Urban Tunnels Excavation Project Using
Mechanized Tunnelling Technology

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Abstract

Nowadays, there are already more than 3 billion people living in urban areas and it is expected that in 30 years this number will increase to more than the double, i.e., to 6.3 billion people lining in cities. So the cities of today are under a serious pressure to respond to all the demands that this brutal increase requires. However, the surface is already too crowded with buildings and existing structures, so the solution often is to build underground structures, in order to allow the improvement of infrastructures. This trend is increasing as well as the pressure of the modern societies for better ways of life. These facts put tunnels excavations on the agenda of almost all big and middle cities around the world.

In this thesis, it was studied the most popular technique for long tunnels, the mechanized tunnelling technique, through the perspective of the geotechnical project. The four main subjects addressed were the state of the current technology, the feasibility analysis for a preliminary stage of a project, the impact that a tunnelling boring machine operation has on the existing structures in historic cities, or in other words, in century buildings, metropolitan tunnels with more than 50 years old, being also studied possible solutions to mitigate this impacts. To do all this, it was used as reference the Plano Geral de Drenagem de Lisboa 2016-2030 case study.

Keywords: tunnelling boring machine, geology, settlements, finite elements, numerical modelling

1. Introduction

With the increase of human population in urban areas and with the high demands when it comes to quality of life of the modern societies, the cities of today are under a huge pressure to enhance their infrastructures. But due to the limited space at the surface, the decision makers often resort to underground space to fulfil the required new demands. Therefore, the underground space of the European modern cities today already have a significant number of structures like metro tunnels, water tunnels, pipes and others, which means that a new project in underground space is currently a problem and a big challenge, with several difficulties.

To promote a better understanding of these difficulties and challenges this work focus its attention on tunnel excavation projects in urban areas resorting to mechanized tunnelling technology in a real case study.

2. Mechanized Tunnelling

To describe the state of the art of this technology, it is important to briefly explain which is the traditional scope of application and the work principles of all kind of TBMs.
Briefly Hard Rock TBM or Gripper TBM works through the principle that the cavity open in the geomaterial is stable, so these kind of machines do not have a complete shield and they take advantage of the rock strength to push the TBM forward with the grippers system, which basically are hydraulic jacks pushed against the tunnel walls.

Single shield TBMs (SS), on the other hand, are TBMs conceived to excavate soft rock, where a high degree of weathering is present and the strength of the rock does not allow the use of the gripper system. So these machines have a complete shield and the way that cutter head excavates is turning against the tunnel front with the help of hydraulic jacks that are pushing it against the lining, instead of the tunnel walls. The lining in this conditions is made of reinforced concrete segment rings.

The excavation in soil, in its turn, follows even a different principle due to the characteristics of these materials. In soils, the cavity and the front are assumed instable, so to operate under this conditions the mechanized tunnelling industry developed TBMs that apply a pressure to the front and thus avoid the chimney collapse mechanism, which consists on an over excavation in the tunnel front that lead to big surface settlements ahead of the front. Obviously, this machines have a complete shield and the way that they advance is the same as Single Shields, due to the mentioned instability of the tunnel walls.

So the big difference between an EPB shield and a Slurry Shield (STM) is how these machines apply the counter pressure.

But before this explanation, it is worth to mention that both of these machines are able to apply the counter pressure due to a chamber behind the cutter head that is called excavation chamber. In STMs, this chamber is filled with bentonite slurry and, on the other hand, EPBs fill it with the excavated material.

Traditionally, the reason for these two ways of applying counter pressure is to overcome the different behaviour and permeability of different types of soils – coarse grained soils and fine grained soils. It is possible to say that the STMs were developed for the first type and EPB for the second one. But due to the development of the soil conditioning techniques, nowadays both of these tunnelling boring machines can be applied in both scenarios, even though the STM is still more appropriate to coarse grained soils and EPB have their limitations in materials like gravel and rock fill under phreatic level. For example, EPBs still need grounds which have 5% of fines minimum, a permeability coefficient less than $10^{-5}$ m/s as well as a water pressure smaller than 3 bars (Galli & Thewes, 2014). In contrast, STMs in materials like clay and milt need a big separation effort between the renewable bentonite slurry and the muck and also severe anti-clogging measures might need to be taken, like soil conditioning. All of these will be reflected in a probable very low performance of this TBMs in these grounds (Lamanna, 2016).

