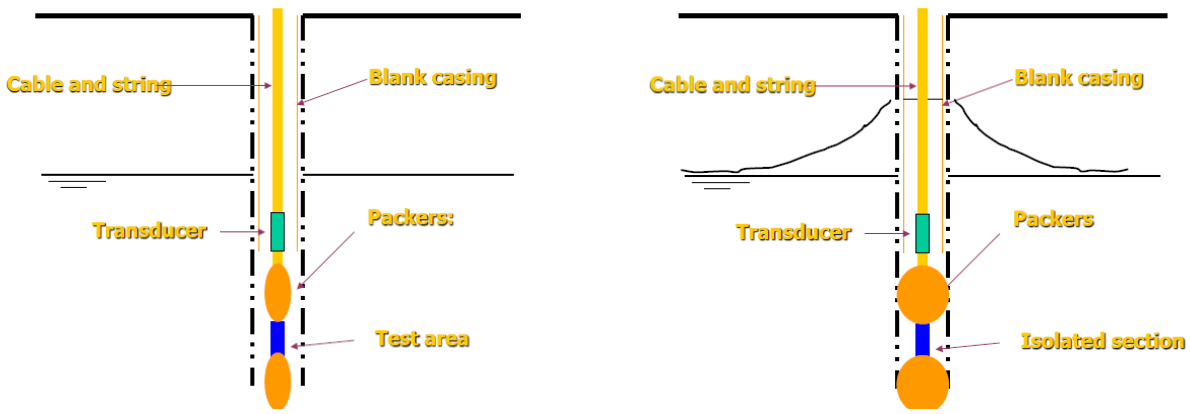

Figure 41: Slug test (ftp://ceres.udc.es/International%20Master%20in%20Water%20Engineering/1.7%20Groundwater%20Engineering/2016_2017/clase%2011%20hydraulic%20testing%20fractured%20rocks.pdf)

According to Table 6, the K-values obtained classify the medium as waterproof and without capacity as an aquifer.

Table 5: Slug test results

Bore hole	Depth (m)	Permeability K (m/s)
S1	1,5-14,0	$1{,}76.10^{-9}$
S2	1,5-9,5	$2{,}01.10^{-9}$

4.7 Argumentation of the chosen techniques

The most important aspect that must be taken into account when choosing the techniques is the very small width of the construction site. A more detailed plan of the construction site can be found in appendix 4. At the rear wall there is 5,50m available and at the front wall there is only 5,12 m. This ensures that the chosen technique has to be possible within these dimensions. The Munich-type walls

are one of Portugal's most used containment systems because there's no need of specialized employees or technology. It's perfectly possible to execute these walls in a small work area and it saves a lot of space because they are executed against the soil. On top of that, it's supposed to be one of the cheapest options.

4.7.1 Estimated cost of Munich walls

We can calculate the cost of this technique, taking into account the general materials for the execution of the Munich walls to give a brief overview. The cost is € 232,32 per m² of the retaining wall plan view.

Table 6: Estimated costs of Munich walls

Element	Quantity	Price/quantity	Total price
Micropiles			
Micropiles Ø127,0x9mm	384,10 m	€ 85/m	€ 32 648,50
Micropiles Ø88,9x9mm	23,70 m	€ 70/m	€ 1 659
Metal cantilevers supporting micropiles	46	€ 200	€ 9 200
Core beam			
Formwork	95,50 m²	€ 20/m²	€ 1 910
Concrete C30/37	22,10 m³	€ 100/m³	€ 2 210
Steel reinforcement	3 973,40 kg	€ 1/kg	€ 3 973,40
Bolts	546,80	€ 50	€ 27 340
Munich walls			
Formwork	446,40 m²	€ 20/m²	€ 8 928
Concrete C30/37	123,60 m³	€ 100/m³	€ 12 360
Steel reinforcement	22 879,40 kg	€ 1/kg	€ 22 879,40
Total price:			€ 123 108,30

4.7.2 Top/down excavation

In this case the basement floors are constructed as the excavation progresses. This method is called 'top/down excavation' and is often used for deep excavation projects where ground anchor installation is not feasible and/or neighbouring constructions don't allow this technique, but soil movements have to be minimized. The micropiles are constructed before any excavation takes place and each floor rests on these basement columns that were constructed in the beginning.

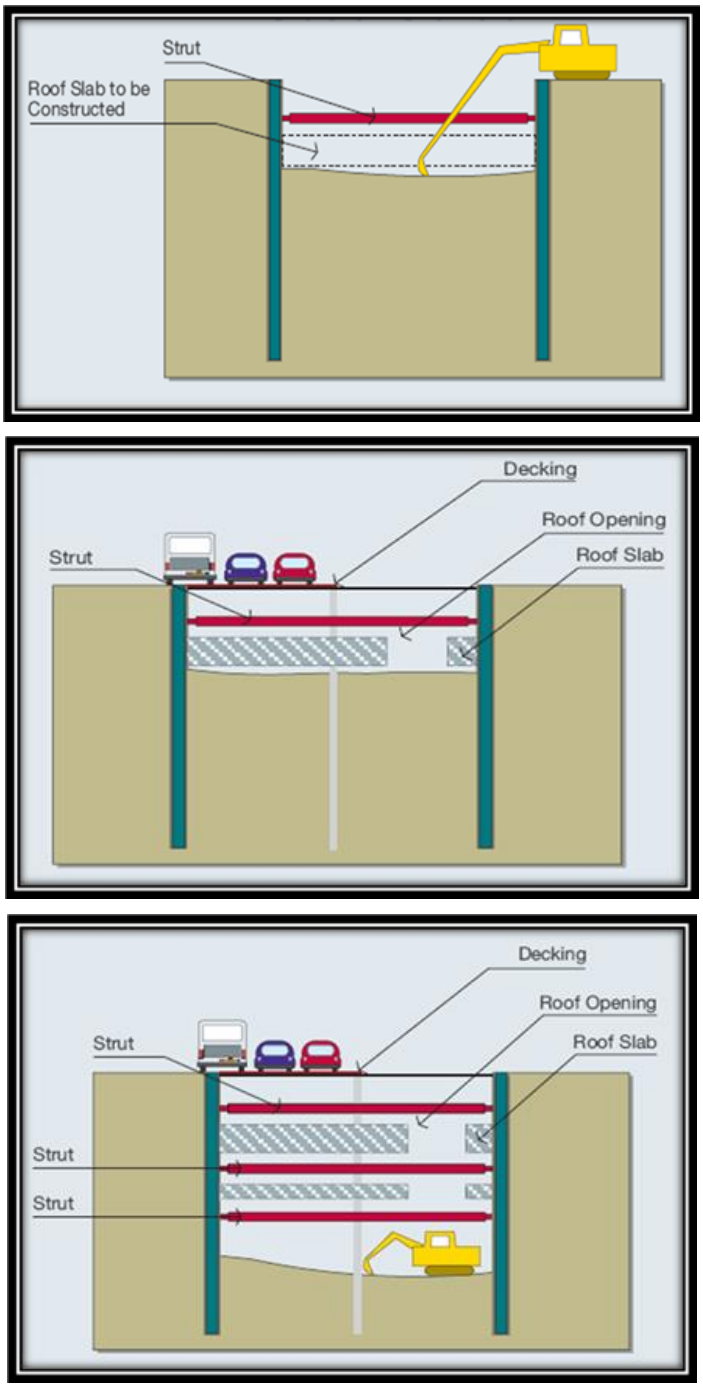

Figure 42:Traditional top/down construction method (https://www.haxworld.net/civil/top-down-construction-method/)

4.8 Operating sequence

The first months of this case study will be spent observing the demolishing and shoring of the buildings' structure. This should be done with a lot of care to make sure safety is secured. When this is finished, the micropiles can be installed vertically and the crown beam can be made. Finally, the excavation can start while executing the Munich-type walls. This last process can't be observed because of the short term Erasmus exchange.

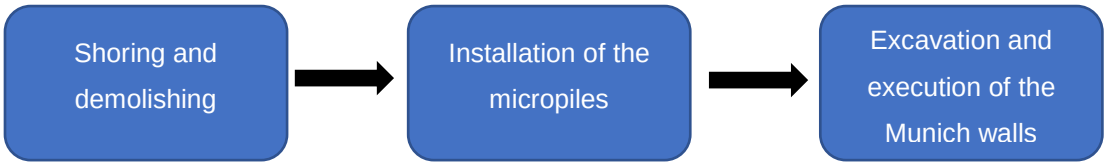

Figure 43: Operating sequence

4.9 Shoring and demolishing

Shoring the facades is necessary when the building's structure will be demolished and the facades have to be preserved. The shores must be chosen in function of the expected movements. Horizontal and oblique shores counteract the general rotation of the walls. The facades are anchored to the neighbouring buildings, so they could not suddenly start to move and fall over.

Figure 44: Shoring and demolishing upper floors

Figure 45: Shoring and demolishing existing basement

4.10 Installation of the micropiles

The micropiles are installed, starting from the back of the building and moving to the front facade. A small diameter steel casing is placed and the boring process starts while fluid is flushed inside the drill casing. When the desired depth has been reached, the drilling fluid is replaced by cement grout and the micropile is formed.

Figure 46: Execution of the micropiles

4.11 Excavation and execution of the Munich walls

The process of the excavation using the top/down-method and the execution of the Munich walls can’t be observed because of the short term of the Erasmus exchange. The behaviour of the soils during this excavation and further progress of the construction can be studied in the future.

5. Modelling in PLAXIS 2D

Geotechnical applications require advanced constitutive models for the simulation of the non-linear, time-dependent and anisotropic behaviour of soils and/or rock. PLAXIS 2D is a finite element program, developed for the analysis of deformation, stability and groundwater flow in geotechnical engineering. With Staged Construction the software can accurately model the construction process, by activating and deactivating soil clusters and structural elements in each calculation phase. With plastic, consolidation and safety analysis calculation type, a broad range of geotechnical problems can be analysed.

5.1 General Characterization of the Program

The development of PLAXIS began in 1987 at Delft University of Technology as an initiative of the Dutch Ministry of Public Works and Water Management. The initial purpose was to develop an easy-to-use 2D finite element code for the analysis of river embankments on the soft soils of the lowlands of Holland. In subsequent years, PLAXIS was extended to cover most other areas of geotechnical engineering. The PLAXIS 2D software has implemented the following constitutive laws to model different soils.

5.1.1 Linear Elasticity

The Linear Elastic model is based on Hooke's law of isotropic elasticity. By definition this means that the material properties are independent of direction. Such materials have only two independent variables (elastic constants) in their stiffness and compliance matrices, as opposed to the 21 elastic constants in the general anisotropic case. The two elastic parameters are Young's modulus *E* and the Poisson's ratio ν. The Young's modulus is a measure of the stiffness of a solid material which defines the relationship between stress and strain in a material. The Poisson's ratio is a measure of the Poisson effect, the phenomenon in which a material tends to expand in directions perpendicular to the direction of compression. It's known as the negative of the ratio of transverse strain to axial strain. Although the Linear Elastic model is not suitable to model soil, it may be used to model stiff volumes in the soil like concrete walls or intact rock formations.

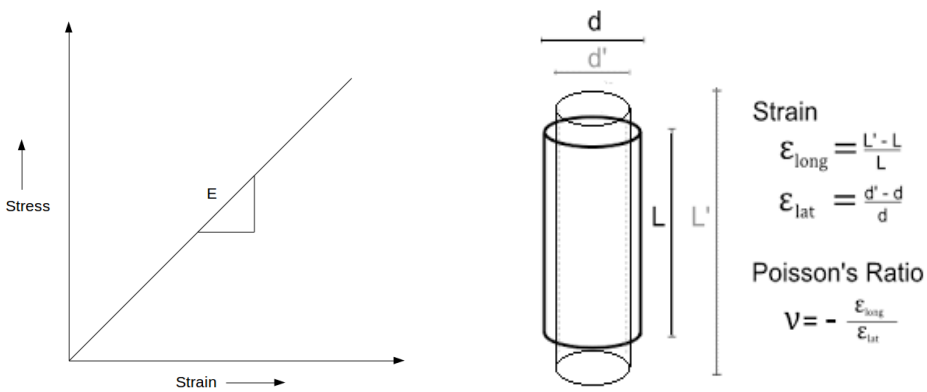

Figure 47: Young's modulus and Poisson's ratio (https://www.simscale.com/docs/content/simulation/model/materials/youngsModulus.html and https://www.quora.com/What-is-the-poison-ratio)

5.1.2 Mohr-Coulomb

The linear elastic perfectly-plastic Mohr-Coulomb model is the most common model in the context of geomaterials and in particular soils (Owen D.R.J. & Hinton E., 1980; Pietruszczak, S., 2010). The specification of this model and its yield criterion typically involves Coulomb's hypothesis, which postulated a linear relationship between shear strength on a plane and the normal stress acting on it. The model involves five input parameters, *E* and ν for soil elasticity; φ and *c* for soil plasticity and ψ as an angle of dilatancy. This Mohr-Coulomb model represents a 'first-order' approximation of soil or rock behaviour. For each layer one estimates a constant average stiffness or a stiffness that increases linearly with depth. Due to this constant stiffness, computations tend to be relatively fast and one obtains a first estimate of deformation.

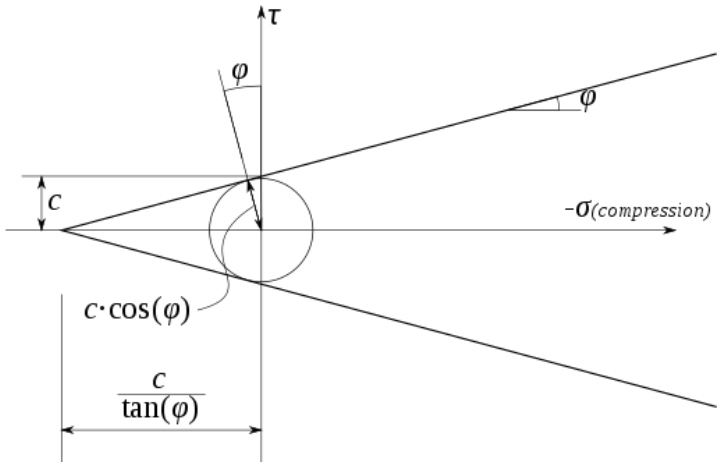

Figure 48: Mohr-Coulomb theory (https://www.doitpoms.ac.uk/tlplib/granular_materials/mohr-coulomb.php)

5.1.3 Hardening-Soil Model

As for the Mohr-Coulomb model, limiting states of stress are described by means of the friction angle, φ, the cohesion, c and the dilatancy angle, ψ. The Hardening Soil model is an advanced model for the simulation of soil behaviour. Soil stiffness is described much more accurately by using three different input stiffness's: the triaxial loading stiffness, E_{50}, the triaxial unloading stiffness, E_{ur}, and the oedometer loading stiffness, E_{oed}. As average values for various soil types $E_{ur} \approx 3\ E_{50}$ and $E_{oed} \approx E_{50}$ are suggested as default settings, but both very soft and very stiff soils tend to give other ratios of $Eoed/_{E50}$. In contrast to the Mohr-Coulomb model, the Hardening Soil model also accounts for stress-dependency of stiffness moduli. This means that all stiffness's increase with pressure. Hence, all three input stiffness's relate to a reference stress, usually taken as 100 kPa (1bar). Besides the model parameters mentioned above, initial soil conditions, such as pre-consolidation, play an essential role in most soil deformation problems. This can be taken into account in the initial stress generation. This model will be used in the 'Jasmin Noir' case.

