Tunnelling on terrace soil deposits: Characterization and experiences on the Bogota-Villavicencio road

Julio E. Colmenares\textsuperscript{1*}, Juan M. Dávila\textsuperscript{2}, Jairo Vega\textsuperscript{2} and Jong-Ho Shin\textsuperscript{3}

\textsuperscript{1) Department of Civil and Agricultural Engineering, Universidad Nacional de Colombia, Carrera 30 No. 45-03 - Bogotá D.C., Colombia. jecolmenaresm@unal.edu.co}
\textsuperscript{2) EDL SAS, Calle. 26 No. 59 - 41 - Bogotá D.C., Colombia. info@edlingenieros.com}
\textsuperscript{3) Department of Civil Engineering, Konkuk University, 120 Neungdong-ro, Jayang 1(il)-dong, Gwangjin-gu, Seoul, Korea. jhshin@konkuk.ac.kr}

ABSTRACT

Terrace deposits are often encountered in portal areas and tunnels with low overburden. Those deposits exhibit a great mechanical and spatial heterogeneity and a very high stiffness contrast within the ground. Terrace deposits are challenging to excavate, and difficult to characterize. Lessons learned and experiences and challenges encountered during tunnelling in terrace deposits on the Bogota-Villavicencio road (central-east Colombia) are presented. Considering that samples for laboratory testing are almost unfeasible to obtain from such deposits, and laboratory tests may not be representative because of scale effects, this document presents the approach taken for their characterization during the design stage and its posterior validation performed during the construction stage. Lessons learned suggest that based on numerical simulations, laboratory testing and tunnel system behaviour documented, in several tunnels on the Bogota-Villavicencio road, where terrace soil deposits were found, an observational approach allows the engineer to optimize the excavation and support methods for the encountered ground conditions, resulting in a more economic and safe construction.

Keywords: alluvial deposits, terrace, complex terrain, ground characterization, ground behaviour.

1. INTRODUCTION

Rivers may form different types of deposits whose engineering behaviour depends on the size and nature of the placed materials. Those deposits occur due to particle deposition and they are called alluvial soil deposits. One special alluvial deposit of
engineering interest is the terrace soil deposit (Maher, M. (2015), “Highways: Soil and soil testing for roadworks.” ICE Virtual Library: Essential engineering knowledge.). Terrace formation occurs mainly in river valleys and it is controlled by several factors whose contributions are still difficult to determine and understand; they can be formed also by glacial deposition. Terraces are characterized according to their cross section, which is formed of large relatively horizontal and scarped adjoining layers. Once the stream opens its way by eroding the ground creating meanders and reaches greater depths, a new floodplain is formed and what used to be the existing floodplain becomes in a terrace. This process happens over time and many terraces can be formed, therefore, the higher a terrace is, the older it is (Tevelev, A.V. (2014), “Tunnel Erosion, Upward-Unsealing Terraces, and Dynamic Interpretation of Alluvial Deposits.” Moscow University Geology Bulletin, Vol. 69(1), 17-27.). This kind of soil deposits often are involved with civil engineering projects such as earth dams, highways and railways (after proper compaction), high buildings whose foundations generally are piles due to the insufficient shear strength of the soil deposit, and tunnels, which are the focus of this document.

The need for economic and social development, sometimes requires the design and construction of complex engineering projects on terrace soil deposits, involving special engineering solutions, such as tunnels. Therefore, prediction and monitoring of tunnel behaviour are important issues during excavation and construction works. Most monitoring activities take place to update geological and geotechnical models, among them are: exploratory drillings ahead of the excavation, identification of structural features at the tunnel face through stereographic imaging and lithological observation and water inflow evaluation, among others. On the other hand, the excavation behaviour considering the implemented support system (system behaviour) is assessed by measuring deformation on the tunnel and stresses on the support elements. In recent years, the use of geodetic methods has increased for measuring deformations (Schubert, W., Riedmüller, G. (2000), “Tunnelling in Fault Zones – State of the Art in Investigation and Construction.” Felsbau, Vol. 18(2), VGE, 7-15.). All the above activities are based on the principles of the observational method, leading to a safe and economical tunnelling activities by achieving a best fit result.