Apart from soil conditioning techniques, the industry developed another strategy to have a TBM able to excavate any material, or in other words, to achieve the universal tunnelling boring machine. As described until now, to excavate a tunnel that goes through rock and soils it would be necessary two different machines or doing big mechanical transformations in the TBM inside the tunnel. To avoid this, manufacturers conceived what is called hybrid TBMs, which means having a TBM with more than one TBM type in the same machine. Of course to do so the lining applied must be versatile, that is to say have to be reinforced concrete segments.

The first TBM developed under this strategy was the Double Shield TBM or, for some, universal TBM for rock materials. The two principles at stake are the Gripper TBM and the Single Shield TBM.

Basically, the Double Shield TBM has a complete shield that has 3 independent parts and can operate either by gripper system or by what is called in this work by Singe Shield system. From the front to the tail, the three shield parts are: first the gripper shield, which has two grippers that are responsible to push the cutter head forward in bad rock materials; the second part is the telescope shield; and the third part is the tail shield that is similar
to a SS. The unique advantage of this machine is its ability to excavate and build the lining at the same time. This is accomplished by the telescope shield, because it has the function to contract and extend the global shield, which makes the gripper shield independent – responsible for the excavation – from the tail shield that is responsible for the protection of the segment erector, or in other words, for the construction of the lining. Note that this achievement is only possible in hard rock due to the fact that not all kind of rocks have the strength to support the cutter head advance and when they do not, this TBM works as a normal SS, because in the tail void are also present the hydraulic cylinders ready to require the lining as the support middle for the advance of the excavation.

To mix the other principles, as for example EPB-SS or STM-SS, is also possible and with that the TBM is prepared to excavate rock and soil with pressurized front. In fact, these kind of solutions already have some time except having in the same tunnelling machine an EPB and a STM with a diameter less than 9 m. This accomplishment was reached recently by Herrenknecht, between 2013 and 2015, in Kuala Lumpur, to build the blue line of the respective metropolitan. The conceived machine has on its own physiognomy tools to operate not in two but in four different modes: Traditional Slurry type for high water pressure in coarse soils; STM type with dense slurry when the void ratio of the soils is too high for traditional bentonite slurry; EPB with slurry circuit for situations under phreatic level and fine soils with high permeability; and also the conventional mode EPB. As one can see, this is a real universal TBM for soils and even it is not available in the manufactures brochure as a current solution it is a good example of what can be done nowadays in this industry.

It is true that the universal TBM is a goal that has not been achieved yet but the state of the art of mechanized tunnelling technology for big diameters (D>5 m) already offers solutions for a wide range of situations. To complete this task, the last step is to have in the same tunnelling boring machine the ability to excavate hard rock after a fine soil leg without big transformations inside the machine.

### 3. Plano Geral de Drenagem de Lisboa 2016-2030

After showing the advantage and scope of application of the current technology, in this section it is analysed the limitations of the mechanized tunnelling technique resorting to a real project under development in Lisbon, Plano Geral de Drenagem de Lisboa 2016-2030 (PGDL). These limitations nowadays are regarding performance, or in other words, the economical point of view of the project, because technically, as showed before, there is almost no limitations.

The PGDL is a project of the Lisbon council, designed to reinforce the drainage capacity of the actual network of the city. To do it, several tunnels were planned but the present paper is focused only in the biggest one, Monsanto-Santa Apolónia (MS) tunnel (Figure 1).