5.1.4 Soft-Soil Model

The Soft Soil model is a Cam-Clay type model especially meant for primary compression of near normally-consolidated clay-type soils. The Original Cam-Clay model is based on the assumption that the soil is isotropic, elasto-plastic, deforms as a continuum, and it is not affected by creep. Although the modelling capabilities of this model are generally superseded by the hardening Soil model, the Soft Soil model is better capable to model the compression behaviour of very soft soils.

5.1.5 Soft-Soil-Creep Model

The Hardening Soil model is generally suitable for all soils, but it does not account for viscous effects like creep and stress relaxation. In fact, all soils exhibit some creep and primary compression is thus followed by a certain amount of secondary compression. The latter is most dominant in soft soils, i.e. normally consolidated clays, silts and peat. For unloading problems, as normally encountered in tunnelling and other excavation problems, the Soft Soil Creep model hardly supersedes the simple Mohr-Coulomb model.

5.1.6 Jointed Rock Model

The Jointed Rock model is an anisotropic elastic-plastic model, especially meant to simulate the behaviour of rock layers involving stratification and particular fault directions. Plasticity can only occur in a maximum of three shear directions (shear planes). Each plane has its own strength parameters φ and c. The intact rock is considered to behave fully elastic with constant stiffness properties E and ν. Reduced elastic properties may be defined for the stratification direction.

5.2 Geometry

The input of structures, loads and boundary conditions is based on convenient CAD drawing procedures, which allows for a detailed modelling of the geometry. The section that will be used is the one where the excavation is the deepest, which is at section DE/FG, closest to the front facade. Appendix 4 shows the location of this section.

First, the four different layers of soil have to be made and then the excavation pit can be defined. The excavation is 5,68m wide and 11,05m deep. Munich type walls are used, in combination with micropiles to each wall. To model the Munich walls, plates are used on each side of the excavation. These plates have to be divided into lengths of the successive phases of the excavation, which are: 2,86m, 2,74m, 2,35m and 2,60m. The micropiles are presented as plates, to transfer the loads vertically to the soil of ZG1 and the slabs are presented as plates as well. The interaction between the wall and the soil is modelled at both sides by means of interfaces, to obtain a more accurate stress distribution and to avoid unrealistic bearing capacity. The interfaces allow for the specification of a reduced wall friction compared to the friction in the soil. The main types of loading available in the program are distributed loads and point loads. Since the weight of the facade is transferred directly to the micro-slabs to the foundation

soil, the only loads considered were those of neighbouring buildings (distributed loads), which is equivalent to 10 kN / m² per floor since the program already takes into account the impulses caused by the terrain. To define the load the 'Distributed load - load system A' is used, with 40 kN/m² for the left building and 50 kN/m² for the building on the right of the excavation. To create the boundary conditions, the 'Standard fixities' button is used. As a result, the program will generate full fixities at the bottom and vertical rollers at the vertical sides. Prescribed displacement in the x-direction is set to 0m where there are adjacent buildings.

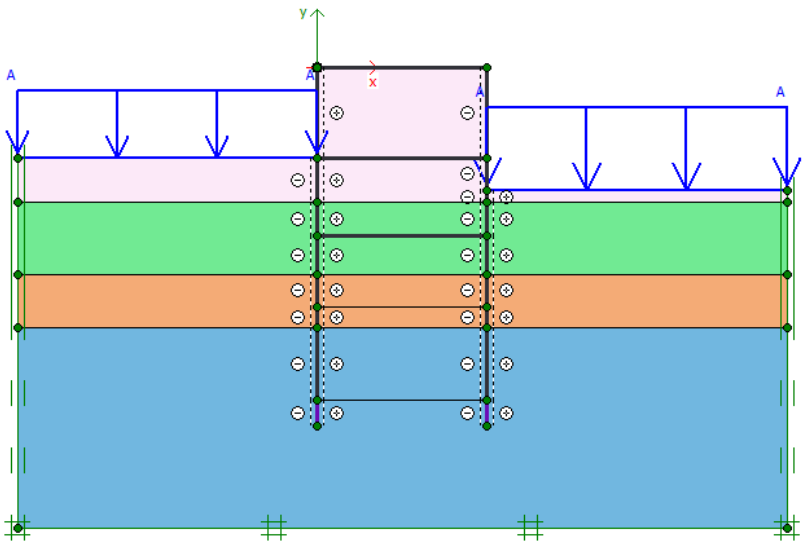

Figure 49: Geometry Munich walls PLAXIS

5.3 Defining the materials

The relevant part of the soil consists of four distinct layers. First of all, four data sets for soil and interfaces are made, based on the average parameters provided by the geological-geotechnical report in Table 2 and some correlations between the parameters. The Hardening Soil model is an advanced model for simulating the behaviour of different types of soil, both soft soils and stiff soils. (Schanz, T., Vermeer, P. A., & Bonnier, P. G., 1999) Therefore, it's necessary to define the following parameters.

Table 7: Hardening soil parameters

Parameter	Definition
γ_{unsat}	Specific weight, unsaturated
γ_{sat}	Specific weight, saturated
Failure parameters as in Mohr-Coulomb model	
c_{ref}	Cohesion of the soil
ϕ	Angle of internal friction
ψ	Angle of dilatancy
Basic parameters for soil stiffness	
m	exponent of the power law that expresses the dependence of rigidity on the level of tension (power);
E^{ref}_{50}	Triaxial loading stiffness, for p_{ref}, considered equal to 100 kPa
E^{ref}_{ur}	Triaxial unloading stiffness, for p_{ref}
E^{ref}_{oed}	Oedometer loading stiffness, for p_{ref}
R_{inter}	Interface resistance reduction factor
Advanced parameters (advised to use default setting)	
p_{ref}	reference tension for stiffness
ν^{nu}_{ur}	Poisson's ratio in the discharge phase
R_f	Breaking coefficient
K_0	Impulse coefficient at rest
G_{ref}	Distortion module

We can define some correlations between these parameters, represented in the following equations:

$$E \approx E^{ref}_{50} \quad [5.3.1]$$

$$E^{ref}_{ur} \approx 3 \times E^{ref}_{50} \quad [5.3.2]$$

$$E^{ref}_{oed} \approx E^{ref}_{50} \quad [5.3.3]$$

A suitable value for m-factor is given in Table 9. For the soils in the Jasmin Noir project m = 0,7 is used.

Table 8: Power factor m

Type of soil	M
Sand	0,5
Silt	0,5-0,7
Clay	1

Table 9 shows the parameters used for making data sets for the different layers of soil.

Table 9: Used parameters for the hardening-soil model

Parameters	Geotechnical zones			
Hardening-soil model	**ZG4**	**ZG3**	**ZG2**	**ZG1**
γ_{unsat} **[KN/m³]**	10	19	20	21
γ_{sat} **[KN/m³]**	14	21	21	22
e_{init}	0,5	0,5	0,5	0,5
E^{ref}_{50} **[KN/m²]**	3500	24500	40500	60000
E^{ref}_{ur} **[KN/m²]**	10500	73500	121500	180000
E^{ref}_{oed}**[KN/m²]**	3500	24500	40500	60000
c_{ref} **[KN/m²]**	0	36	80	125
ν^{nu}_{ur}	0,2	0,2	0,2	0,2
Φ [°]	17,5	31,5	35	37,5
Ψ [°]	0	0	0	0
R_{inter}	1	1	1	1
R_f	0,9	0,9	0,9	0,9
p_{ref} **[KN/m²]**	100	100	100	100
M	0,7	0,7	0,7	0,7
K_0	0,5	0,5	0,5	0,5

Structural materials such as the Munich walls and micropiles were also assigned characteristics. For the curtain plate two materials were defined: the micropiles N80 and reinforced concrete. The values of the axial stiffness (EA) and the flexural stiffness (EI) of the reinforced concrete (e = 0.25m) were calculated based on equations [4.3.5] and [4.3.6], where E represents the modulus of elasticity of concrete (30 GPa). The weight of the reinforced concrete is based on the weight density of 25 kN/m³.

$$EA = E\ .\ e \text{ [kN/m]} \qquad [5.3.5]$$

$$EI = E\ .\ \frac{e^3}{12} \text{ [kNm²/m]} \qquad [5.3.6]$$

$$w = 25\ {}^{kN}/_{m^3}\ .\ e \text{ [kN/m/m]} \qquad [5.3.7]$$

The values for the micropiles are calculated based on E=210 GPa for steel, D=127mm, d=9mm and using equations [4.3.8], [4.3.9] and [4.3.10]. The weight of the micropiles is based on the weight density of steel, 78,5 kN/m³. These values are divided by 3m of the perimeter of the curtain wall.

$$EA = E\ .\pi\ .(D^2 - d^2) \text{ [kN/m]} \qquad [5.3.8]$$

$$EI = E\ .\pi\ .\frac{(D^4 - d^4)}{64} \text{ [kNm²/m]} \qquad [5.3.9]$$

$$w = 78{,}5\ {}^{kN}/_{m^3}\ .\pi\ .(D^2 - d^2)\ \text{[kN/m/m]} \qquad [5.3.10]$$

Table 10: Used parameters for the structural materials in PLAXIS

Parameters	Micropiles N80	Reinforced concrete
EA [kN/m]	3 529 139,52	7 500 000
EI [kNm²/m]	893,86	39 062,5
w [kN/m/m]	1,32	6,25
v	0,3	0,2

5.4 Mesh

PLAXIS 2D allows you to create curves, obtained from pre-selected points in the finite element mesh. The process of generating the mesh is an automatic function, the geometry of each predefined zone is divided into triangular isoparametric elements of six or fifteen knots. The accuracy of the results depends on the shape and size of the mesh that represents the physical system. More refined meshes tend to give better results.

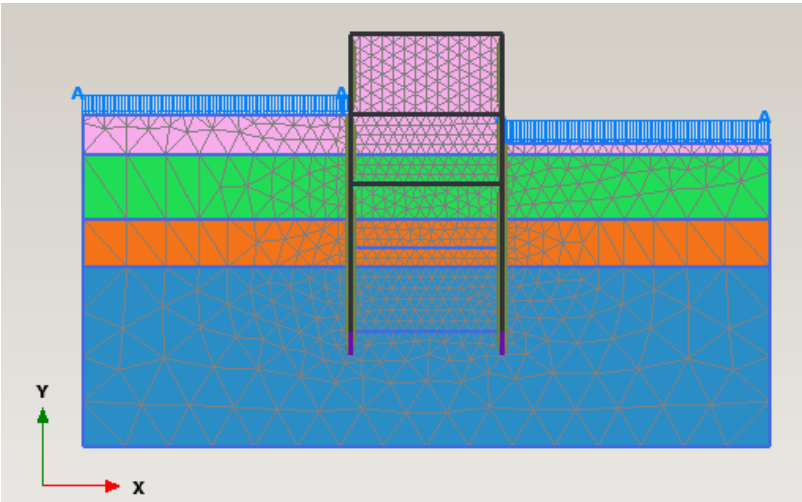

Figure 50: Generated mesh for Munich walls

5.5 Calculation

In the calculate subroutine it is necessary to characterize the various phases of the construction of the excavation, trying to reproduce it as closely as possible to reality. In practice, the construction of an excavation is a process that can consist of several phases. In the initial phase (phase 0), the initial stresses are generated. Phase 0 is defined by default by the program, where all the displacements are due to the weight of the ground, overloads and the initial conditions. All structural elements and loads that are present in the geometry are initially automatically switched off, only the soil volumes are initially active. It's important to activate the prescribed displacements in this phase, so the soil body doesn't

collapse. In the phase 1, the surface loads are activated and in phase 2 the micropiles are placed vertically on both sides of the excavation, activating the plates and assigning them the material settings of micropiles. This way the loads can be transferred vertically to soil ZG1. The first slab is placed in phase 3 and then the excavation can start.

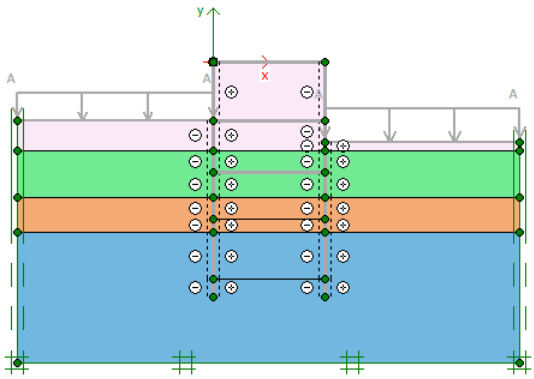

Figure 51: Phase 0: initial phase

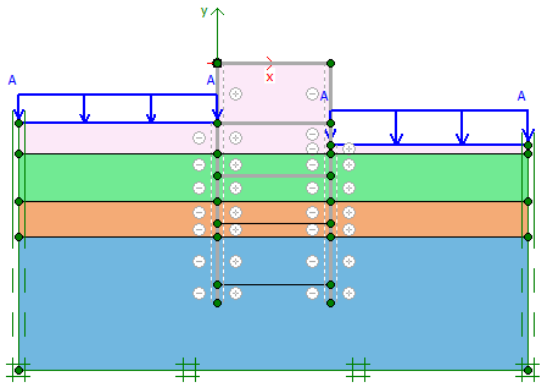

Figure 52: Phase 1: external load

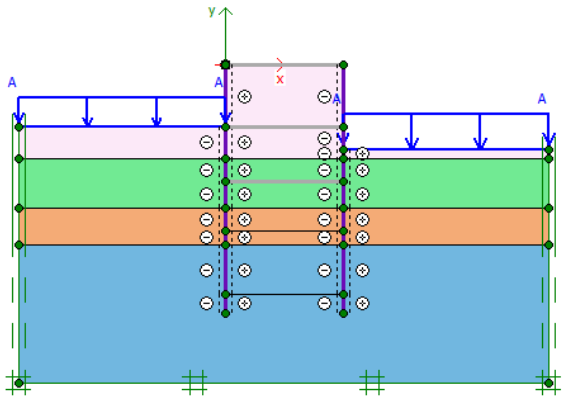

Figure 53: Phase 2: micropiles

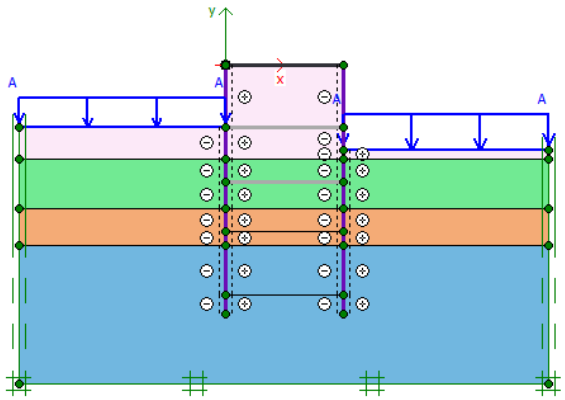

Figure 54: Phase 3: first slab

After the upper cluster of the excavation is de-activated in phase 4, the Munich walls are placed at the sides of this cluster, assigning them the material of reinforced concrete instead of micropiles. When doing a 2D analysis rather than 3D in this type of flexible containment system the arch effect (part 3.2.3) is underestimated, this is the phenomenon of pressure transfer in the soil. As already mentioned, excavation in the Munich-type wall technology is done alternately by executing the primary and secondary panels in distinct phases, so that the created arch effect can be taken advantage of, allowing the excavation to occur without decompression of the terrain. One of the limitations of the program is that it does not take this into account, but we can split the phase of the execution of the Munich walls into two phases. In general, the total multiplier associated with the staged construction process, ΣM_{stage}, goes from zero to unity in each calculation phase where staged construction has been selected as the loading input. In some very special situations it may be useful to perform only a part of a construction stage. This can be done by specifying to ΣM_{stage} a value lower than 1. In the first phase, a ΣM_{stage} value of 0.7 is adopted, this represents the moment where the secondary panels are placed so the Munich wall in PLAXIS is not activated. For the second phase a ΣM_{stage} value of 0.3 is adopted, here the primary

panels are placed and the Munich walls can be activated in PLAXIS. Thus, this solution does not present the exact solution, but it is the one that allows a better approximation to reality. Finally another slab is placed and the excavation can continue this way until the desired depth is reached.

