Geomechanics and Engineering, *Vol. 15, No. 3 (2018) 899-910*
DOI: https://doi.org/10.12989/gae.2018.15.3.899

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

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(Received June 1, 2017, Revised February 12, 2018, Accepted March 23, 2018)

Abstract. Terrace deposits are often encountered in portal areas and tunnels with low overburden. They are challenging to excavate considering their great mechanical and spatial heterogeneity and a very high stiffness contrast within the ground. Terrace deposits are difficult to characterize, considering that samples for laboratory testing are almost unfeasible to obtain, and laboratory tests may not be representative due to scale effects. This paper presents the approach taken for their characterization during the design stage and their posterior validation performed during construction. Lessons learned from several tunnels excavated on terrace deposits on the Bogota-Villavicencio road (central-east Colombia), suggest that based on numerical simulations, laboratory testing and tunnel system behaviour monitoring, an observational approach allows engineers 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, back analysis, observational method

1. Introduction

Rivers may form different deposit types whose engineering behaviour depends on the size and nature of the placed materials. Soil deposits generated due to particle deposition are called alluvial soil deposits, one special alluvial deposit of engineering interest is the so-called terrace soil deposit (Maher 2015). Terrace formation occurs mainly in river valleys (also by glacial deposition) and it is controlled by several factors whose contributions are still difficult to determine and understand. 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, it creates meanders, reaches greater depths and a new floodplain is formed, ultimately transforming the existing floodplain into a terrace. This process happens over time and many terraces can be formed, therefore, the higher a terrace is, the older it is (Tevelev 2014). This kind of soil deposits are often involved with civil engineering projects such as earth dams, highways and railways (after proper compaction), high buildings whose foundations are generally piles due to the insufficient shear strength of the soil deposit, and tunnels, which are the focus of this paper.

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 in order to update geological and geotechnical models. Among them are: exploratory drillings ahead of the excavation, identification of structural features at the tunnel face through image analysis, lithological observation and water inflow evaluation. 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 (Schubert and Riedmüller 2000). All the above activities are based on the principles of the observational method, leading to a safe and economical tunnelling activities, 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 paper introduces a state of practice regarding tunnelling in terrace soil deposits. Moreover, a project involving multiple tunnels in the new Bogota-Villavicencio road in Colombia is being used as a case study. The approach taken to optimize the excavation and support methods for the encountered ground conditions of the project is presented below.

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http://www.techno-press.org/?journal=gae&subpage=7
ISSN: 2005-307X (Print), 2092-6219 (Online)

2. Tunnelling terrace soil deposits

2.1 Bimrocks approach for soil terrace deposits

The approach proposed by Medley (1994) 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 geological 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 (Medley 2007). In general, the term block-in-matrix or "Bimrock", as defined by Medley (1994), is also used to describe alluvial terraces with a relevant volumetric block proportion.

To focus on the fundamental engineering problems related to the characterization of these and many other "rock/ soil" mixtures, Medley (1994) 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 2007). 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 1994, Lindquist and Goodman 1994). 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 2007).

Lindquist (1994), Lindquist and Goodman (1994) and Goodman and Ahlgren (2000) determined that the overall strength of a bimrock is related to the volumetric proportions of the blocks, establishing that below ~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 a matter of scale. Medley (1994) 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 suggests: "*…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 *et al.* 2002). This behaviour can be considered one of the most critical situations and is often associated with more severe overbreaks or top heading collapses (Dissauer *et al.* 2002).

2.2 Back analysis

The idea of a back analysis procedure is to vary soil parameters and hypotheses (numerically expressed) so that the results of the analysis match a predicted performance of the soil (i.e., tunnel walls) as much as possible. 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), which is necessary to resolve the numerical modelling with minimum error, in the shortest possible time and with the lowest possible cost.

On the other hand, the use of monitoring instrumentation has expanded as the utilized instruments have become more precise, reliable and sufficiently robust to be used in the hostile environment of a tunnel (Oreste 2005). Other analysis techniques are also used during the design stage, considering a nearby structures (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 parameters estimations (design stage) used for the ground (Oreste 2005). Oreste (1997) stated that, 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.

Generally, there are two different approaches in a back analysis procedure; the inverse approach and the direct approach. In the inverse approach the input parameter in the governing equations of the hypothetical numerical model is the known performance and the solution gives the original parameters. It is worth mentioning that this approach should be applied when the model is simple enough to get inverted, and the experiment execution must be adequately controlled. On the other hand, the direct approach is more adaptable to typical projects due to it involving several unknown and non-linear governing equations. This approach is known as the minimization method, in which a minimization of the error between the predicted and measured performance is possible through numerical processes.