![Figure 1 - Tunnel MS path (Matos, et al., 2015).](image)

### 3.1. Feasibility Analysis

The tunnel MS has a length of approximate 5 km, 90% of them with a circular cross section, where the internal shape must have a diameter of 5 m, and the last 10% (500 m) are designed with a variable rectangle shape to low the velocity of the flow before drain in Tejo river. Due to the cross section shape and low coating of the last 500 m, the length in analysis is the first 4.5 km.
Figure 2 shows that for a tunnel bigger than 3.2 km, the fixed cost related to the acquisition or renting of the machine, plus the costs of the assemble and disassemble operations, are minimized with the high excavation performance that a TBM can achieve. So having in mind that tunnel MS has a length of 4.5 km in circular cross section, it is easy to conclude that regarding the length of the tunnel, the method more suitable for the current case study is TBM.

Apart from the fixed costs, there are two phenomena that must be evaluated to predict the performance of a TBM, because they are a serious economic threat to meet a predefined budget, they are the tool wear rate and clogging.

The tool wear rate is a complex phenomenon and depends on more than one condition, but in mechanized tunnelling, it is often simplified to the abrasiveness of the geomaterials. Even abrasiveness is a parameter that is not intrinsic of the material, in other words, it depends on mineralogical composition, strength of the rocks or degree of compaction of the soils and the size of the grains, but it also relies (especially in soils) on temperature, type of TBM, forces involved, and others. But to be able to quantify this parameter, it is current to limit the evaluation to the properties of the geomaterials.

On the other hand, clogging is a phenomenon characteristic of the ground and can be described as the ability that a material have to stick in the steel parts of the TBM. This phenomena is highly dependent on mineralogical composition (here the problematic minerals are the clay minerals, especially montmorillonite) and consistency of the soils.

Having these two phenomena in mind, it was analysed all the geological formations intersected by tunnel MS (Figure 3). Note that CVL, C²c and C³c are geological formations formed by rocks and M, MII, MIII and IV are formations from the Miocene and composed mainly by soils, which means that approximately half of the tunnel length is going to be excavated in rock and the other half in soils. In his analysis the author resorts to the results of the geological survey campaign showed in Figure 3, done by (Caldeira, Jeremias, & Ramos, 2017) and because the information present in the geological report did not have tests for the abrasiveness of the Miocene soils, together with the fact that this is a subject where the non inherent parameters have an important role regarding these formations, it was only done the clogging analysis, which means that the formations were divided in two geotechnical zones (ZG I and ZG II), where the ZG I is composed by the clay formations (MI and MIII) and ZG II embrace the other formations (MII and MIV).

Regarding C²c and C³c, these are formations from the Cenomanian period composed by limestones, without any restriction in relation to strength or other parameter related to either abrasiveness or clogging. The survey campaign showed that the maximum UCS was UCS C³c = 95 MPa, which is far less than the maximum assumed value for TBMs (UCS=250 MPa) according to (Gong, Yin, Ma, & Zhao, 2016), and the minimum RQD in both was around 25%, excluding the border with CVL formation, which indicates that in terms of irregularity of the front, no problems are expected. With respect to mineral composition, only C²c showed quartz in a significant proportion, but due to the low strength of the connection between grains in the limestones present in this formation (UCSmax=40 MPa), it is not expected any problem related to tool wear rate.
About the almost 600 m of CVL, that is a volcanic formation from Neocretaceous period, composed by pyroclastic rocks and basalt, the conclusions are the opposite. It is expected a high too wear rate and clogging due to the difference on resistance, mineralogical composition and disposal of the two mentioned types of rocks. With these facts it is anticipated mixed faces at the tunnel front and also rock fragments, resulting from the excavation, with high predisposition to stick in metal surfaces. (Caldeira, Jeremias, & Ramos, 2017) assess the strength of the volcanic tuffs present around UCS = [2-25] MPa and for basalts UCS = [95-135] MPa. Either basalt or tuffs have montmorillonite in their composition in a high proportion and basalt also has as the two predominant minerals quartz and feldspar (two of the most abrasive minerals).