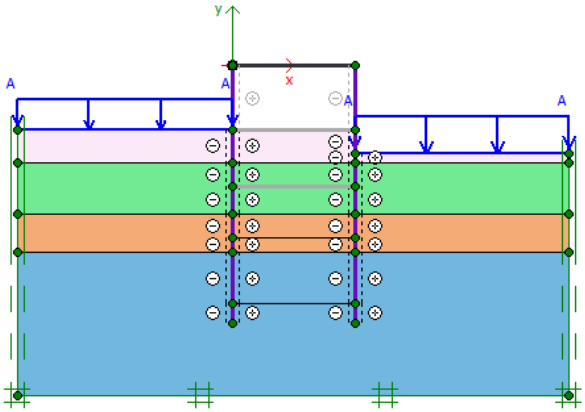

Figure 55: Phase 4: first level excavation (2,86m)

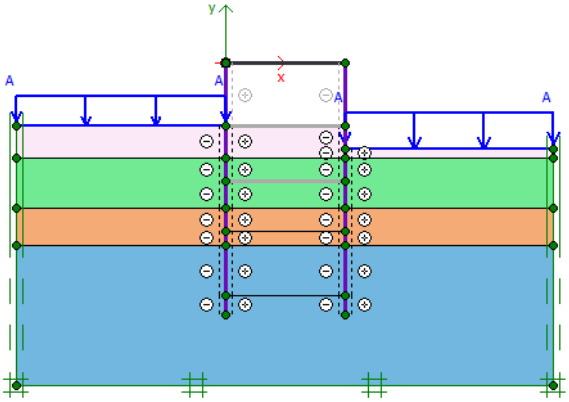

Figure 56: Phase 5: first level Munich walls (ΣMstage = 0.7) (2,86m)

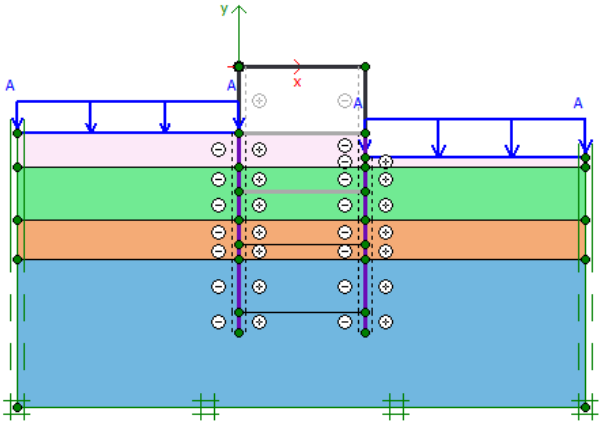

Figure 57: Phase 6: first level Munich walls (ΣMstage = 0.3) (2,86m)

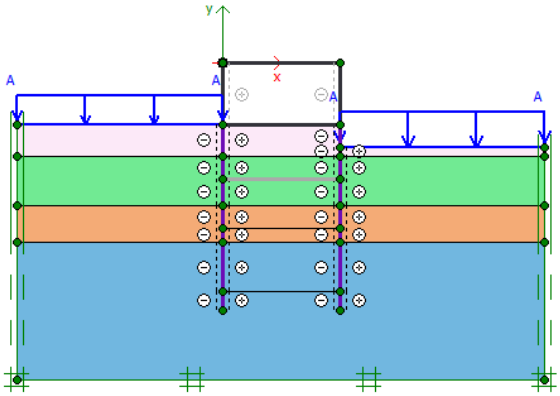

Figure 58: Phase 7: second slab

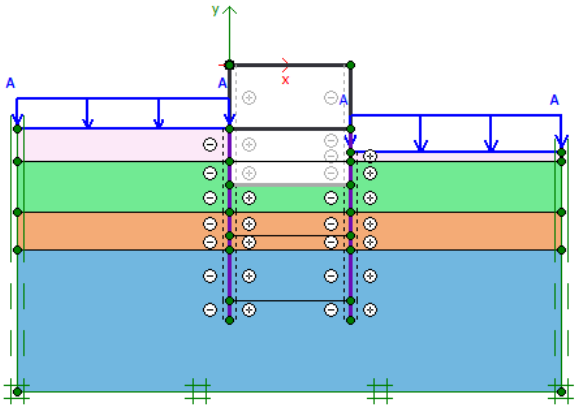

Figure 59: Phase 8: second level excavation (2,74m)

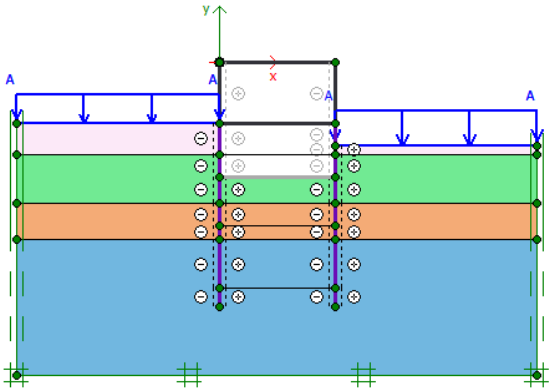

Figure 60: Phase 9: second level Munich walls (ΣMstage = 0.7) (2,74m)

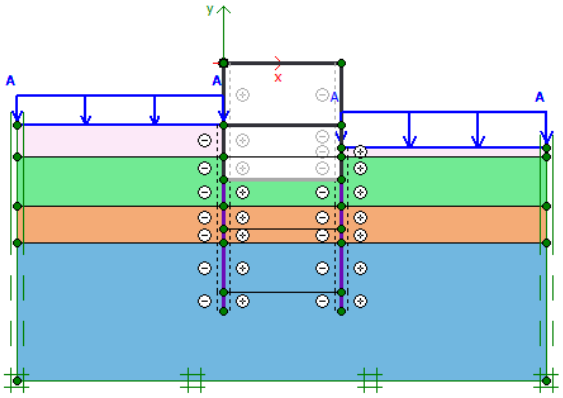

Figure 61: Phase 10: second level Munich walls (ΣMstage = 0.3) (2,74m)

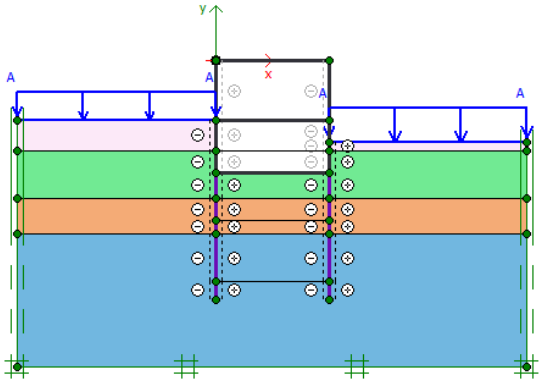

Figure 62: Phase 11: third slab

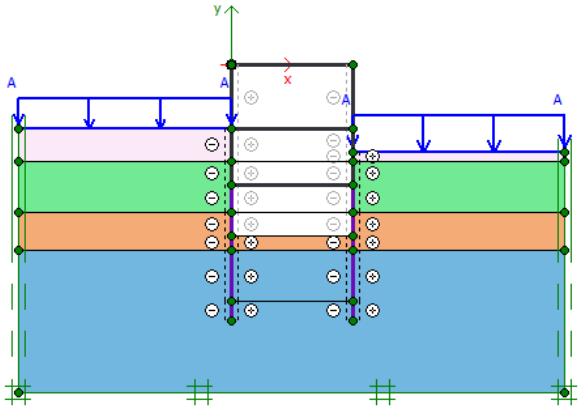

Figure 63: Phase 12: third level excavation (2,35m)

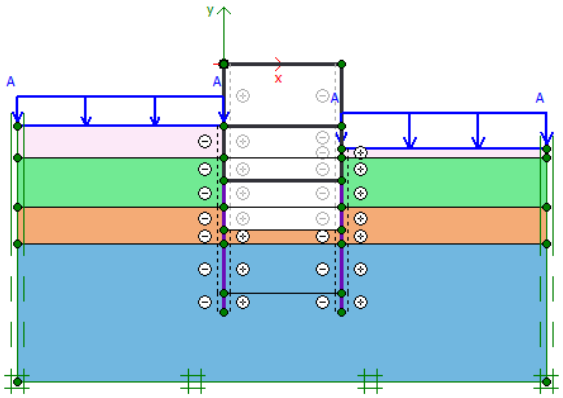

Figure 64: Phase 13: third level Munich walls (ΣMstage = 0.7) (2,35m)

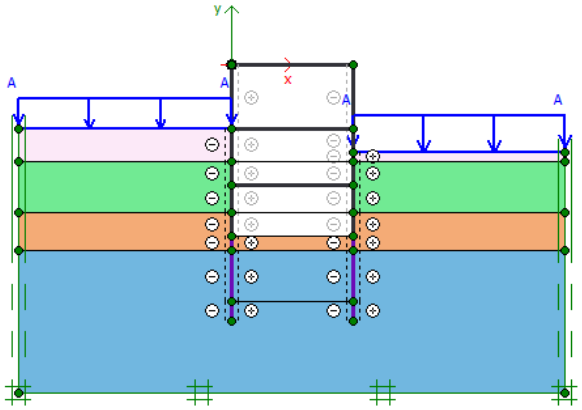

Figure 65: Phase 14: third level Munich walls (ΣMstage = 0.3) (2,35m)

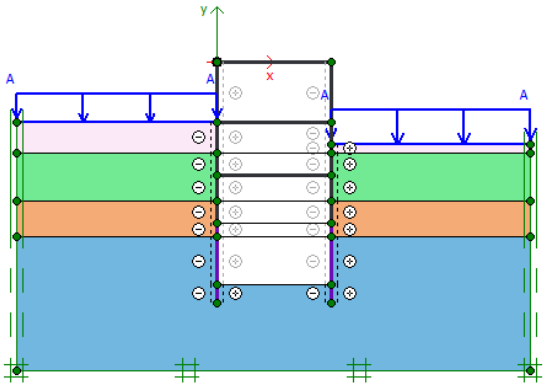

Figure 66: Phase 15: fourth level excavation (2,60m)

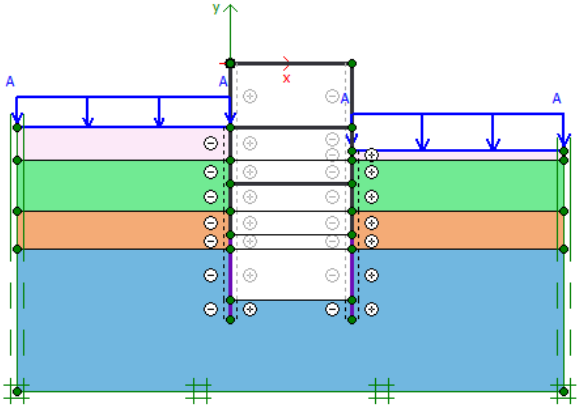

Figure 67: Phase 16: fourth level Munich walls (ΣMstage = 0.7) (2,60m)

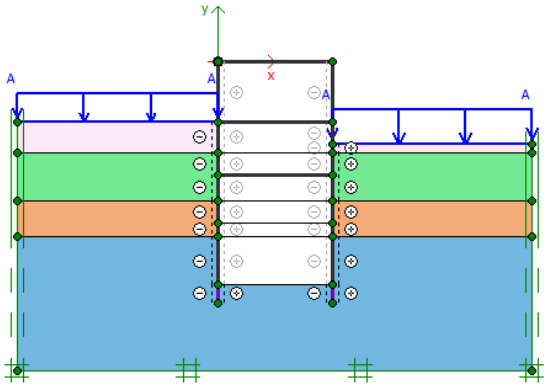

Figure 68: Phase 17: fourth level Munich walls (ΣMstage = 0,3) (2,60m)

5.6 Main results

Figure 71 and Figure 72 show the horizontal and vertical expected displacements of the soil. The criteria for evaluating movements are shown in Table 12.

Table 11: Criteria for evaluation movements

Horizontal	Vertical
< 15mm	< 10mm
> 15mm	> 10mm
> 30mm	> 20 mm

Admissible values
Alert!
Alarm!

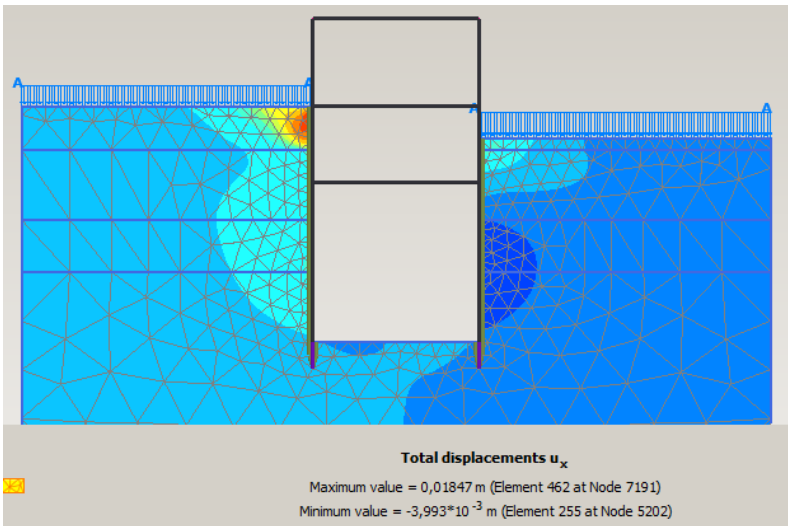

Figure 69: Horizontal displacements u_x Munich walls

According to Figure 71, it can be seen that the maximum horizontal displacement occurs behind the pile curtain, under the adjacent building number 32. This horizontal displacement is in the direction of the interior of the excavation and has a maximum value of approximately 18 mm. This is 3mm more than the admissible value, but is not an alarm criteria. Eventually the reinforcement can be increased. However, experience has shown that this deformation value is considerably higher than those that happen in reality.