Although additional instrumentation such as stress and strain measuring devices, extensometers, and geophysical methods have been implemented, the experience determines that from a practical point of view, a proper approach for soil deposits such as terraces should be developed by combining a sophisticated theoretical approach adjusted to onsite monitoring and observations. Therefore, this document introduces a state of practice about tunnelling in terrace soil deposits. Moreover, a tunnelling project in the new Bogota-Villavicencio road in Colombia is taken as an example of the studied subject. The approach taken in order to optimize the excavation and support methods for the encountered ground conditions of the project is presented.
2. TUNNELING TERRACE SOIL DEPOSITS

2.1. Bimrocks approach for soil terrace deposits

The approach proposed by Medley (Medley, E. (1994), “The engineering characterisation of melanges and similar block-in-matrix-rocks (bimrocks).” PhD Thesis. Department of Civil Engineering, University of California at Berkeley, California.) has shown to be adequate when dealing with soil terrace deposits. The author’s approach is based on the so called “Bimrocks” (block-in-matrix rocks) which include weathered rocks, fault rocks, deposits and melanges. Bimrocks can be found in many geologic regions of the world. Despite different origin processes, these globally common soil/rock mixtures have a similar fabric of relatively hard blocks of rock surrounded by weaker matrix rocks. Characterization, design and construction with “bimrocks” is challenging because of their considerable spatial, lithological and mechanical variability. Geotechnical engineers and engineering geologists often mischaracterize them. In general, the term block-in-matrix or “Bimrock”, as defined by Medley (Medley, E. (1994), “The engineering characterisation of melanges and similar block-in-matrix-rocks (bimrocks).” PhD Thesis. Department of Civil Engineering, University of California at Berkeley, California.), is also used to describe alluvial terraces with a relevant volumetric block proportion.

In order to focus on the fundamental engineering problems related to the characterization of these and many other “rock/soil” mixtures, Medley (Medley, E. (1994), “The engineering characterisation of melanges and similar block-in-matrix-rocks (bimrocks).” PhD Thesis. Department of Civil Engineering, University of California at Berkeley, California.) coined the neutral word “bimrocks”, which has no geological connotations. Bimrocks are defined as “a mixture of rocks, composed of geotechnically significant blocks within a bonded matrix of finer texture.” The expression “geotechnically significant blocks” means that there is mechanical contrast between blocks and matrix, and the volume and size of the blocks influence the rock mass properties at the scales of engineering interest (Medley, E. (2007), Bimrocks Article Part 1: "Introduction”. Newsletter of HSSMGE, pp. 17-21.).

Bimrocks are widespread and include weathered rocks, geological faults, and deposits, which are mixtures of decomposed soil surrounding fresher corestones or heterogeneous and complex geological mixtures containing competent blocks of varied lithologies, embedded in sheared or soil matrix.
Although the geological literature contains thousands of references on this material, there are few treatments related to geoengineering. Geoengineers often neglect the contributions of blocks to overall bimrock strength, choosing instead to design on the basis of the strength of the matrix. However, this practice may be conservative for many bimrocks and often results in ignoring the presence of blocks altogether, to the detriment of accurate characterizations. As block proportions increase, stiffness increases and deformation decreases depending on the relative orientation of blocks to the applied stresses (Lindquist, E.S. (1994), “The strength and deformation properties of melange” Ph.D. Dissertation; Dept. of Civil Engineering, Univ. California at Berkeley, California., Lindquist, E.S. and Goodman, R.E. (1994), “The strength and deformation properties of a physical model melange” Proceedings of 1st North American Rock Mechanics Conference (NARMS), Austin, Texas; ed. Nelson, P.P. and Laubach, S.E., A.A. Balkema, Rotterdam.). Stress distributions in bimrocks depend on the lithologies; size distributions; orientations and blocks shape; and the orientations of matrix shears, all of which influence stability on underground excavations (Medley, E. (2007), Bimrocks Article Part 1: "Introduction". Newsletter of HSSMGE, pp. 17-21.).

Lindquist (Lindquist, E.S. (1994), “The strength and deformation properties of melange” Ph.D. Dissertation; Dept. of Civil Engineering, Univ. California at Berkeley, California.), Lindquist and Goodman (Lindquist, E.S. and Goodman, R.E. (1994), “The strength and deformation properties of a physical model melange” Proceedings of 1st North American Rock Mechanics Conference (NARMS), Austin, Texas; ed. Nelson, P.P. and Laubach, S.E., A.A. Balkema, Rotterdam.) and Goodman and Ahlgren (Goodman, R.E., and Ahlgren, C.S. (2000), “Evaluating the safety of a concrete gravity dam on weak rock-Scott Dam” Journal of Geotechnical and Geoenvironmental Engineering, Vol 126, 429-442, with Discussion (by J. H. Hovland, E.W. Medley and R.L. Volpe; and Authors), v. 127, October 2000, p. 900-903.) determined that the overall strength of a bimrock is related to the volumetric proportions of the blocks, establishing that below about the 20 percent volumetric block proportion, the strength and deformation properties of a bimrock is that of the matrix; between about 20 percent and 75 percent, the friction angle and modulus of deformation of the bimrock mass proportionally increases (and cohesion decreases); and, beyond 75 percent block proportion, the blocks tend to touch each other and there is no further increase in bimrock strength. However, blocks matrix supported do not directly contribute to the mechanical behaviour of the bimrock, it is matter of scale. Medley (Medley, E. (1994), “The engineering characterisation of melanges and similar block-in-matrix-rocks
defined a “characteristic engineering dimension, Lc”, which may vary depending on the scale of the assessed engineering structure. Lc for tunnels is defined by the author as the tunnel diameter. The author says: “…the smallest geotechnically significant block within a volume of bimrock is about 0.05 Lc, which constitutes the size between blocks and matrix at the chosen scale. For any given volume of bimrock, blocks smaller than 0.05 Lc may be greater than 95 percent of the total volume but contribute less than 1 percent to the total volume of bimrock and thus have negligible effect on the bimrock strength”.