In terms of soils, it was investigates the potential clogging with (Hollmann & Thewes, 2013) diagram, as it is possible to observe in Figure 4. The conclusions in this case are that ZG I needs soil conditioning injection in order to avoid some clogging problems. While in ZG II is not expected any clogging problem, despite the three showed samples, because only these three, out of twelve samples, own a plasticity index bigger than 15%.

So doing an overall evaluation of the ground, it is concluded that the tunnel MS does not show any critical problem for the application of the mechanized tunnelling technology. The only geological formation that potentially showed some situations is CVL, but due to its length (13% of the entire analysed path), it is not a significant threat to the successful TBM utilization.

4. Structures Damage Analysis

Upon the conclusion of the feasibility analysis for the current case study, it was estimated the impact of a TBM excavation in the existing structures of the city. For that, it was selected the section of the tunnel path where the TBM goes closer to a build structurer. This section occurs in Avenida Almirante Reis, where the cover of the tunnel is around 12 m and there is a metro tunnel approximately 5,8 m above the tunnel MS. It is worth to mention that the outer diameter of the TBM at stake was assumed to be 5,9 m (to accounted for 0,3 m segments thickness, 0,15 m for the tail void).

At the surface, the selected section belongs to one of the most important avenues in Lisbon which was built in the beginning of the 20th century and still has most of the original buildings. The metropolitan tunnel in this avenue was constructed later but already has more than fifty years old, and when it was conceived, only cut and cover technology was available. Furthermore, the materials utilized at that time for tunnels were simple concrete, which makes this structure very sensitive as well as the existing buildings. Due to the these facts, the damage criteria defined for the buildings was done taking in consideration the Burland criteria and was admitted that the maximum category allowed was category 2, which is related to slight damage. For the underground structure, the metropolitan of Lisbon has established their own criteria and it can be seen in
Table 2, as well as the Burland damage categories taken in consideration Table 1.

One last note just to mention another important restriction imposed by (Metropolitano de Lisboa, 2016). This document also establish that any new structure or any new sealing of a micropile or an anchor has to be done in a minimum 3 m distance from any metropolitan structure in use.

![Diagram of the plan view of the studied section]

Table 1 - Building damage category - adapted from (ITA/AITES, 2007).

<table>
<thead>
<tr>
<th>Damage Category</th>
<th>Strain(%)</th>
<th>$S_{v,\text{max}}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 Negligible</td>
<td>0 - 0.05</td>
<td>&lt; 10</td>
</tr>
<tr>
<td>1 Very Slight</td>
<td>0.05 - 0.075</td>
<td></td>
</tr>
<tr>
<td>2 Slight</td>
<td>0.075 - 0.15</td>
<td>10 &lt;&lt; 20</td>
</tr>
</tbody>
</table>

Legend: $S_{v,\text{max}}$ = maximum surface settlement

Table 2 - Metropolitan tunnel criteria - adapted from (Metropolitano de Lisboa, 2016).

<table>
<thead>
<tr>
<th>Structure</th>
<th>Deformations (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alert</td>
<td>7</td>
</tr>
<tr>
<td>Alarm</td>
<td>10</td>
</tr>
</tbody>
</table>

Vertical Deformations (in Longitudinal Profile) of the railway tracks for a 6 m string

<table>
<thead>
<tr>
<th>Deformations (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alert</td>
</tr>
<tr>
<td>3 (positive or negative)</td>
</tr>
<tr>
<td>Alarm</td>
</tr>
<tr>
<td>5 (positive or negative)</td>
</tr>
</tbody>
</table>

Horizontal Deformations (in Plan view) of the railway tracks for a 4 m string

<table>
<thead>
<tr>
<th>Deformations (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alert</td>
</tr>
<tr>
<td>3 (positive or negative)</td>
</tr>
<tr>
<td>Alarm</td>
</tr>
<tr>
<td>5 (positive or negative)</td>
</tr>
</tbody>
</table>

The assumed interaction of the tunnel MS with all this structures is shown in Figure 5.