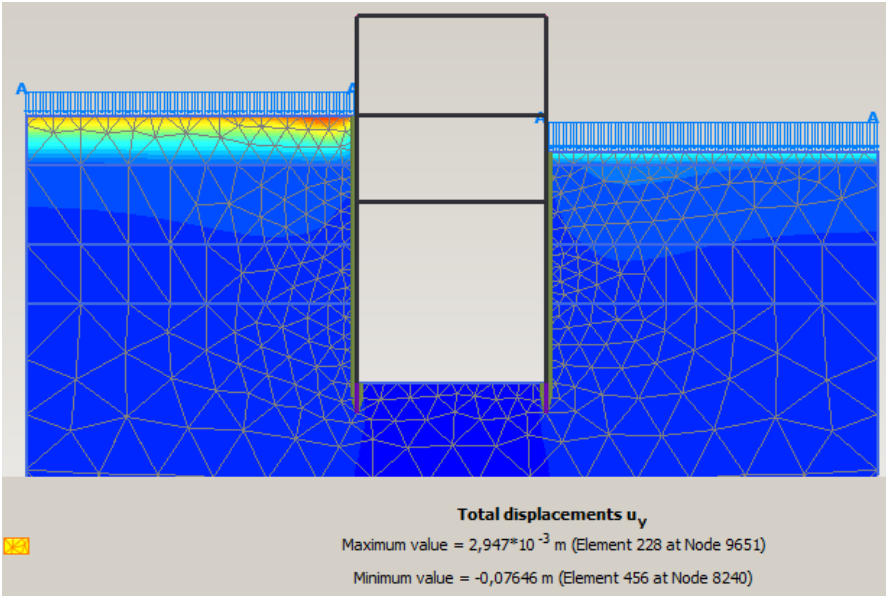

Figure 70: Vertical displacements u_y Munich walls

In Figure 72 it can be seen that the maximum vertical displacements occur under the neighbouring building number 32, corresponding to settlements of 76 mm. This is because of the load of the building, not because of the excavation. The upper layer of the soil will settle under the load because the foundations of the adjacent buildings are not modelled. At the base of the excavation and next to the curtain wall, there is a displacement of about 8 mm. This is lower than 10mm, the admissible value.

6. Alternative solutions

Several companies who design and develop geotechnical engineering solutions are asked to propose other possible solutions for the Jasmin Noir project. Belgian and Portuguese companies have different experience gathered through numerous geotechnical projects, so they give different solutions. These solutions are examined technically, practically and economically for this case study.

6.1 Underpinning in a contained slot

Munich type walls are an easy, cheap containment method which can be executed against the soil. This is a very common used technique in Portugal, but isn't used like this is other countries. In Belgium there's a very common used method with almost the same advantages, but there are some differences. As far as the stability of the upper structure allows it, a pre-excavation is realized with a minimum of 0,50m above the existing foundation that should be kept. The excavation of the slot will happen in horizontal phases, like the Munich type walls, to avoid the soil-arching effect to damage the structure of adjacent buildings. The width of these horizontal strips is usually 1m, while the sloth has a length of 1,5 to 2m, which makes it possible to carry out the excavation work with sufficient room for manoeuvre.

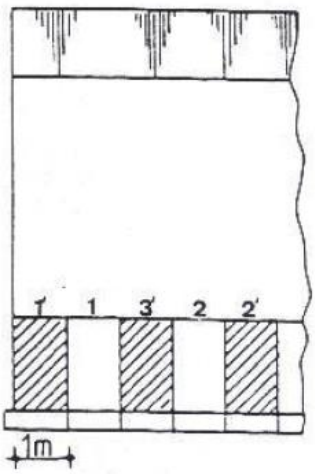

Figure 71: Horizontal staging (Wylaers M., 2016, Bouwtechniek 1 – Underpinning.)

The soil is excavated in vertical stages of 0,4m while systematically applying formwork over the entire perimeter of the pit. The wall that is under the existing foundation is covered with lost prefabricated concrete elements. The other three sides are temporarily covered (for example with wood). The evacuation of the excavated soil is usually done with the help of a bucket lift. This process is repeated until the predetermined depth is reached. After the completion of the excavation process, a reinforcement is placed first and then the formwork of the front side of the wall is put in place. Eventually the concrete can be poured. These first phase strips have to harden for a couple of days, before the second phase can start. This second phase follows the same steps, but the prefabricated concrete formwork has to be placed behind the ones of the first phase, so a continuous wall is created.

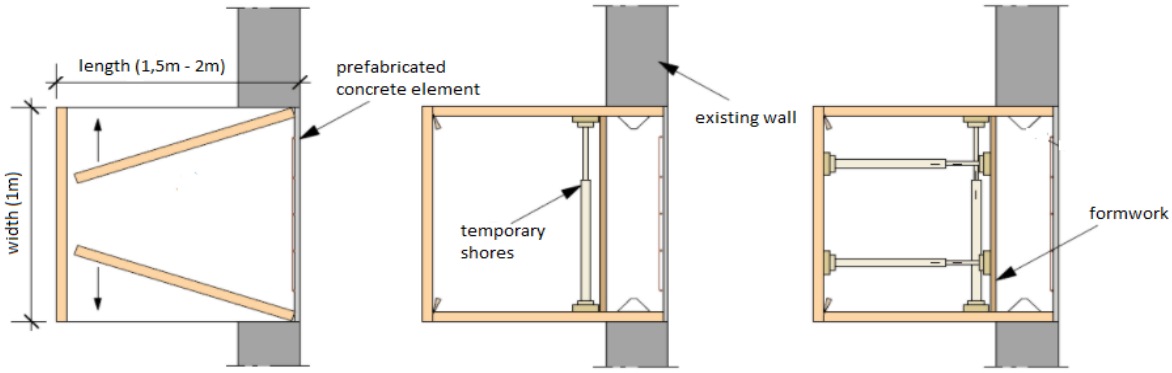

Figure 72: Contained slot (Wylaers M., 2016, Bouwtechniek 1 – Underpinning.)

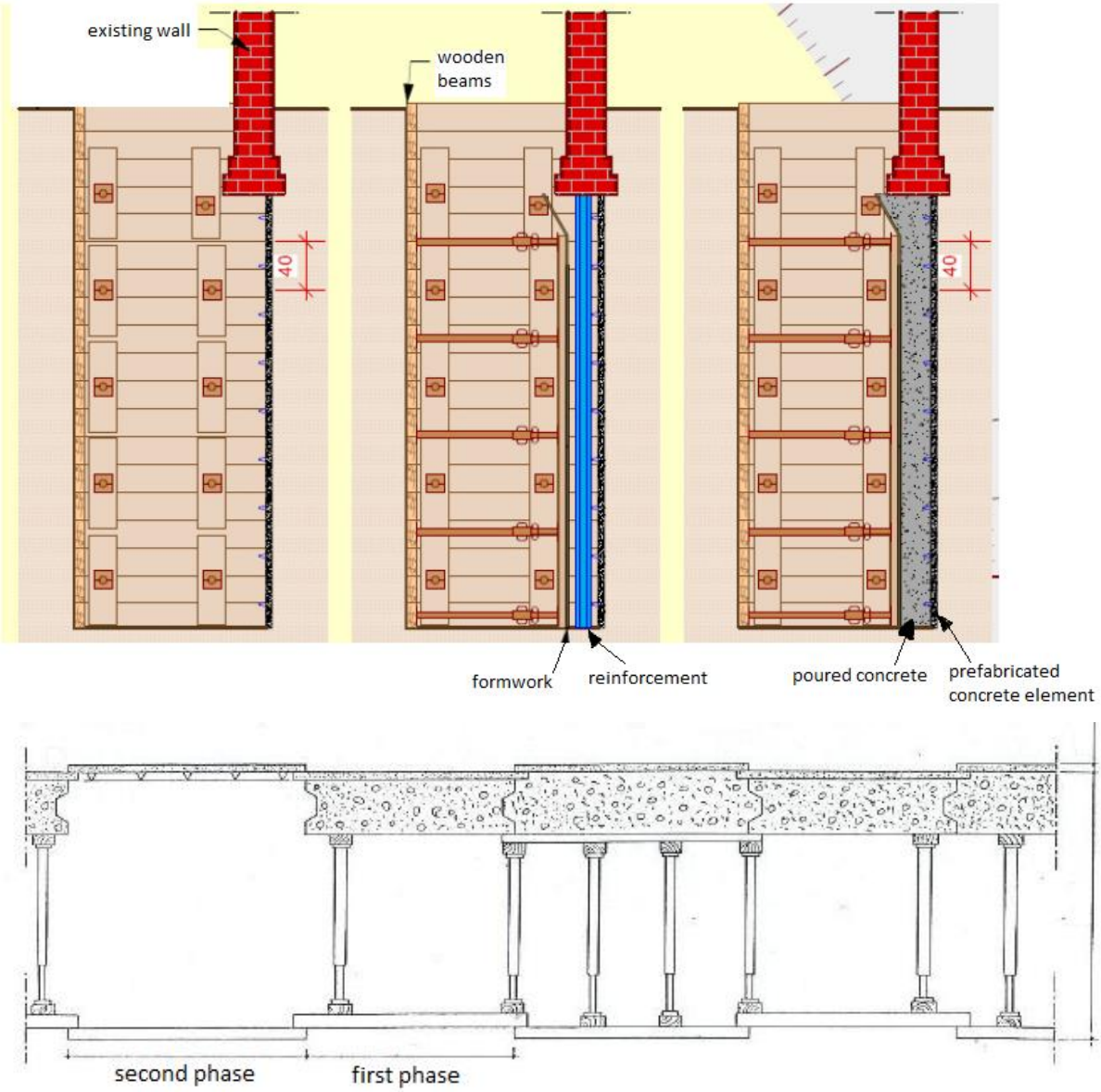

Figure 73: Underpinning in a contained slot (Wylaers M., 2016, Bouwtechniek 1 – Underpinning.)

This method can also be executed in combination with micropiles.

6.1.1 Advantages and disadvantages

The advantages of this technique are very similar to these of Munich walls:

- A new wall can be made almost everywhere and under every existing wall.
- It can be carried out in practically every soil type.
- There is no loss of place in the building pit.
- Manual excavations allow a great freedom of execution.

However, considering the construction pit reaches a depth of more than 10m in certain locations, the safety aspect is very important in this case. In Belgium they still use this technique, usually for underpinning cases of 3-6m, although depths of more than 15m are also possible. Sufficient attention must be paid to dimensioning and implementation, since careless realization can entail major risks. For this reason, the technique hasn't been used in Portugal for a lot of years and especially in this case, the safety of the employees must be guaranteed. On top of that, the execution of this technique would take a long time because it's a manual excavation which exists out of a lot of stages.

6.1.2 Estimated cost for underpinning in a contained slot

Costs for the execution of this technique are estimated in the following table. The cost is € 219,20 per m² of the retaining wall plan view.

Table 12: Estimated costs for underpinning in a contained slot

Element	Quantity	Price/quantity	Total price
Micropiles			
Micropiles Ø127,0x9mm	384,10 m	€ 85/m	€ 32 648,50
Micropiles Ø88,9x9mm	23,70 m	€ 70/m	€ 1 659
Metal cubes supporting micropiles	46	€ 200	€ 9 200
Concrete walls			
Formwork	669,60 m²	€ 20/m²	€ 13 392
Concrete C30/37	123,60 m³	€ 100/m³	€ 12 360
Steel reinforcement	22 879,40 kg	€ 1/kg	€ 22 879,40
Shoring			
Shoring elements	17 156,04 kg	€ 1,4/kg	€ 24 018,46
Total price:			€ 116 157,36

6.1.3 Modelling in PLAXIS 2D

The results found from modelling the Munich walls in PLAXIS are the same as these for the underpinning in a contained slot. Other techniques are used to carry out the work, but the final result is the same. There is no core beam executed in this technique, but this was not modelled in PLAXIS anyway.

6.2 Mini CFA piles

Mini CFA piles can be made out of micro concrete with HEA-profiles and have a smaller diameter as usual. CFA piles are typically installed with diameters ranging from 0,3 to 0,9 m, while the mini CFA piles have a diameter of 250mm and are constructed with the same machine as micropiles. CFA piles are a type of drilled foundation in which the pile is drilled to the final depth in one continuous process using a continuous flight auger. While the auger is drilled into the ground, the flights of the auger are filled with soil, providing lateral support and maintaining the stability of the hole. At the same time the auger is withdrawn from the hole, concrete or a sand/cement grout is placed by pumping the concrete/grout mix through the hollow center of the auger pipe to the base of the auger. Simultaneous pumping of the grout or concrete and withdrawing of the auger provides continuous support of the hole. To create a continuous earth retaining wall with enough stiffness, HEA 160-profiles are placed every two columns.

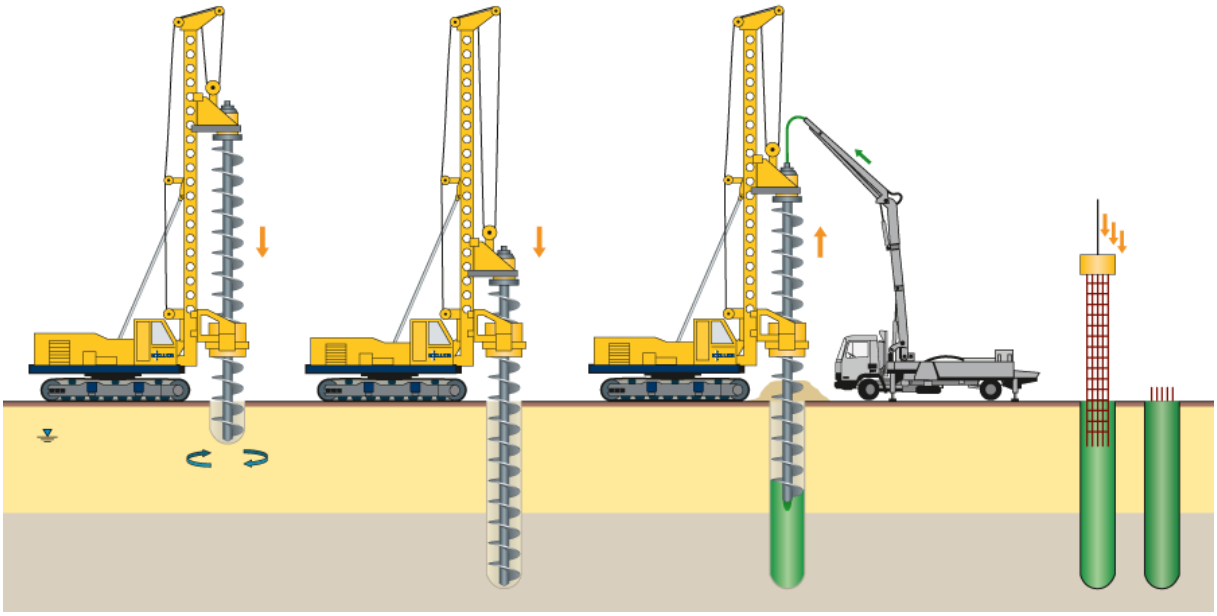

Figure 74: Mini CFA piles (http://www.kellerholding.com/bored-piles.html)

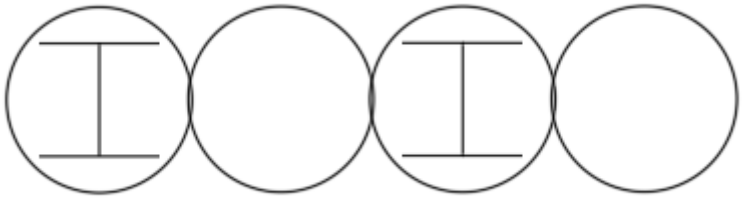

Figure 75: mini CFA piles with HEA 160

6.2.1 Advantages and disadvantages

CFA columns differ from conventional drilled shafts or bored piles, and exhibit both advantages and disadvantages over conventional drilled shafts.