Blocks typically have a larger permeability and storability than the fine grain matrix. This contrast can create significant water and seepage forces between the blocks, the matrix, and the excavation. Blocks located just outside of the excavation, may create a high potential for a water pressure inducing failure on the weaker matrix (Button, E.A., W. Schubert and G. Riedmueller (2002), “Shallow Tunneling in a Tectonic Melange: Rock Mass Characterization and Data Interpretation” Proceedings of the 5th North American Rock Mechanics Symposium, Toronto, Canada.). This behaviour can be considered one of the most critical situations and it is often associated with more severe overbreaks or top heading collapses (Dissauer, J., Leitner, A., Mittelbach, H. (2002), “Tunnel Spital-Tunnelbau in schwierigen Verhältnissen.” Felsbau, Vol. 20(1), 40-48.).

2.2. Back analysis

Medley’s approach (Medley, E. (1994), “The engineering characterisation of melanges and similar block-in-matrix-rocks (bimrocks).” PhD Thesis. Department of Civil Engineering, University of California at Berkeley, California.) is useful to characterize strength properties of a terrace deposit. However more complex constitutive models require deformation parameters in order to simulate the tunnel behaviour (Jing, L. & Hudson, J. (2002), “Numerical Methods in Rock Mechanics.” Int. J. Rock Mech. and Min. Sci., Vol. 39(4), 409-427.). Considering the complexity of such deposits, it is not feasible to obtain deformation parameters from the preliminary geological studies, from the geotechnical and geophysical explorations or even from laboratory testing. Only during the construction of the tunnel itself, or from a pilot tunnel, is it possible to obtain a complete evaluation of the behaviour of the rock mass (ITA, W.G.C.T., General report on Conventional Tunnelling Method. 2009. p. 28.).

On the other hand, the use of monitoring instrumentation has expanded as the instruments used have become more precise, reliable and sufficiently robust to be used in the hostile environment of a tunnel (Oreste, P.P. (2005), “Back-analysis techniques for
the improvement of the understanding of rock in underground constructions." Tunnelling and Underground Space Technology Vol. 20, 7-21.). Other analysis techniques are also used during the design stage, considering a nearby structure (e.g. slopes, foundations, etc.) constructed on similar ground conditions. The same constitutive model could be used for tunnel design. However, it is not an easy task and experience plays an important role in order to avoid misleading the ground characterization and therefore the design itself. During the construction stage, measurements of the displacements on the tunnel perimeter and the loads on the support structures, are often used to calibrate the initial estimations (design stage) of the parameters used for the ground (Oreste, P.P. (2005), “Back-analysis techniques for the improvement of the understanding of rock in underground constructions.” Tunnelling and Underground Space Technology Vol. 20, 7-21.). Oreste (Oreste, P.P. (1997), “Tecniche di Back-Analysis per il Miglioramento della Conoscenza della Roccia nelle Costruzioni in Sotterraneo.” GEAM-GeolIngegneria Ambientale e Mineraria, Vol. 91, 49–55.) stated that in order to carry out a back-analysis, it is necessary to choose:

a) a representative constitutive model able to determine the stress and strain conditions of the ground, updated with the evolution of the excavation phases,
b) the error function based on admissible deviations and
c) an efficient algorithm to reduce the error between the calculations of the numerical model and the observed in situ measurements.

There are many reasons why back-analysis techniques are being used more frequently. Among the most important are: the development of numerical methods for the analysis of ground stresses and strains, and the computers capability to assess large amounts of data (parametric analyses), necessary to resolve the numerical modelling with minimum error, in the shortest possible time and with the lowest possible cost.