Relatively to the ground, this section is located in the soil leg of the tunnel path in Miocene formation as shown in Figure 3. Furthermore, (Caldeira, Jeremias, & Ramos, 2017) detected that close to the surface exists a layer with 3 m thickness of landfill and alluvium, which is not a surprise due to the fact that Avenida Almirante Reis is one of the biggest waterline inside Lisbon. But to have in consideration the way the metropolitan was built, for the ground under the road it was considered a thickness of approximately 9 m. This is justified through the assumption that the tunnel of the metropolitan (ML) is 7 m high and it was all constructed with a safety margin of 3 m, to prevent any perturbation in the ground after its construction. For the numerical modelling analysis it was created two different geotechnical zones, one with the Miocene soils (ZG I) and other with the superficial materials (ZG II) and the parameters considered were those present in Table 3.

Table 3 - Ground parameters adopted for numerical modelling.

<table>
<thead>
<tr>
<th></th>
<th>ZG I</th>
<th>ZG II</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Y_{\text{unsat}}$ (kN/m$^3$)</td>
<td>18</td>
<td>20</td>
</tr>
<tr>
<td>$Y_{\text{sat}}$ (kN/m$^3$)</td>
<td>21</td>
<td>23</td>
</tr>
<tr>
<td>$k$ (m/s)</td>
<td>$10^{-6}$</td>
<td>$10^{-8}$</td>
</tr>
<tr>
<td>$E_{\text{ref}}$ (MPa)</td>
<td>10 – 30</td>
<td>40 – 50</td>
</tr>
<tr>
<td>$E_{\text{res}}$ (MPa)</td>
<td>8 – 24</td>
<td>32 – 40</td>
</tr>
<tr>
<td>$E_{\text{ur}}$ (MPa)</td>
<td>30 – 90</td>
<td>120 – 150</td>
</tr>
<tr>
<td>$m$</td>
<td>0,7</td>
<td>0,5</td>
</tr>
<tr>
<td>$c$ (kPa)</td>
<td>5</td>
<td>47,5</td>
</tr>
<tr>
<td>$\phi$ (º)</td>
<td>30</td>
<td>31</td>
</tr>
<tr>
<td>$K_0$</td>
<td>-</td>
<td>0,58</td>
</tr>
</tbody>
</table>

As it can be seen in Figure 5, the problem in his case is complex and a 3D analysis might be well worth it, but due to the fact that the author, by the time of this work, did not have any software with this capacity available, it
was done a 2D analysis. To do so, a simplified method was needed. In this case, the choice was for the volume loss method. Basically, this method consists in saying that the volume of the settlement trough is given by two parcels, one relative to the volume variation of the soil and the other which is given by volume loss ($V_L$). This value has concentrated all kind of effects that cause settlements. Particularly in TBM excavations, the $V_L$ value accounts for the over excavation caused by an error in the alignment of the pre-established path, the tail void and the hydrodynamic consolidation of the clay soils. To estimate accurately this value, it is needed to do a back analysis because it depends on variables that are only known when the construction is finished. So, in mechanized tunnelling projects, the only possibility is to assume a value of $V_L$ from other tunnel excavation, in the same geological conditions and with the same type of TBM. For the present work, the $V_L$ considered was 1%, because it was the value assumed characteristic of the Lisbon Miocene formation by (Amaral, 2006) that analysed several TBM excavation for Lisbon metropolitan.

4.1 Building Damage Assessment

To do the building damage analysis, it was followed the approach proposed by (ITA/AITES, 2007), where they suggest that for the two first stages of the analysis, the Gaussian method conceived by Peck in 1969 should be determined with the equations above.

\[ S_v(x) = S_{v,max} \cdot e^{-\left(\frac{x^2}{2ix_x^2}\right)} \]  
(4.1)

\[ S_{v,max} = \frac{V_s}{\sqrt{2\pi} \cdot ix} \]  
(4.2)

Where $S_{v,max}$ is the maximum settlement in the transversal cross section, $x$ is the horizontal coordinate measured in relation to the tunnel axis, $V_s$ is the volume of the settlement trough and $ix$ is the point of inflection of the Gaussian distribution (this value is also empirical and is given by an empirical parameter $K$, multiplied by the depth of the axis of the tunnel).