- ✓ Cemented column creation without causing huge ground disturbances (subsoil);
- ✓ Offer a practical and cost-effective solution to costly alternative pile systems as well as a solution to job sites with difficult access;
- ✓ Foundation element for any ground condition.

The main difference is that the use of casing or slurry to temporarily support the hole is avoided. Drilling the hole in one continuous process is faster than drilling a shaft excavation, an operation that requires lowering the drilling bit multiple times to complete the excavation. Handling of spoils can be a significant issue when the soils are contaminated or if limited room is available on the site for the handling of material. Another disadvantage of the CFA piles compared to driven piles is that the available methods to verify the structural integrity and pile bearing capacity for CFA piles are less reliable than those for driven piles.

6.2.2 Estimated cost for mini CFA piles

Costs for this project with mini CFA columns of 250mm are estimated in the following table. The cost of the columns is calculated for a cross section of about 0,2m². The cost is €337,162 per m² of the retaining wall plan view.

Table 13: Estimated costs of mini CFA piles

Element	Quantity	Price/quantity	Total price
Mini CFA piles			
Drilling	2120,66 m	€ 25/m	€ 53 016,5
Concrete C30/37	106,03 m³	€ 100/m³	€ 10 603
H-profiles	82 175,80 kg	€ 1,4/kg	€ 115 046,11
Total price:			€ 178 665,61

6.2.3 Modelling in PLAXIS 2D

For the geometry subroutine the same input can be used as for the Munich type walls. The plates don't have to be divided in different parts, but can be drawn as one line. The values for the mini CFA piles are calculated based on E=78,5 GPa for the steel profiles and E=30 GPa for the concrete and D=250mm, using the following equations. The weight of the piles is based on the mean weight density of HEA160 profiles.

$$EA = 2 \,.\, Esteel \,.\, Asteel + Econcrete \,.\, l \,.\, b \;\; \text{[kN/m]} \qquad [6.2.3.1]$$

$$EI = 2 \,.\, Esteel \,.\, Isteel + Econcrete \,.\, \frac{l \,.\, b^3}{12} \;\; \text{[kNm²/m]} \qquad [6.2.3.2]$$

Table 14: Used parameters for the structural material of mini CFA piles

Parameters	Mini CFA piles
EA [kN/m]	6 608 689
EI [kNm²/m]	22 626,61
w [kN/m/m]	0,31
V	0,3

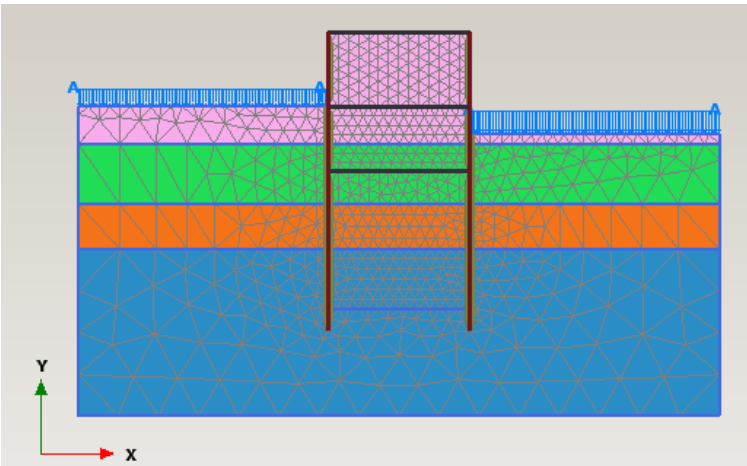

Figure 76: Mesh generation mini CFA piles

The construction of the excavation is not executed in so many phases when using the mini CFA piles. The initial phase is made by default in PLAXIS. The first phase to be defined is the activation of the distributed loads of the neighbouring buildings. After this, the piles are placed vertically on both sides of the excavation by activating the plate. Finally, the first slab can be made and the excavation can continue until the desired depth.

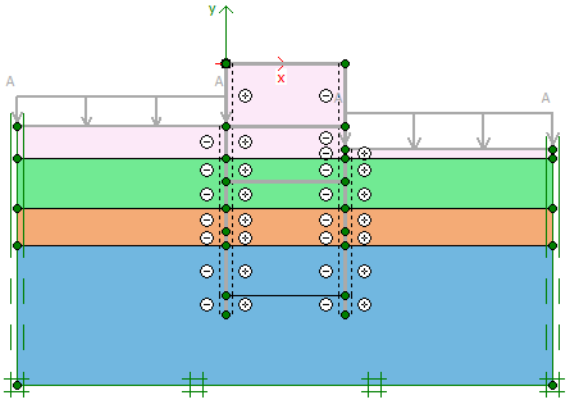

Figure 77: Phase 0: initial phase

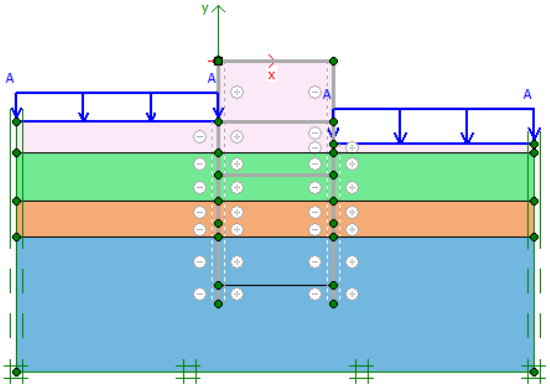

Figure 78: Phase 1: external load

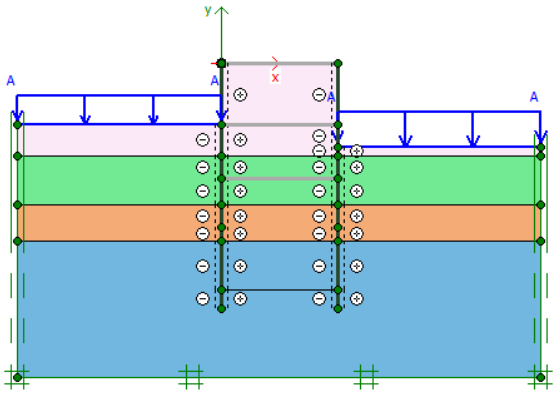

Figure 79: Phase 2: mini CFA piles

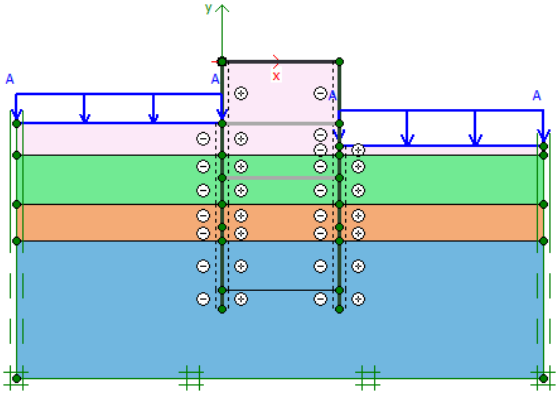

Figure 80: Phase 3: first slab

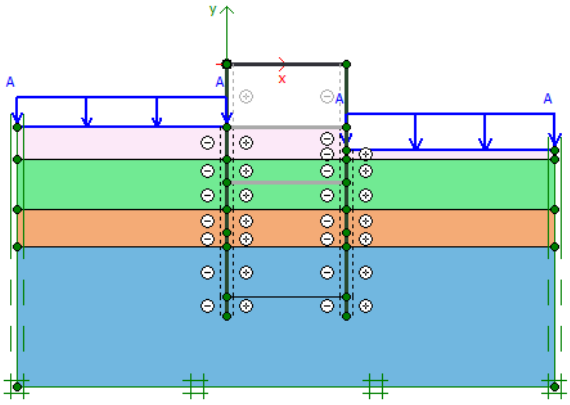

Figure 81: Phase 4: first level excavation (2,86m)

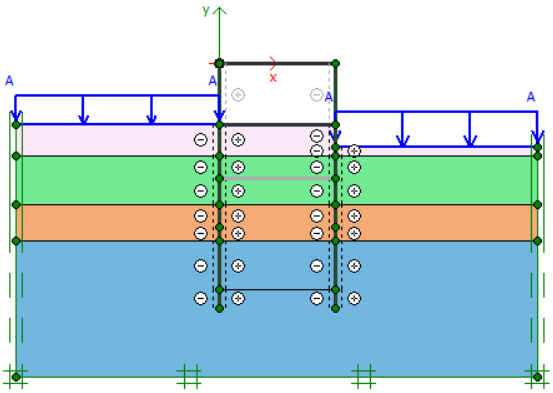

Figure 82: Phase 5: second slab

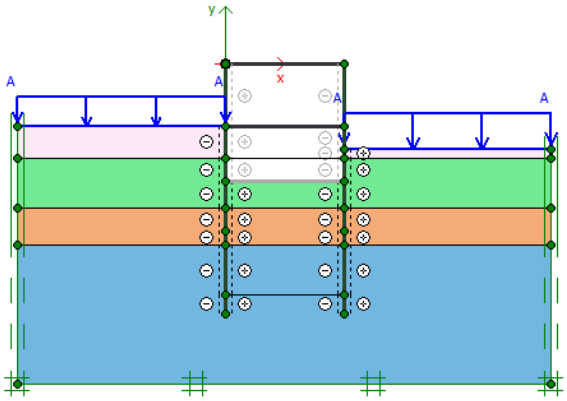

Figure 83: Phase 6: second level excavation (2,74m)

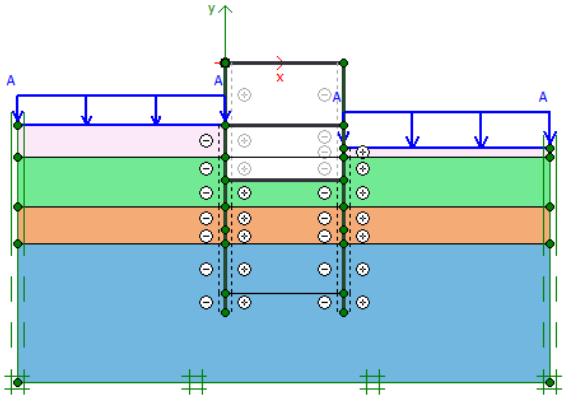

Figure 84: Phase 7: third slab

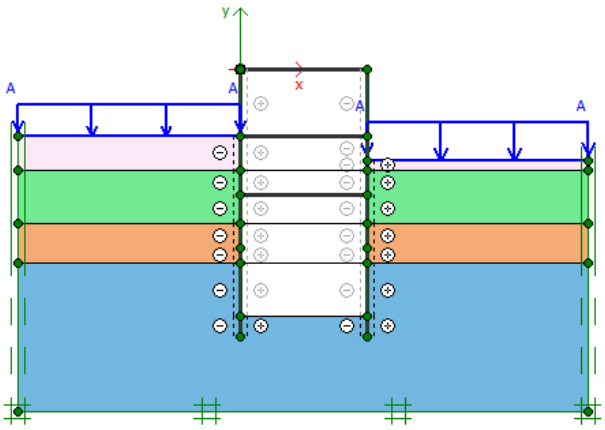

Figure 85: Phase 8: third level excavation (4,95m)

Figure 86 and Figure 87 show the horizontal and vertical displacements of the soil. The criteria for evaluating movements are shown in Table 12.

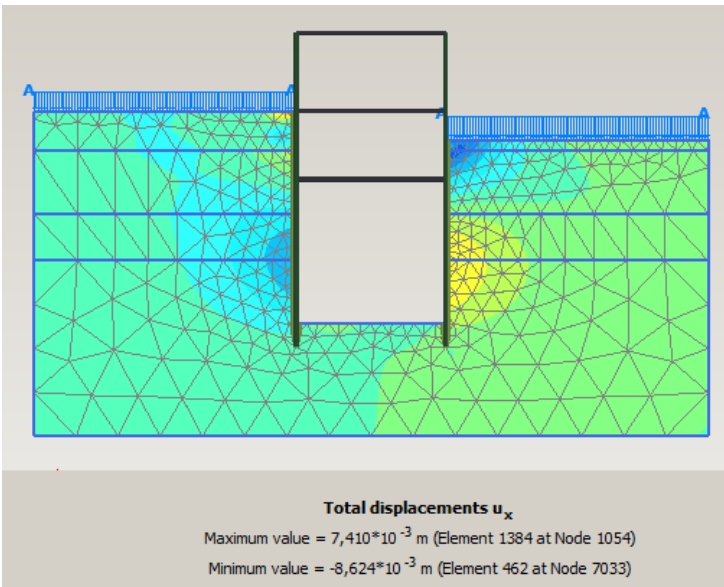

Figure 86: Horizontal displacements u_x mini CFA piles

According to Figure 86, it can be seen that the maximum horizontal displacement occurs behind the pile curtain and has a maximum value of approximately 9 mm in the direction of the interior of the excavation. This is lower than 15mm, the admissible value.

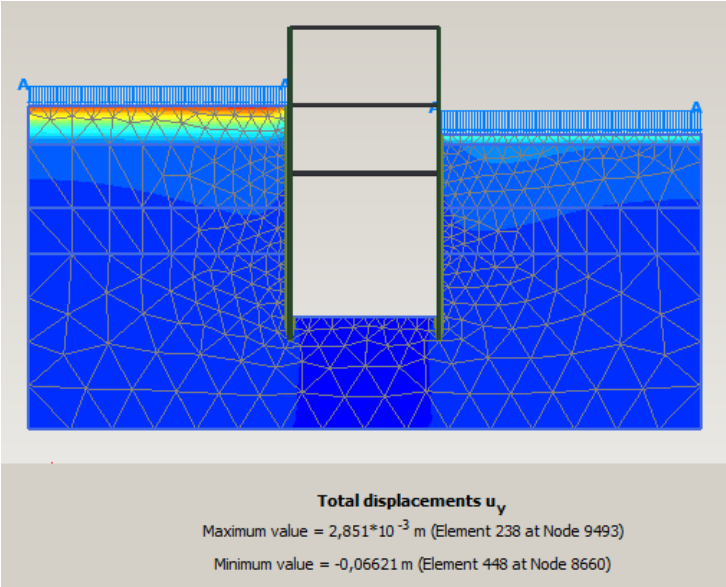

Figure 87: Vertical displacements u_y mini CFA piles

Figure 87 shows that the maximum vertical displacements occur under the neighbouring building number 32, corresponding to settlements of 66 mm. This is again because of the load of the building, not because of the excavation. At the base of the excavation and next to the curtain wall, there is a displacement of about 9 mm. This is lower than 10mm, the admissible value.