3. THE BOGOTA-VILLAVICENCIO ROAD: ITS BEGINNING AND UPGRAADING

The Bogota-Villavicencio road is the most important connection between Bogota (Colombia’s capital), and the Eastern plains, where the main oil and agricultural production of the country takes place. The road is located on a corridor that crosses the eastern branch of the Andean mountain range in Colombia. Fig. 1 shows the project location. With a length of approximately 86km, the road starts in the limits between urban and rural area in the southside of Bogota and ends at entrance of Villavicencio city.
The corridor in which the road is located, is characterized by a highly heterogeneous geology mainly set by different soil deposits and sedimentary and metamorphic rocks, immerged on a highly tectonic activity area. That complex geological-tectonic situation has favoured the occurrence of multiple landslides, whose consequences vary from economic losses, as happened in 2008 where concentrated landslides near Quetame (km46) caused 7 days of road closing and enormous economic losses (Romero, R.D. (July 30th, 2004) El Tiempo: Bogotá- Villavo: tragical and unfinished road (In Spanish). Retrieved from: http://www.eltiempo.com/archivo/documento/MAM-1584433), to tragic consequences as recorded in June 1974 when a landslide in Quebrada Blanca buried approximately 300 people (Gomez, M. (2015) Cronicles – Villavicencio, so close and so far, Aguilar, Bogotá, Colombia (In Spanish).).
The road is divided into three parts, the first of them (km0 to km34) runs from Bogota to a place called El Tablón intersection, entailing a geometrical alignment with a considerable elevation difference (approximately 1400m) between the initial and final points. This section offers great geotechnical challenges derived by the presence of ground water on a geology characterized by the occurrence of soft sedimentary rocks (sand, mud and clay stones) and thick colluvial deposits. The second third, which is the focus of this paper, has an extension of approximately 29km (from km34 to km63) starting at El Tablon intersection and ending at the Chirajara Bridge. This section of the road is composed of several tunnels of varying length built on terrace deposits. The last third runs from Chirajara Bridge to Fundadores Intersection (km63 to km 85.7). The last two thirds of the road display a great geotechnical challenge due to the high tectonic activity combined with deep soil deposits, and the presence of sedimentary and metamorphic rocks which are present along the valleys of River Negro, River Blanco and their affluents.

An initial major intervention to the road took place between 1995 and 2002. During this period the Colombian government signed two big contracts to intervene the second and last third of the road, including the construction of several bridges, a 10km by-pass (Caqueza by-pass) and a 4.4km tunnel (Buenavista tunnel). Additionally, the government signed another concession contract with CoviAndes for the first third, which included the construction of a 2.4km tunnel (Boqueron tunnel). The intervention to the road concluded in 2002 with the inauguration of the Buenavista Tunnel.

Operation and maintenance of the road was granted to the concessionaire in 2006. That year the road was completed as a 2-lane bidirectional road and it experienced a traffic volume increase of approximately 6% each year since 2005. This may also be due to the more stable social and political conditions on the area and the oil increase production on the eastern planes. The concessionaire (CoviAndes) proposed and signed a further intervention on the road’s second third (El Tablon – Chirajara), where landslides forced the closure of the road almost on a monthly basis. The intervention project, proposed by the concessionaire, focused on the construction of two additional lanes based on bridges and tunnels in order to avoid surface geotechnical problems (i.e. landslides). The ambitious proposal included the construction of 18 tunnels (total length of 15.4km) and 46 bridges (total length of 5.2km).

Terrace deposits along the road project studied display a flat to wavy morphology. They are located on both sides of the main rivers of the area, rivers Negro, Blanco and their affluent streams. Deposit thickness is variable and it may be up to 230m, as it was found on exploratory drillings performed near the place called the Santandereana Ridge. Terrace soil deposits are composed by materials of different grain sizes. The studied terrace soil deposits, are usually block supported, displaying volumetric block proportions above 25%. The matrix is mainly integrated by sand-clay soil, showing low signs of consolidation. Block sizes vary from 0.4m to 0.8m diameter, and occasionally can be found blocks with average size above 1.5m (see Fig. 2). Blocks are mainly
formed by phyllites, sandstone or metasandstone, with different degrees of compaction and weathering, depending on the age and origin of the deposit.

Fig. 2. Tunneling in terrace soil deposits, tunnel 12 Bogota-Villavicencio road.

Due to the presence of steep slopes and the characteristic tectonism found in the area, a combination of geological events is often found overlapping terrace and colluvium deposits with a higher content of clay in the matrix. Considering the characteristics of the matrix in the colluvium deposits, and the rainfall intensity of the area, the contact between Terrace-Colluvium deposits may be fully saturated, creating a possibly unstable surface.