Note that in this approach it is considered that the soils are in undrained conditions, which means that the volume of the settlement trough ($V_g$) is equal to $V_L$. As for empirical parameter $K$ it was assumed the values given by (Amaral, 2006) that result in $ix = 7.9$ m.

In terms of the application of the two equations above, the result is the curve shown in Figure 6.

![Figure 6 - Surface transversal settlements for green field conditions.](image)

As one can see in the above figure, the criteria assumed for the buildings related to $S_{v,max}$ is clearly violated. But in a second stage analysis, where it is assessed the strain of the buildings neglecting the rigidity of themselves, the results show no violation of the established criteria. For the most unfavourable building, building 3 in Figure 5, that has its middle coincident with the tunnel axis and a length of 20 m the strain is far less (DR=0.04%) than what was assumed for damage criteria 2 (DR=0.15%). These conclusions, regarding the buildings, can be explained by the cover of the tunnel compared with the tunnel diameter ($C/D_{ext}=2.3$), which is considerable for a urban tunnel.

4.2. Tunnel ML Damage Assessment

For assessing the damage in the Lisbon metropolitan (ML) tunnel, the approach followed was going directly in a finite elements analysis with the software Plaxis version 8.2, due to its proximity and higher demands in terms of deformations.

To calibrate the ground parameters, it was done a sensitivity analysis, having as reference the curve for
surface settlements, given by the Gaussian method. The best results are shown in Figure 6 and it is clear that the curves from Plaxis meet the reference curve with a slight difference for distances bigger than 10 m, where the Plaxis model show a less conservative curve regarding strains. In order to take into consideration this aspect was chosen $E_{50}^{ZG\ II}=50$ MPa for $ZG\ II\ $ and $E_{50}^{ZG\ I}=30$ MPa for $ZG\ I$.

With the mentioned parameters, it was conceived another model taking into consideration the real conditions of the tunnel ML (Model 1) to assess the expected transversal settlements of this structure when the TBM is excavating below. The results are shown in Figure 6. This new model has two big differences compared with the first reference model, first it is considered a bigger thickness for $ZG\ I$, since it is taken into account the embankment done more than 50 years ago, when it was constructed the tunnel of the metropolitan, and a consideration of a traffic surcharge of 10 kN/m.

Due to the environment where this solution has to be done, some demands were defined for the reinforcement technique chosen, for example: it has to be versatile – which means if some problem with TBM or with the execution of solution happen and the TBM excavates the reinforced ground it should not cause any damage to the TBM or to the tunnel ML –, it must be done with less interference in the normal traffic of the avenue and has to be as current as possible.

To meet all this demands the choice made was micropiles reinforced with fiberglass. To reflect this solution in the calculations of the finite elements program, it was conceived an equation (eq. 4.3) affecting only the deformation modulus of the reinforced ground.

$$E_{50}^{homog.} = \frac{(A_{inf.microest} - A_{microest})\ E_{ZG\ II} + A_{microest}\ E_{calda}}{A_{inf.microest}}$$ (4.3)

Where $E_{50}^{homog.}$ is the deformation modulus of the reinforced ground; $A_{inf.microest}$ is the influence area of the micropile sealing; $A_{microest}$ is the cross section area of the free length of the micropile (it was considered a $D_{hole}=250$ mm); $E_{calda}$ is the deformation modulus of the grouting (it was assumed 7.5 GPa) and $E_{ZG\ II}$ is the modulus of the soil without intervention, or in other words the deformation modulus of $ZG\ II\ $ ($E_{ZG\ II}=50$ MPa).