6.3 Comparison analysis

To compare the different solutions of the Munich walls, underpinning in a contained slot and mini CFA piles, we have to look at all the properties. An economical comparison is made, the results obtained by PLAXIS have to be compared and also other aspects like the need of specialized employees and technology, safety, the time factor and the ability to save space have to be investigated.

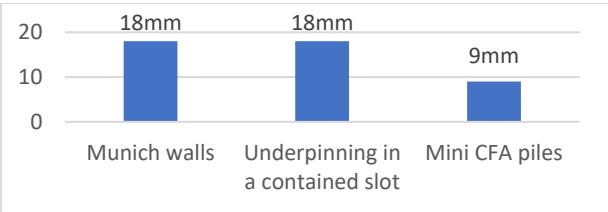

Figure 88: Maximum horizontal displacement of the soil [mm]

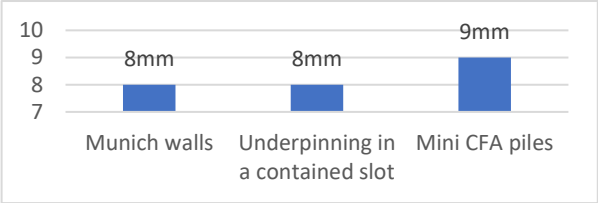

Figure 89: Maximum vertical displacement of the soil [mm]

Figure 88 and 89 show the horizontal and vertical maximum displacements caused by the excavation. All the techniques give a safe solution for the excavation and earth retaining of the basement floors, all values are lower than the alarm criteria.

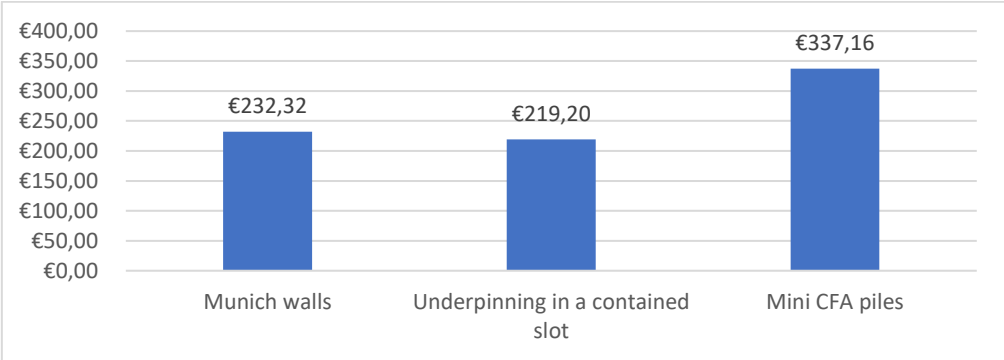

Figure 90: Estimated cost per m² of the retaining wall plan view [€]

The cost per m² of the retaining wall plan view of underpinning in a contained slot is similar to the Munich wall technique, the main difference is the lack of the core beam. The use of mini CFA piles is the most expensive option, because of the HEA-profiles. The cost of the employee's salary is not included in this estimation, but can't be forgotten. The execution of Munich walls or underpinning in a contained slot is a manual excavation and will take a long time, approximately they can make six panels/week. The mini CFA piles is a faster solution, which can be executed in three weeks per level of 3m depth.

Except for the technical and economical comparison, other important aspects have to be taken into account. Values are estimated in % of positive effect in Figure 91, to visualise these properties. This way, the option of underpinning in a contained slot can immediately be neglected, because of safety reasons. Doing a manual excavation in a deep contained slot brings too much risks for the employees. The execution of mini CFA piles enables the safest solution because this is not a manual excavation technique and also allows the excavation to be executed faster. For this case study with a small width of approximately 5m, it's very important to execute the retaining wall against the soil with the least loss of space for the basement. This is the main advantage of the option of the Munich walls, but also for the mini CFA piles this problem can be solved by demolishing a small part of the piles on the inside of the basement.

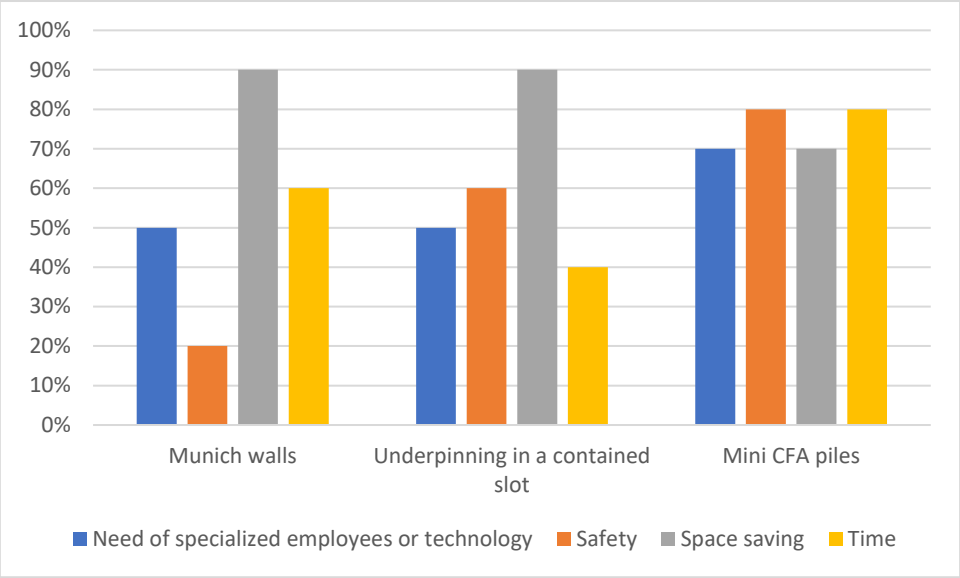

Figure 91: Other aspects [estimated %]

7. Conclusion

7.1 General remarks

To accomplish the objectives, a general historical overview of Lisbon and its rebuilding actions after the Great Lisbon earthquake are described. Through its history, Portugal mainland has experienced the effects of various moderate to strong earthquakes, thus presenting a moderate seismic risk. The geotechnical soil characterization is of the utmost importance for seismic risk assessment, being used, in particular, for site effect assessment. In old cities there is an increasing market of buildings rehabilitation and it is usually necessary to keep the main facades, which implies more difficulties to the earth retaining structures. Allied to this, facade retention solutions must be associated with excavations for basements execution. When King Post walls are structures which consist of metal profiles with between them, profiles of wood or precast concrete, these types of retaining walls are temporary and they are called Berliner walls. Although, when the execution of the walls is a permanent solution that uses reinforced concrete poured in site, supported by micropiles staled at the ground vertically, this technique is called Munich walls. This is a common used technique in Portugal with many advantages and based on the geological, geotechnical and topographic site characteristics one of the easiest techniques to excavate the Jasmin Noir site. Because of the small width of the construction site, the main advantage is the possibility to execute these walls in a small work area and against the soil to save space. An advanced constitutive model for the simulation of the non-linear, time-dependent and anisotropic behaviour of the soils of the site is made in PLAXIS 2D. This finite element program, developed for the analysis of deformation, stability and groundwater flow in geotechnical engineering gives a clear view of the expected displacements. However, experience has shown that these values are considerably higher than those that happen in reality. The results obtained show that the chosen Munich walls technique gives a safe solution for the execution of the basement floors at the Jasmin Noir site. Other solutions are investigated to optimize the work, such as underpinning in a contained slot and mini CFA piles. Both have at least one dominant disadvantage that justifies the choice of the Munich walls. Underpinning in a contained slot was suggested by Belgian companies, but because of safety reasons this method hasn’t been applied for years in Portugal. The use of mini CFA piles is a faster solution but will dissolve in a higher cost. The frequency of use of any engineering technique depends mainly on the technical feasibility and economics of the system. In geotechnical engineering, the more problems a construction technique can solve, and the more soil types in which it is effective, the more applications will be available for the system's use.

7.1 Future developments

The process of the excavation using the top/down-method and the execution of the Munich walls couldn’t be observed because of the short term of the Erasmus exchange. Further follow-up of the ‘Jasmin Noir’ site is advised and monitoring results can be found. A retro-analysis of the final solution could be executed, as an attempt to narrow both the displacement values provided by the instrumentation and the modelling values.

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9. Appendices

Appendix 1: Survey charts test boring S1 and S2

Appendix 2: Geotechnical zones Z1-Z4 and ground water level

Appendix 3: Inspection shafts extra information

Appendix 4: AutoCAD plans ‘Jasmin Noir’ building

tecnasol ELEVOLUTION GROUP SGPS	OBRA: EDIFÍCIO NA PRAÇA DO PRÍNCIPE REAL, 33 CLIENTE: ATTITVDE construção e manutenção LOCALIZAÇÃO: Lisboa	SONDAGEM S1 OBRA Nº 19.4033

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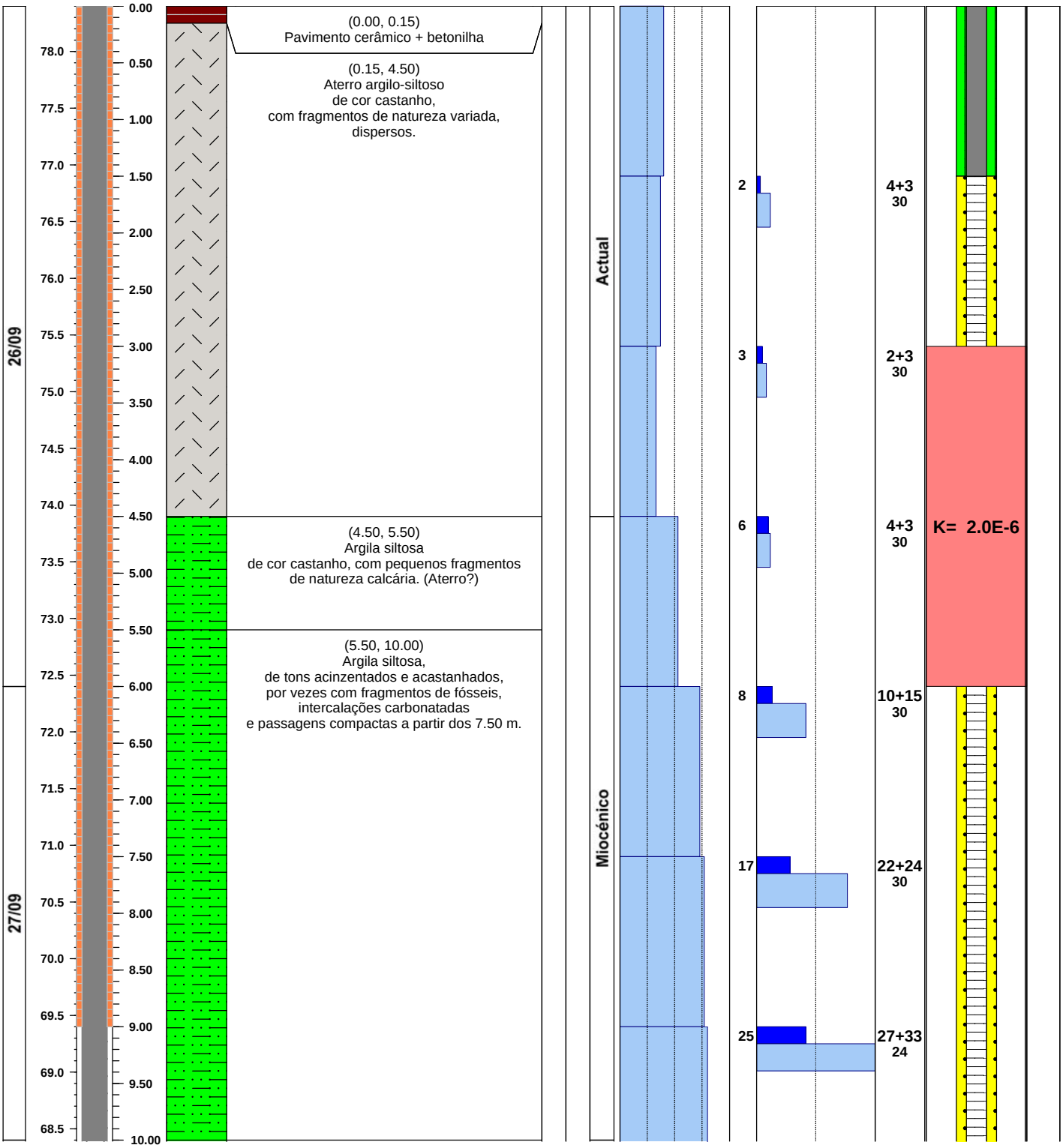
FURAÇÃO 0.00 m - 15.00 m= 86mm	REVESTIMENTO 0.00 m - 9.00 m= 98mm	NÍVEL DE ÁGUA DETECTADO: ESTABILIZADO: 12.46	Des. Set/17 CMG Ver. Set/17 CPR
EQUIPAMENTO CLIVIO	INICIO: 26/09/2017 FIM: 28/09/2017		Pág. 1 de 2



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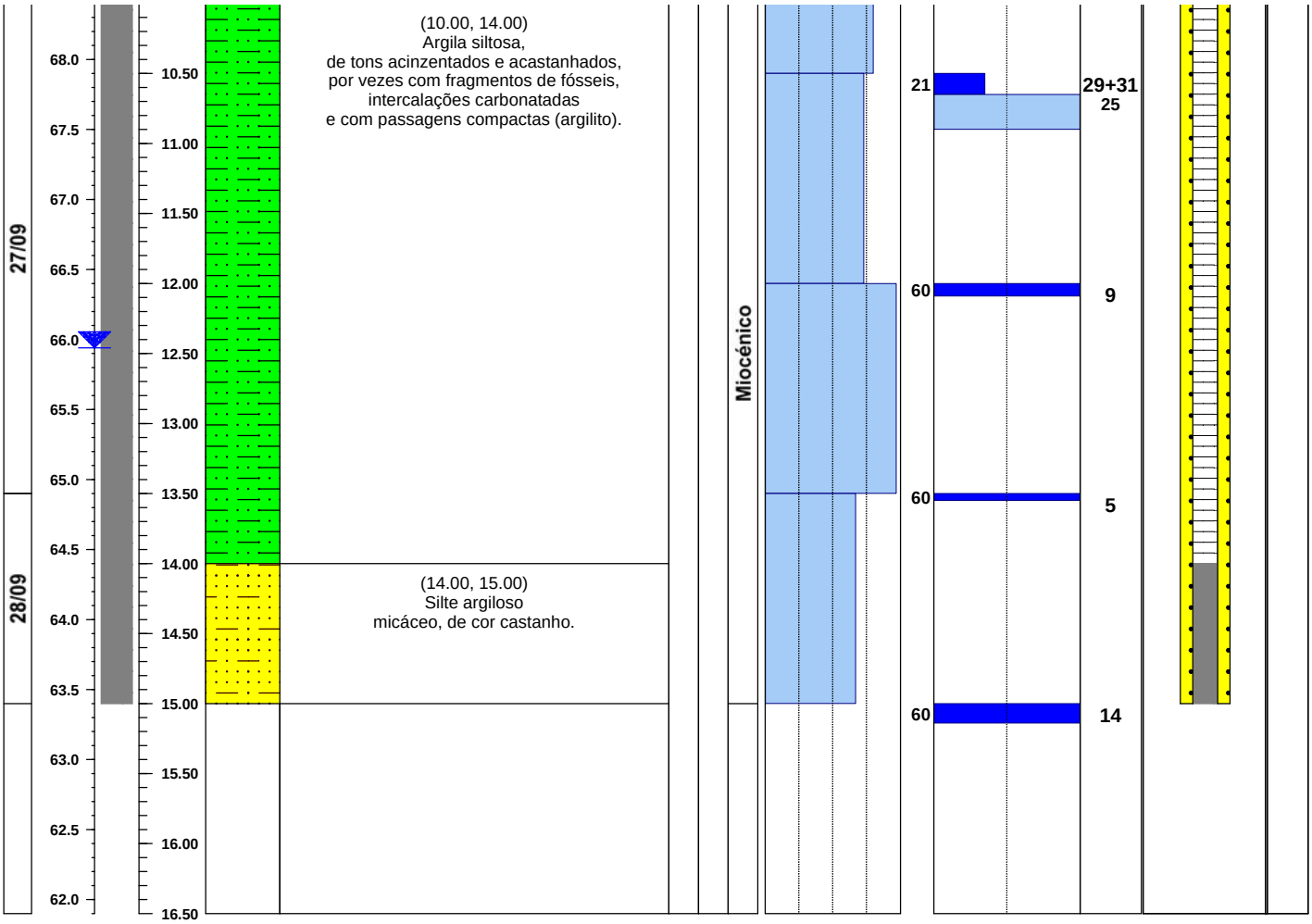
EQUIPAMENTO CLIVIO	INICIO: 26/09/2017 FIM: 28/09/2017		Pág. 2 de 2