4. TUNNELLING TERRACE SOIL DEPOSITS ON THE BOGOTA – VILLAVICENCIO ROAD

Tunnel designs in terrace soil deposits for the Bogota-Villavicencio road were mainly developed using the methodology proposed by the Austrian Society of Geomechanics (OeGG (2010), Guideline for the Geotechnical Design of Underground Structures with Conventional Excavation. Austrian Society for Geomechanics.). It places especial attention to the so called “ground behaviour”, which by definition is the reaction of the ground to a full-face excavation, without implementing any support elements. Understanding the ground behaviour is an unavoidable task that a tunnelling engineer must face to achieve an adequate design. It is important to clearly identify failure mechanisms given by the ground and influencing
factors (i.e. excavation shape and size, ground water and primary stresses). The assessment of the ground behaviour is a useful tool to design excavation sequences and support elements which must be designed to control the already identified failure mechanisms.

The design process continues with the system behaviour assessment, which by definition is the reaction of the ground including a support system (excavation sequences and support elements). The final stage of the design validates the assessed support systems with specific requirements, given by the environment in which the tunnel is going to be constructed (e.g. safety factors, restriction on ground water inflow, maximum displacements, utilization factors of the support elements, etc.).

During construction, the methodology proposed by the OeGG makes use of an observational approach to evaluate whether the ground properties, the influencing factors and the system behaviour assessed during design, match those found during construction. Observational methods are characterized by analysing field measurements and their rational interpretation, not only for the evaluation of tunnel stability but also for the verification or modification of the initial design and the construction method (Sakurai, S., Akutagawa, S., Takeuchi, K., Shinji, M. and Shimizu, N. (2003), "Back analysis for tunnel engineering as a modern observational method." Tunnelling and Underground Space Technology, Vol. 18, 185-196.).

Considering the above-mentioned methodology, there are still many uncertainties when dealing with soil terrace deposits. The nature of such deposits involves a notorious spatial heterogeneity and high stiffness contrast between matrix and blocks, deriving on a difficult characterization in terms of strength and deformability properties; which at the same time lead to uncertainties on the system behaviour.

4.1 Ground investigation

There is no doubt that difficult ground conditions, such as the existence of terrace soil deposits, means problems for tunnelling. Those problems can range from different level of overbreaks, deformations, severe water inflows, and even the tunnel collapse (Button, E.A., W. Schubert and G. Riedmueller (2002), "Shallow Tunneling in a Tectonic Melange: Rock Mass Characterization and Data Interpretation" Proceedings of the 5th North American Rock Mechanics Symposium, Toronto, Canada.). Typically, during tunnel design, the soil and rock properties and joint behaviour are determined in the laboratory, while the ground mass system characteristics are determined from field investigations and subsurface exploration programs. These results are commonly supported with numerical simulations to evaluate the ground behaviour and/or support loads for different support and excavation methods.

Ground investigation includes any activity needed to define geotechnical/geological relevant conditions for the design, on the area to intervene. In
other words, to evaluate the ground type (e.g. rock, soil, deposits, etc.) and its conditions. The material characterization focuses on establishing the material properties which can be used during the design process. It is worth mentioning that both, ground investigation and material characterization techniques, should be selected based on the expected material type.

Terrace soil deposits on the Bogota-Villavicencio road project, were initially investigated through geological mapping, followed by a second stage in which core drilling was executed, aiming to determine ground water conditions, the contact between soil deposit and rock and, if no contact was found, to evaluate the thickness of the deposits. Drilling on such deposits represents a challenge due to a complex environment displaying stiffness variation and heterogeneity on the ground, deriving on low drilling rates and difficulties to recover laboratory samples for testing.

Geophysics was consistently used throughout the project. For tunnels on terrace deposits, seismic refraction and reflection were performed. On grounds, different to terrace deposits, where no blocks are found in a matrix, seismic geophysics is quite useful to establish the ground variation with depth, based on wave velocity differences recorded on geophones. However, it was observed that such techniques are difficult to implement in such kind of ground, taking into consideration the high stiffness variation within the deposit, radical variation on results were recorded pending on the geophone arrangement and source location.

4.2 Geomechanical characterization

Representative samples of the matrix of the terrace soil deposits were taken mainly from tunnel 13 (exit portal) and tunnel 7 (lateral slope of the portal area) on the second third of the Bogotá-Villavicencio road. They correspond to a brown-olive silty gravel containing some blocks of size higher to 3”. Particle size distribution of the matrix of the sample was determined. Additionally, shape index, specific gravity, and Atterberg limits were measured. The results are presented in Table 1.