By the explanation of equation 4.3 it is possible to observe that the only two parameters undefined are $E_{50}^{homog.}$ and $A_{inf.microest}$ which were determined by an iteration process, but having in mind that $A_{inf.microest}$ has its limitations and in the present work this value was limited for a $1.5\times1.5\ m$.

Doing the iteration process it was concluded that the cheaper solution (the one with bigger $A_{inf.microest}$) solves the problem properly, as it can be seen Figure 7. The curve of this new model (model 2) demonstrates a string of 6 m. Although due to the fact that the structure criteria is violated it is necessary an intervention on the ground between the two tunnels and it was assumed that this intervention must have 20 m of width to guarantee a safe margin for the tunnel ML.

As one can see in Figure 7, the calculation results show a maximum settlement for tunnel ML of 18.7 mm which is almost twice than the 10 mm alarm limit established by (Metropolitano de Lisboa, 2016) for the structure of the tunnel ML. Additionally, the shape of the output curve shows that the alert criteria for railway tracks (3 mm in a string of 6 m) is not violated because it cannot be measured any displacement of 3 mm in any

![Transversal Settlements (Sv)](image)

Transversal Settlements (Sv)

Distance to tunnel axis (m)

Invert ML (Model 1) Invert (Model 2)

Figure 7 - Transversal settlements for tunnel MS’s invert.
maximum settlement for the invert of tunnel ML around 1.5 mm, which verifies not only the alarm criteria established for the rail track and the entire structure but also the alert criteria as desired. This is accomplished through a total confinement of the displacements induced by the TBM of the tunnel MS in the space between the crown of this tunnel and the solution. To mitigate this effect and to ensure the durability of the proposed solution it is also suggested to do a secondary grouting injection from inside the tunnel after the construction of the lining.

4.3. Implementation of the Proposed Solution

Arriving to this stage, it is necessary to convert the equation 4.3 in number of micropiles. If we have in mind that the defined critical area have already its geometry pre-established, due to the imposition of (Metropolitano de Lisboa, 2016) for new structures and due to the conclusion of the presented calculation related to the width of the intervention, considering the value calculated for \( A_{\text{int.microest}} \) it is easy to understand that only 13 micropiles can meet all these requirements.

Moreover, it is important to understand that only one plan of the problem was considered for the calculation, the other, because of the limitation of the software used, was neglected, but it is actually not possible to do such simplification. Thus it was conceived also a solution for this plan and it can be seen is Figure 10.

After doing the calculations and designing a solution, it is also very important to itemize it with drawings and define an instrumentation plan, essential to monitoring the behaviour of the solution. All these aspects can be seen in Figure 8, Figure 9 and Figure 10.

5. Final Remarks

Considering the main goal presented initially, it is possible to conclude that it was accomplished. The current study contributes to highlight all the factors that have to be taken into account when a project with tunnelling boring machines is at stake. Moreover, it gives a detailed description of the current state of the art of this industry, warn for all the main technical and economic analysis that have to be done in a preliminary stage for a good decision related to the use of mechanized tunnelling technique, and at the same time gives an idea of the impact that a TBM can have in the existing structures.
It also reveals that for the PGDL project, the mechanized tunnelling is the most suitable option, despite some challenges that the city of Lisbon has to offer. The peak of these challenges are the excavation of CVL formation, that reveal some problems related to clogging and tool wear rate, and the damage that the metropolitan might suffer.

Specifically regarding the section of Almirante Reis Avenue, it is concluded that the construction of Monsanto-Santa Apolónia (MS) tunnel will not induce any significant damage in the surface buildings but will induce some problems in the metropolitan (MS) tunnel presented in this avenue. To solve this issue it is shown that a solution with 68 micropiles is a good solution and has a lot of practical advantages. Of course all these conclusions must be checked in an advanced stage of the project, with complementary geological and geotechnical prospection plan.

In addition, it also highlights the limitations of 2D finite element modelling regarding tunnels. It is shown that when this is the only tool available, the geotechnical engineer has to overcome this problem with some degree of creativity, but always in the safe side, which can lead to an over conservative solution.

References