OBSERVAÇÕES:

EDIFÍCIO NA PRAÇA DO PRÍNCIPE REAL N°33

Sondagem S1

0.00 m – 6.50 m

6.50 m – 10.75 m

10.75 m – 15.00 m

tecnasol ELEVOLUTION GROUP SGPS	OBRA: EDIFÍCIO NA PRAÇA DO PRÍNCIPE REAL, 33 CLIENTE: ATTITVDE construção e manutenção LOCALIZAÇÃO: Lisboa	SONDAGEM S2 OBRA Nº 19.4033

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EQUIPAMENTO CLIVIO	INICIO: 03/10/2017 FIM: 05/10/2017		Pág. 1 de 2

DATAS	COTA	DIÂMETROS	PROF. (m)	SIMBOLOGIA	DESCRIÇÃO De acordo com os critérios definidos pela Classificação Triângular de Solos [LNEC E-219] [LNEC E-239]	ALTERAÇÃO	FRACTURAÇÃO	ESTRATIGRAFIA	PERCENTAGEM DE RECUPERAÇÃO / ÍNDICE RQD (0 % 100)	ENSAIOS E AMOSTRAGEM: ENSAIO SPT (1ª Fase; Nº de pancadas (N) 0 60; 2ª e 3ª Fase Penet. (cm))	Piezómetro/ Ensaios Lefranc (K= m/s)	Z. GEOTÉCNICAS
03/10	72.0		0.00		(0.00, 0.15) Pavimento em lajetas de calcário			Actual				
	71.5		0.50		(0.15, 2.50) Aterro argilo-siltoso, de tons acastanhados, com fragmentos de natureza variada, dispersos.							
	71.0		1.00									
	70.5		1.50							7 / 6+9 30		
	70.0		2.00								K= 4.5E-8	
	69.5		2.50		(2.50, 4.50) Argila siltosa, de tons acinzentados, com laivos acastanhados, pontualmente com pequenos fragmentos de natureza variada, dispersos.			Miocénico				
	69.0		3.00							10 / 10+12 30		
	68.5		3.50									
	68.0		4.00									
04/10	67.5		4.50		(4.50, 8.20) Argila siltosa levemente margosa, de tons acinzentados, com intercalações carbonatadas e presença de conchas.					15 / 17+19 30		
	67.0		5.00									
	66.5		5.50									
	66.0		6.00							6 / 10+13 30		
	65.5		6.50									
	65.0		7.00									
	64.5		7.50							11 / 14+18 30		
	64.0		8.00									
	63.5		8.50		(8.20, 10.00) Argila siltosa de tom acastanhado e acinzentados.							
	63.0		9.00							19 / 29+31 25		
	62.5		9.50									
			10.00									

tecnasol ELEVOLUTION GROUP SGPS	OBRA: EDIFÍCIO NA PRAÇA DO PRÍNCIPE REAL, 33 CLIENTE: ATTITVDE construção e manutenção LOCALIZAÇÃO: Lisboa	SONDAGEM S2 OBRA Nº **19.4033**

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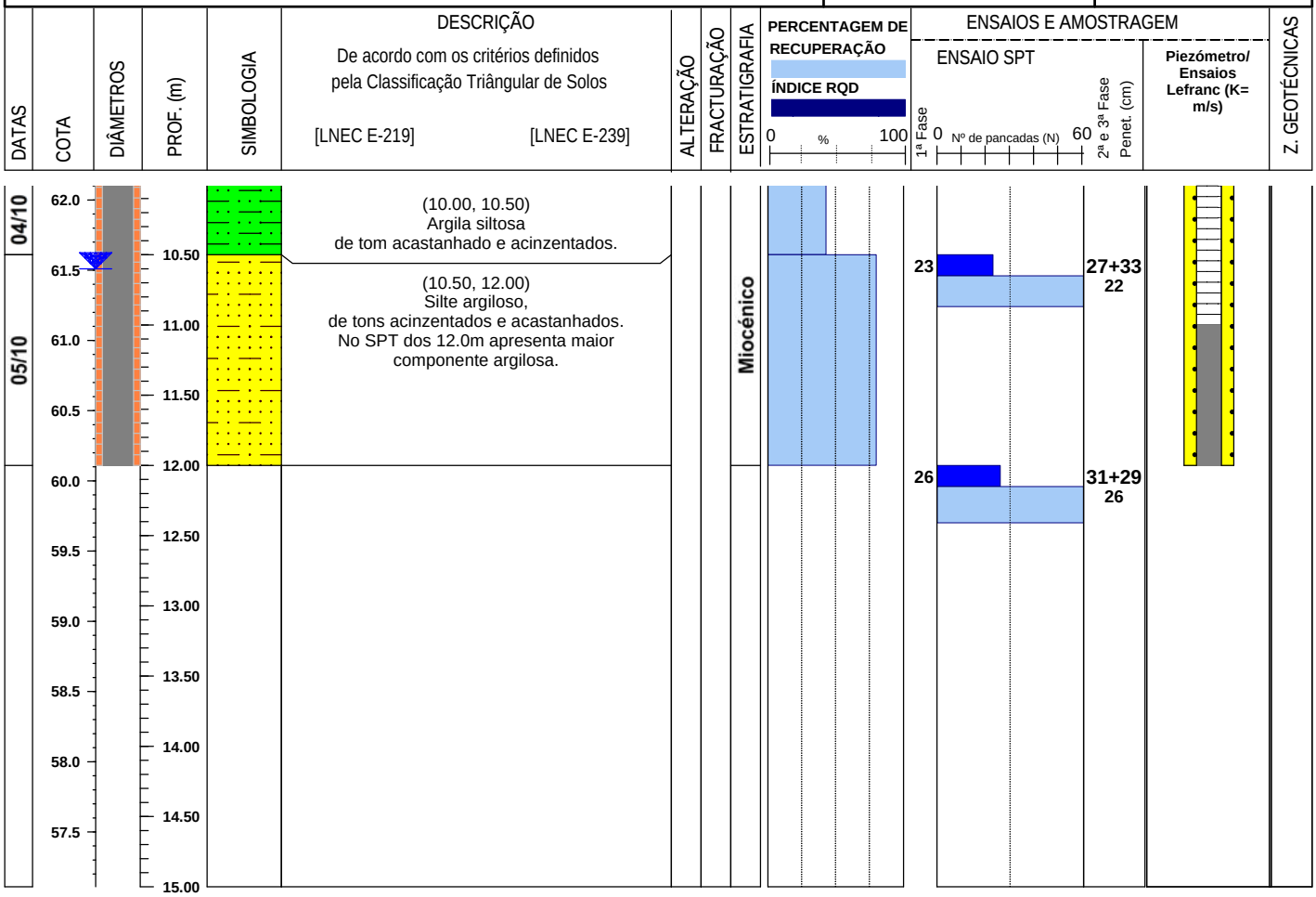
NÍVEL DE ÁGUA: DETECTADO: ; ESTABILIZADO: 10.6

Des. Out/17 CMG
Ver. Out/17 CPR

EQUIPAMENTO CLIVIO INICIO: 03/10/2017 FIM: 05/10/2017



EDIFÍCIO NA PRAÇA DO PRÍNCIPE REAL N°33

Sondagem S2

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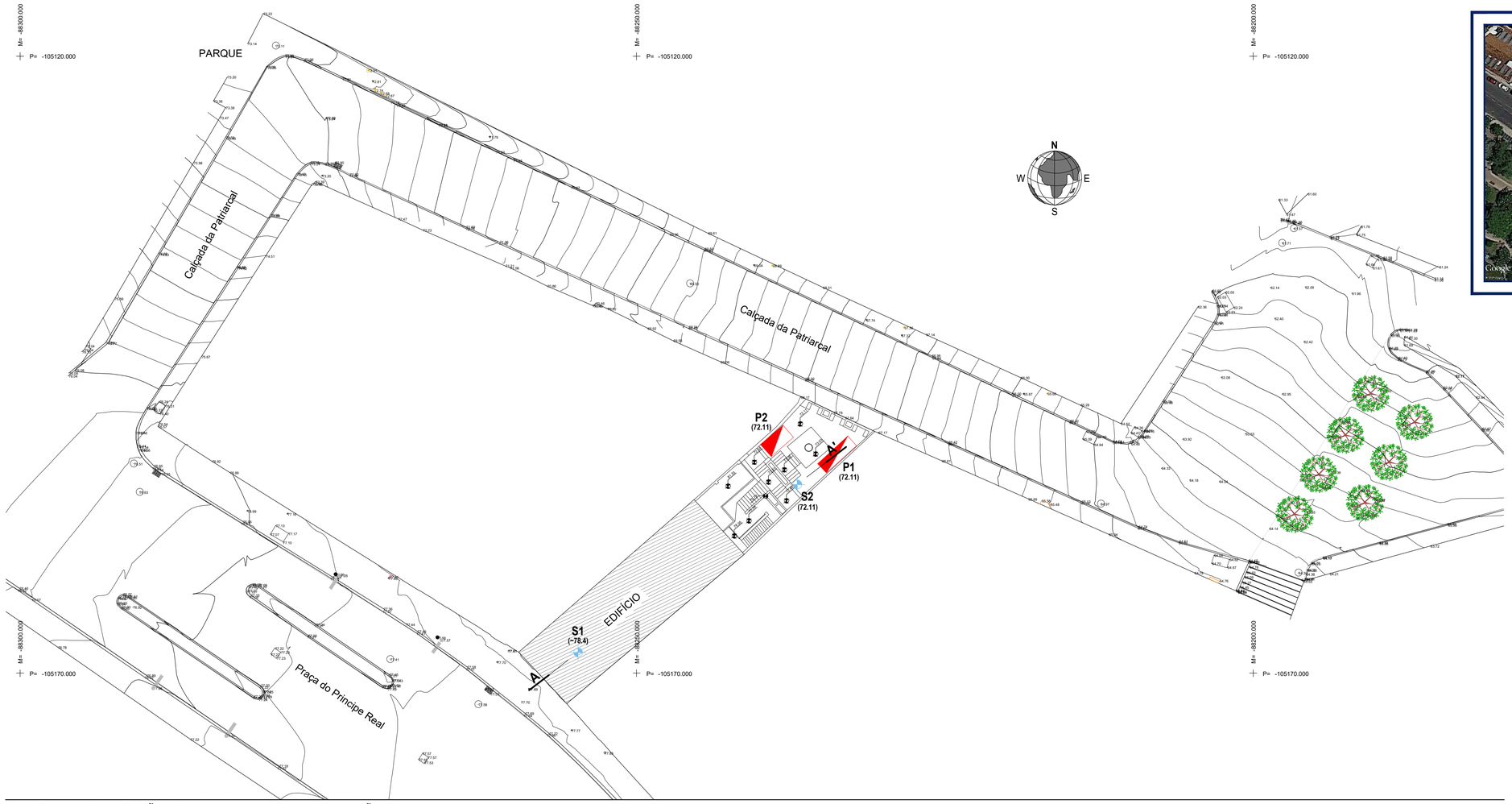
6.50 m – 12.00 m



PLANTA DE LOCALIZAÇÃO DOS TRABALHOS DE PROSPECÇÃO

Escala 1/200

FOTOGRAFIA AÉREA

QUADRO DE COORDENADAS E COTAS DAS SONDAGENS

SONDAGENS	COORDENADAS		COTA
	M	P	Z (m)
S1	-88255	-105168	~78.4
S2	-88237	-105155	72.11

Sistema de coordenadas: Datum 73
Ponto de Origem: Ponto Central da Melriça

QUADRO DE COORDENADAS E COTAS DOS POÇOS

POÇOS	COORDENADAS		COTA
	M	P	Z (m)
P1	-88234	-105152	72.11
P2	-88239	-105151	72.11

Sistema de coordenadas: Datum 73
Ponto de Origem: Ponto Central da Melriça

LEGENDA:

PLANTA

S...
(00.00)
Sondagens com Piezómetros executadas pela empresa "Tecnasol FGE - Fundações e Geotecnia, S.A.", entre setembro e outubro de 2017

P...
(00.00)
Poços executados pela empresa "Tecnasol FGE - Fundações e Geotecnia, S.A.", em setembro de 2017

A
Perfis Interpretativos

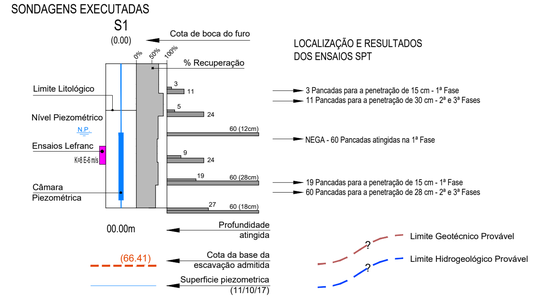
LITOLOGIA

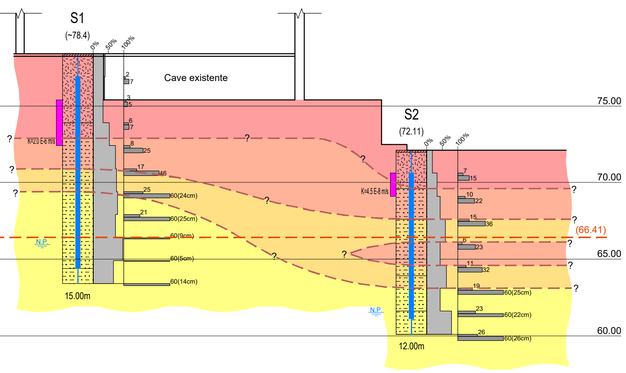
Actual
- Pavimento
- Lajetas de calcário
- Argamassa cimenticia