Table 1. Summary of the main index properties of the material studied

<table>
<thead>
<tr>
<th>Specific gravity of solids</th>
<th>2,773</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid Limit (%)</td>
<td>NL</td>
</tr>
<tr>
<td>Plastic limit (%)</td>
<td>NP</td>
</tr>
</tbody>
</table>

Particle size distribution

<table>
<thead>
<tr>
<th>Sieve</th>
<th>Percentage passing (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3&quot; (75 mm)</td>
<td>100,00</td>
</tr>
<tr>
<td>4 (4,75 mm)</td>
<td>43,60</td>
</tr>
<tr>
<td>10 (2 mm)</td>
<td>36,30</td>
</tr>
<tr>
<td>40 (0,425 mm)</td>
<td>26,00</td>
</tr>
<tr>
<td>200 (0,075 mm)</td>
<td>16,60</td>
</tr>
</tbody>
</table>

Shape Index of soil particles

<table>
<thead>
<tr>
<th>Elongation Index (%)</th>
<th>28,43</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flattening Index (%)</td>
<td>31,88</td>
</tr>
</tbody>
</table>
In order to establish the deformation modulus of the granular matrix, consolidation tests were performed on samples with the initial characteristics presented in Table 2. Medium size oedometers, 4.4 inches internal diameter, were used. Samples with particles smaller than 1 inch were prepared at different initial densities. They were saturated and loaded up to a maximum vertical stress ranging between 450 and 950 kPa (reasonable stresses to be found on the ground). Initial and final Particle size distribution of the material was evaluated in order to verify particle crushing. The results confirmed that no significant crushing was taking place under this level of confinement.

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Table 2 Summary of the characteristics of the samples tested in one dimensional consolidation.

<table>
<thead>
<tr>
<th></th>
<th>Condition 1</th>
<th>Condition 2</th>
<th>Condition 3</th>
<th>Condition 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Final water content (%)</td>
<td>17.64</td>
<td>17.40</td>
<td>16.97</td>
<td>17.50</td>
</tr>
<tr>
<td>Initial Unit Weight (kN/m³)</td>
<td>13.47</td>
<td>16.26</td>
<td>16.70</td>
<td>17.75</td>
</tr>
<tr>
<td>Final Unit Weight (kN/m³)</td>
<td>21.52</td>
<td>21.15</td>
<td>21.67</td>
<td>22.79</td>
</tr>
<tr>
<td>Initial void ratio</td>
<td>1.04</td>
<td>0.69</td>
<td>0.65</td>
<td>0.55</td>
</tr>
</tbody>
</table>

Fig. 3 shows the stress – strain relationship of the studied materials. Typical behaviour of granular materials is displayed. The materials become more rigid as the confining stresses increase and consequently its compressibility reduces.
Fig. 3. Stress-Strain relationship of the samples studied

Considering the oedometer testing results and assuming Poisson’s ratio values between 0.20 and 0.35, Elasticity Modulus were estimated for each load increment. The effect of the influence of the Poisson’s ratio was estimated for each condition, as a function of the vertical strain, and the results are presented in Fig. 4, for the densest condition tested.

Additionally, direct shear tests, using medium size shear boxes, were performed on samples prepared according to the characteristics shown on Table 2. The initial particle size distribution was used. Samples were saturated and they were sheared at a very slow speed (0.035 mm/min) until they reached a displacement of about 10% of the sample diameter. Fig. 5 shows the stress paths followed during shearing for the loosest samples (i.e. condition 1).

Table 3 shows the results of the shearing resistance angle, measured for the samples studied. The experimental results, obtained from the oedometer and direct shear tests, indicate that the Elasticity Modulus of the studied material is strongly influenced by the initial density. Presumably the presence of rock blocks also influence the compressibility as it was found from the back analysis reported below where the Elasticity modulus was estimated to be 1 to 2 orders of magnitude higher than the measured for the granular matrix. The Poisson’s ratio effect was found to be not significant (from a practical engineering point of view), as there is no significant variation of the estimated Elasticity modulus as a function of the Poisson’s ratio (see Fig. 5). The material studied exhibits a relatively high permeability and it has cohesion ranging between 0.0 and 0.03 MPa and shearing resistance angles ranging between 36° and 42°. The behaviour is consistent to the one expected for granular materials, considering the particle size distribution.
Fig. 4. Estimation of the influence of the Poisson's ratio (condition 4)

Fig. 5. Stress paths on a σ-τ diagram (condition 1).

Table 3. Direct shear testing
4.3 Strength parameters

Strength parameters were assessed following Medley's approach (Medley, E. (1994), “The engineering characterisation of melanges and similar block-in-matrix-rocks (bimrocks),” PhD Thesis. Department of Civil Engineering, University of California at Berkeley, California.). Samples of the matrix were taken and characterized through direct shear testing (see Table 3). The material friction angle was increased based on the volumetric block proportion and taking the tunnels diameter for the characteristic engineering dimension. Block proportion was determined using the software ShapeMetrix3D, which acquires surfaces with three-dimensional images, Fig. 6 shows an example of the surfaces taken from tunnel 13.

<table>
<thead>
<tr>
<th>Sample type</th>
<th>Direct shear test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Initial unit weight</td>
</tr>
<tr>
<td>1</td>
<td>13.02</td>
</tr>
<tr>
<td>2</td>
<td>16.45</td>
</tr>
<tr>
<td>3</td>
<td>16.66</td>
</tr>
<tr>
<td>4</td>
<td>17.33</td>
</tr>
</tbody>
</table>

(a) Analysis 1 (VBP = 40.6%)  (b) Analysis 2 (VBP = 30.3%)

Fig. 6. Volumetric block proportion (VBP) at the entrance of tunnel 12

For this example, it was determined that tunnel 13 had a 35% volumetric block proportion, which according to Lindquist (Lindquist, E.S. (1994), “The strength and deformation properties of melange” Ph.D. Dissertation; Dept. of Civil Engineering, Univ. California at Berkeley, California.), represented and increment of 5 degrees on the friction angle, resulting on an increment from 37° to 42°. The elasticity modulus was assessed based on a back analysis (see section 4.4) approach resulting on an E-modulus equal to 750MPa for the material related with this tunnel.
4.4 Numerical simulations

Ground properties definition and the selection of a constitutive model that accurately represents the ground behaviour, was a challenge. The approach was based on the characterization proposed by Medley and Medley and Lindquist (Medley, E. & Lindquist, E. (1995), “The engineering significance of the scale-independence of some Franciscan melanges in California.” Proceedings 35 US Rock Mechanics Symp.). The strength properties were determined from laboratory testing performed on the matrix. The ground friction angle was increased considering the tunnel’s diameter as “characteristic engineering dimension” and the block volumetric proportion. The approach proved to be very useful for limit state analysis. Back analysis techniques were performed to determine deformability parameters and to calibrate – cross check strength parameters (see Fig. 7).

4.5 Designed support systems and their implementation during construction stage

Following the selected design approach (OeGG (2010), Guideline for the Geotechnical Design of Underground Structures with Conventional Excavation. Austrian Society for Geomechanics.), two main requirements were set, based on the material characterization and expected ground behaviour; they were low deformability capability of the ground and proper behaviour during seismic events, important for tunnels with low overburden. As a consequence, the support system was conservatively designed relying mainly on the support elements and not on the ground contribution to stability. The design support system for Ground Class “Terrace deposits”, included:

Excavation sequence: Top heading (TH) – Bench (B) – Invert (I).
1.2m to 2m round length.
20m distance TH – B.
40m distance B-I.

**Excavated cross section:** 102m² to 106m².

**Ground improvement:** 21 umbrella pipes, 12m long with 3m overlap.
Grouting through umbrella pipes with low pressure (2-3bar, filling).

**Support elements:** 20cm shotcrete, reinforced with wire mesh.
HEB100 steel sets per round length.
15-19 post grouting (PG) rock bolts, 4m long.

**Monitoring system:** Convergence stations.

Four tunnels where excavated completely in soil terrace deposits, below there is a brief summary of each tunnel characteristics and relevant remarks documented during construction.

**Tunnel 7:** The 197m long tunnel with a maximum overburden of 65m, was the first tunnel constructed in terrace deposits at the Bogota-Villavicencio road. High uncertainty on the ground behaviour and lack of experience for tunnelling in this ground led to important support system changes during construction. Implementation of heavier steel sets (HEB160) took place at the beginning of the excavation with a mean separation between steel sets of 0.75cm; this change consequently increase the excavation area from 106m² to 112m² and a 25cm shotcrete layer accordingly to the steel set depth. Additionally, ground improvement made used pressures up to 20Bar, aiming for an improvement on the engineering behaviour of the matrix portion of the material.

Initially, changes proposed by the contractor where accepted by the client, considering a low lateral overburden, deriving on increasing stresses on the sidewall with low confinement (see Fig. 8). Later, technical discussions and a back-analysis study on the ground behaviour leaded to a successful implemented of the original design on approx. 70m of the tunnel. Pre-support on tunnel 7 was completed after 12 months.
Tunnel 11 and tunnel 12: Tunnel 11 and 12 were constructed on the same terrace deposit located between km55 to km56.5. Tunnel 11 and tunnel 12 have a length of 412m and 45m, respectively. Both tunnels faced different challenges during construction, as shown in Fig. 9.

Construction of Tunnel 11 started after tunnel 7 was completed. The contractor implemented the experience gathered during previous construction. Remarkable changes were documented during construction, heavy steal arches and ground improvement was only used on portal areas, as seen in Fig. 10 (b), leading to the
implementation of the designed support system on approximately 75% of the tunnel’s length. Evidence of self-support given by the ground was observed; as a consequence, in approximately 50% of the tunnel’s length, no grouting was used through the umbrella pipes, HEB 100 arches were used, and lengths of about >1.5m where executed.