Actual
- Aterro
- Elementos de fundação constituidos por calhaus e blocos envoltos por argamassa
- Blocos de calcário

Miocénico
- Argila siltosa/Silte argiloso

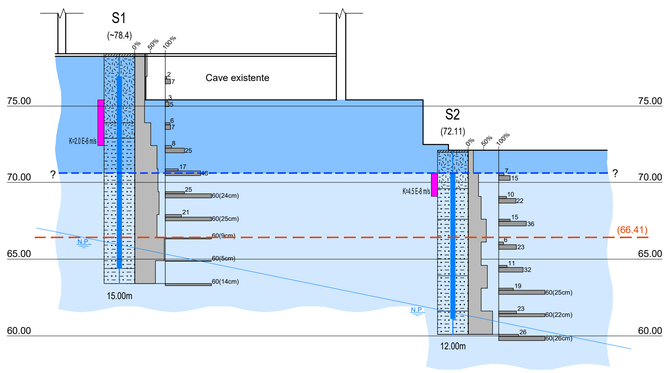
SONDAGENS EXECUTADAS





PERFIL INTERPRETATIVO A-A'

PERFIL GEOLÓGICO
Escala 1/200



PERFIL INTERPRETATIVO A-A'

PERFIL HIDROGEOLÓGICO
Escala 1/200

QUADRO SÍNTESE

Zona Geotécnica		Descrição	N_{SPT}	Peso específico γ (kN/m³)	Ângulo de atrito interno Ø' (°)	Coesão C' (KPa)	Módulo de deformabilidade E' (MPa)
	ZG4	Pavimento e aterros	5, 7 e 15	10 - 14	15 - 20	0	2.5* 3.5**
	ZG3	Argilas siltosas	22, 23 e 25	19 - 21	30 - 33	22 - 50	16.5 - 18.7* 23 - 26**
	ZG2	Argilas siltosas, por vezes ligeiramente margosas	32, 36 e 46	20 - 21	34 - 36	50 - 110	24 - 34* 33 - 48**
	ZG1	Argilas siltosas e siltes argilosos	> 60	21 - 22	35 - 40	100 - 150	45* 60**

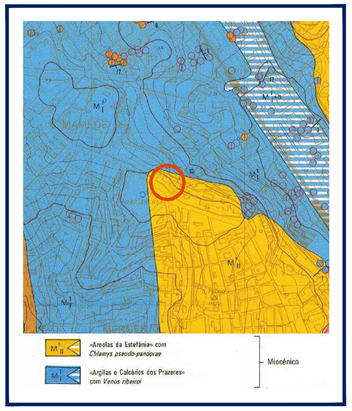
* - Carregamento axissimétrico
** - Deformação plana

QUADRO HIDROGEOLÓGICO SÍNTESE

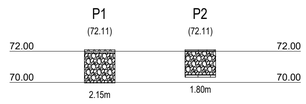
Unidade Hidrogeológica		Permeabilidade	Descrição	Permeabilidade K(m/s)
	UH2	Baixa	Aterro e argila siltosa	10 E-6
	UH1	Muito baixa	Aterro, argila siltosa e silte argiloso	10 E-8 - 10 E-9

EXTRACTO DA CARTA GEOLÓGICA DO CONCELHO DE LISBOA, Folha 4

(Escala original: 1/10 000)



Área em estudo



REPRESENTAÇÃO DOS POÇOS DE RECONHECIMENTO

Escala 1/200

Indice	Alteração	Data	Desenhado	Verificado	Validado

tecnasol
Edifício Prime - Av. Quinta Grande, 53 e 53A - 4º B
Alfragide - 2610-156 Amadora
Telef: +351 214 759 063
Fax: +351 214 759 500
Site: www.elevogroup.com/pt/empresa/tecnasol/
e-mail: geral.tecnasol@tecnasol.com

ATTITVDE

EDIFÍCIO NA PRAÇA DO PRÍNCIPE REAL, 33 / CALÇADA DA PATRIARCAL, 15
LISBOA

ESTUDO GEOLÓGICO-GEOTÉCNICO E HIDROGEOLÓGICO

PLANTA DE LOCALIZAÇÃO DOS TRABALHOS DE PROSPECÇÃO, PERFIS INTREPRETATIVOS E REPRESENTAÇÃO DOS POÇOS DE RECONHECIMENTO

Proj./Rev.	Carlos Gonçalves		
Des.	Rui Silva		
Ver.	Paulo Rodrigues		
Val.	Paulo Rodrigues		
Nome do ficheiro: 170416001.dwg	Data: out.2017	Escala: 1:200	Desenho Nº P17/0416-4033/001/0/11703

A1 (594mm x 841mm)

tecnasol ELEVOLUTION GROUP SGPS	OBRA: EDIFÍCIO NA PRAÇA DO PRÍNCIPE REAL, 33 / CALÇADA DA PATRIARCAL, 15 CLIENTE: ATTITUDE construção e manutenção LOCALIZAÇÃO: LISBOA	POÇO/VALA Poço P1 OBRA Nº **19.4033**

M: -88234	P: -105152	Z: 72.11	AZIMUTE:	PROFUNDIDADE ATINGIDA **2.15 m**	POÇO / VALA	Proj. Nº
ABERTURA MANUAL				NÃO ENTIVADO	NÍVEL DE ÁGUA	Des. SET/17 CMG Ver. SET/17 CPR
OBSERVAÇÕES: POR QUESTÕES DE SEGURANÇA NÃO FOI POSSÍVEL CONTINUAR A ESCAVAÇÃO DO POÇO.				INICIO: 25/09/2017 FIM: 28/09/2017	DETECTADO: ESTABILIZADO:	Pág. 1 de 1

DATAS	COTA	PROF. (m)	SIMBOLOGIA	DESCRIÇÃO De acordo com os critérios definidos pela Classificação Triângular de Solos [LNEC E-219] [LNEC E-239]	ESTRATIGRAFIA	AMOSTRAGEM ENSAIOS	AMOSTRAGEM
	72.0	0.00		(0.00, 0.10) Lajetas em calcário			
	71.5	0.50		(0.10, 0.20) Argamassa cimenticia de regularização, friável.			
	71.0	1.00		(0.20, 2.15) Elemento de fundação constituido por calhaus e blocos (Dmáx.= 40/50 cm), envoltos por argamassa cimentícia. A argamassa apresenta-se compacta. Aos 0.80 m de profundidade há um ligeiro ressalto, afastado cerca de 10/15 cm em relação à prumada da parede de fundação (sapata/viga de fundação ?).			
	70.5	1.50					
	70.0	2.00					
	69.5	2.50					
	69.0	3.00					
	68.5	3.50					
	68.0	4.00					
	67.5	4.50					
		5.00					

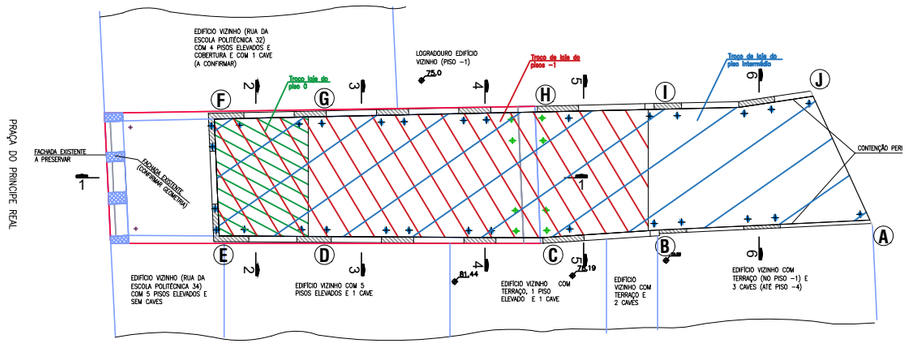
tecnasol ELEVOLUTION GROUP SGPS	OBRA: EDIFÍCIO NA PRAÇA DO PRÍNCIPE REAL, 33 / CALÇADA DA PATRIARCAL, 15 CLIENTE: ATTITVDE construção e manutenção LOCALIZAÇÃO: LISBOA	POÇO/VALA Poço P2 OBRA Nº **19.4033**

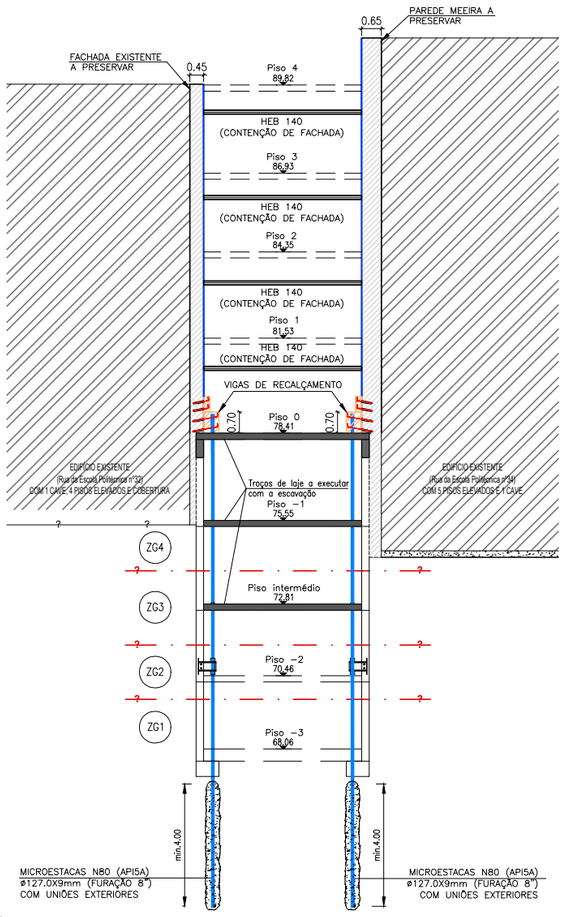
M: -88239	P: -105151	Z: 72.11	AZIMUTE:	PROFUNDIDADE ATINGIDA **1.80 m**	POÇO / VALA	Proj. Nº

ABERTURA MANUAL	NÃO ENTIVADO	NÍVEL DE ÁGUA	Des. SET/17 CMG Ver. SET/17 CPR
OBSERVAÇÕES: DEVIDO À EXISTÊNCIA DE UM SISTEMA DE CANALIZAÇÃO EM GRÉS CERÂMICO DE ÁGUAS RESIDUAIS, NÃO FOI POSSÍVEL CONTINUAR COM A ESCAVAÇÃO DO POÇO	INICIO: 28/09/2017 FIM: 29/09/2017	DETECTADO: ESTABILIZADO:	Pág. 1 de 1

DATAS	COTA	PROF. (m)	SIMBOLOGIA	DESCRIÇÃO De acordo com os critérios definidos pela Classificação Triângular de Solos [LNEC E-219] [LNEC E-239]	ESTRATIGRAFIA	AMOSTRAGEM ENSAIOS	AMOSTRAGEM
	72.0	0.00		(0.00, 0.07) Argamassa cimentícia			
	71.5	0.50		(0.07, 0.15) Aterro arenoso acastanhado e acinzentado.			
	71.0	1.00		(0.15, 1.62) Elemento de fundação constituido por seixos, calhaus e blocos (Dmáx.= 30 cm), envoltos por argamassa, que se apresenta compacta. Este elemento de fundação apresenta uma espessura entre os 0.20 m (limite norte) e 1.50 m (limite sul), que acompanha a inclinação das estruturas encontradas (antigo caneiro e canalização em grés cerâmico)			
	70.5	1.50					
	70.0	2.00					
	69.5	2.50		(1.62, 1.82) Blocos de calcário com comprimentos a variar entre 0.50 m e 1.05 m, argamassados entre si (a argamassa apresenta-se friável e localmente inexistente). Esta estrutura deverá pertencer a um antigo caneiro de circulação de águas residuais, que foi substituido por canalização mais recente em grés cerâmico.			
	69.0	3.00					
	68.5	3.50					
	68.0	4.00					
	67.5	4.50					
		5.00					

PLANTA DE LOCALIZAÇÃO:





CORTE TIPO 2-2
(ALÇADOS DE/FG)
ESCALA 1:100

FORMATO A3 OBTIDO POR REDUÇÃO DO ORIGINAL EM A1

LEGENDA:

Contenção tipo "Berlim Definitivo"

- MICROESTACAS N80 (API5A) ø127.0X9MM (FURAÇÃO 8") COM UNIÕES EXTERIORES
- MICROESTACAS N80 (API5A) ø177.8X9MM (FURAÇÃO 10") COM UNIÕES EXTERIORES
- MICROESTACAS N80 (API5A) ø88.9X9MM (FURAÇÃO 8") COM UNIÕES EXTERIORES, COMPRIMENTO DE SELAGEM=4.0M
- BARRAS GEWI
- VIGA DE RECALÇAMENTO
- VIGA DE COROAMENTO
- PAINÉIS PRIMÁRIOS
- PAINÉIS SECUNDÁRIOS
- ESCORAS METÁLICAS
- ARRANQUES DOS PILARES (VER PROJETO DE ESTRUTURAS)

ZONAS GEOTÉCNICAS

- ZG4 RECENTES (ATERROS)
- ZG3 ARGILAS SILTOSAS (N_{SPT}<25)
- ZG2 ARGILAS SILTOSAS, POR VEZES LIGEIRAMENTE MARGOSAS (32<N_{SPT}<46)
- ZG1 ARGILAS SILTOSAS E SILTES ARGILOSOS (N_{SPT}>60)
- LIMITE GEOTÉCNICO PROVÁVEL

Instrumentação

- AL... ALVOS TOPOGRÁFICOS
- I... INCLINÓMETROS (INCLUINDO REALIZAÇÃO DE SONDAGEM COM AMOSTRAGEM CONT ENSAIOS SPT, NO INTERIOR DOS FUROS PARA INSTALAÇÃO DAS CALHAS)
- M... MARCA DE NIVELAMENTO DE SUPERFÍCIE (A COLOCAR NO PASSEIO)
- PZ... PIEZÓMETROS

Alt.	Data	Designação	Des.	Proj.	Verf.
-	-	-	-	-	-

JET SJ

CLIENTE
FCM

PROJECTO
EDIFÍCIO DE HABITAÇÃO
PRAÇA DO PRÍNCIPE REAL, Nº33. LISBOA

ESPECIALIDADE
ESCAVAÇÃO E CONTENÇÃO PERIFÉRICA

FASE
PROJETO DE EXECUÇÃO

DESIGNAÇÃO
CORTES TIPO (2/3)

ESCALAS 1:100(A1)

DESENHO 05

VERIFICOU Alexandre Pinto | PROJETOU A. Pereira/J. Mirante | DESENHOU Jorge Gomes

DATA 2018/03/26 PROJETO PRO/2016/053

REVISÃO 0

N.º Arquivo JetSJ: final plans.dwg

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A1