Although Tunnel 12 is the shortest tunnel in the project, it was probably one of the most challenging. The complexity, both for design and construction, came from loss of ground confinement due to very low overburden on the right sidewall of the excavation (max. overburden 11m). As seen in Fig. 11 (a), there was a high risk of road closure, if the tunnel’s adjacent slope fails. Consideration regarding high concentration of stresses between the tunnels sidewall and the adjacent slope and seismic effects due to the low overburden (Hashash Y., Hook J., Schmidt B. and I-Chiang J (2001), “Seismic design and analysis of underground structures” Tunneling and Underground Space Technology, Vol. 16, -.), were assessed for the design.

Before the tunnel excavation started, the adjacent slope was stabilized with active anchors, rock bolts and shotcrete. Systematic ground improvement was used during the tunnel excavation along its perimeter; grouting with pressure up to 15 bar was applied (see Fig. 11 and 오류! 참조 원본을 찾을 수 없습니다.). Additionally, the whole tunnel was built using heavy steal sets (HEB160) and umbrella pipes on the top heading. The above-mentioned conditions, made tunnel 12, the most expensive tunnel per meter on the Bogota – Villavicencio road. Tunnel 12 was finished after 7 months.

(a) Entrance portal (b) terrace deposit

Fig. 10. Tunnel 11, Bogota - Villavicencio road
Tunnel 13: During the design, a large portion of tunnel 13 was classified as rock, however, during construction it was seen that the geophysics results and their interpretation, mislead the characterization of the material. This 680m long tunnel had a particularity within the terrace deposits documented on the Bogota – Villavicencio road. During the construction of the tunnel occasionally blocks with sizes over 1.5 m were found close to the rock formation (phyllites). Seismic refraction results documented high velocities, which is characteristic in rocks, however, during a post analysis it was seen that the geophones location and the large blocks within the ground, wrongly indicated rock presence at the tunnel’s level.

Tunnel 13 was the last tunnel on the road to be excavated in terrace soil deposits, due to a complex topographical portal situation the tunnel only had one drift for the excavation. Previous experiences played an important role to complete the tunnel in approximately 13 months. Considering drift situation, tunnel 13 recorded the fastest excavation rate under terrace deposit conditions. Additionally and opposite to the other tunnels, this tunnel was constructed with a shotcrete final lining, which was accepted by the owner based on a technical report which included a complete back analysis of the whole structure, considering the support system (ground improvement, support elements, excavation sequences) implemented during construction and calibrated through displacements and strains measured on the shotcrete (see Fig. 12). It is worth mentioning that the back analysed E-modulus complies with the one used during the design stage (back analysed E-modulus ~750MPa).
5. REVIEW AND FINAL REMARKS

Along this document, it has been evidenced how the initial designs and the construction methods can easily become not appropriate and adjustments are needed when tunneling in terrace soil deposits. Tunnelling, in general, requires engineers to be able to continuously learn during construction. All applied techniques shall be up to date and must be fed with all practical knowledge gained through the construction stage. Therefore, data monitoring and evaluation during tunnelling in such soils must be consistent so that it allows the system behaviour to be analysed and optimized. This information thoroughly studied allows the engineer to optimize the excavation and support methods for the encountered ground conditions, resulting in a more economic and safe excavation.

Designs will be successful when adequate ground investigation and material characterization techniques are used. Adequate ground and material characterization allows the tunnelling engineer to take advantage of an eventual soil strength or deformation parameters increase due to an adequate volumetric block proportion evaluation in order to produce less conservative designs.

Sampling or the application of geophysics for ground investigation in terrace soil deposits is uncertain due to the heterogeneity and high stiffness contrast between matrix and blocks in such soils. The ground investigation is mostly carried out before the construction stage (i.e. sampling in portal areas), and when the construction starts, it is the time for tunnelling engineers to verify the ground properties, the influencing factors, and the system behaviour with those assessed during the design stage. Here is when numerical simulations (i.e. back analysis) allow for the designs and some details in the construction method to be calibrated. Clearly, the accuracy during the design stage must be as high as possible, always keeping in mind the uncertainties that the
tunnelling engineer is dealing with.

6. ACKNOWLEDGMENTS

This paper was supported by the National Research Foundation (NRF) of Korea under Research Project 2015R1A2A1A05001627. The Colombian Consulting firm EDL SAS and the concessionaire Coviandes SAS supplied all the Geological and excavation information for the tunnels reported. The authors greatly appreciate the support provided.

7. REFERENCES