Vardakos (2007), reported several considerations that designer and constructor usually take into account in back analysis methods for optimal tunnel design. There are approaches that can take part in tunnelling methods such as deterministic and probabilistic approaches. For tunnelling, where it is not possible to control completely the measuring methods and therefore high precision measures are not available, the probabilistic approaches are more appropriate (Gioda 1985). This approach is known as the Bayesian approach.

The evolution of the state of the art in back analysis procedures started in the decade of 1980 where the main discussion was related to what should be included in a back analysis procedure. Concepts were introduced by different researchers. The maximum shear strain concept, useful in the estimation of the plastic region around tunnels, was introduced by Sakurai and Abe in 1981. Sakurai *et al.* (1985) later introduced another concept known as the critical strain. Another introduced concept in that work was the equivalent elastic modulus. Later, the research focused on determining adequate numerical methods for different back analysis approaches that minimize the differences (error) between predicted and measured performance in tunnels. For example, Ledesma *et al.* (1996) and Swoboda *et al.* (1999) contribute to the state of the art through probabilistic approaches and the suggestion of the use of the boundary control methods, respectively. The new millennium brought with it a revolutionary calculation in back analysis procedures by combining numerical analysis techniques and a neural network (Feng *et al.* 2000). Other back analysis approaches which related to neural networks were developed by different researchers such as Deng and Lee (2001), Pichler *et al.* (2003), Feng and An (2004), Chua and Goh (2005), Fino and Calvello (2005), Shahrbanouzadeh *et al.* (2015), Gao and He (2017) and Kao *et al.* (2017).

Taking into account that several tunnels of the project were built on terrace soil deposit, it was deemed appropriate to use Medley's approach (1994), which is useful to characterize strength properties for these kind of deposits. However more complex constitutive models require deformation parameters in order to simulate the tunnel

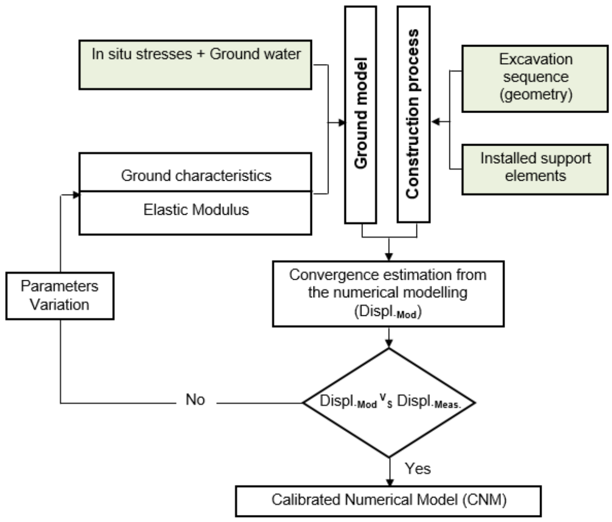


Fig. 1 Flow chart: Mumerical model calibration.

behaviour (Jing and Hudson 2002). Considering the complexity of such deposits, it is not feasible to obtain deformation parameters from the preliminary geological studies, the geotechnical and geophysical explorations or even from laboratory testing. Only during the tunnel construction, or from a pilot tunnel, is it possible to obtain a complete evaluation of the behaviour of the rock mass (ITA 2009).

2.3 Applied procedure

Due to the natural complexity of the soil deposits involved, complementary techniques, such as back analyses, were applied for the terrace soils characterization, complementing the procedure shown on section 4.2. The characterization of the materials and the improvement of the numerical model involved the following steps:

a) Characterization of the material: Shear strength parameters were evaluated following the techniques explained in section 4.2.

b) Definition and evaluation of the main variables influencing the tunnel behaviour: the stress state was calculated (as a function of the soil cover and the morphology of the area), the excavation geometry (excavation sequences), groundwater conditions (established from the ground investigation) and the installed support elements during construction.

c) Back analysis: a numerical model for the evaluation of the problem is proposed.

d) Analysis of the measured excavation behaviour and changes to the support systems: the Adjusted Numerical Model (ANM) allows simulation of complementary works or adjustments to the support systems of the tunnel.

From steps a) and b), the initial soil shear strength and compressibility parameters are obtained. A Geotechnical model is also used for the numerical simulation of the tunnel behaviour (step d). The numerical model used commercial software (Phase2-Flac3D). The implemented back analysis method over the different tunnels studied in this paper (see Section 4.4) was developed in two main stages:

• Development of the Initial Numerical Model (INM): Elastic perfectly plastic constitutive models were used (i.e., Mohr-Coulomb).

• Back analysis (adjustment of the INM): the INM is adjusted to fit the convergence data measured on the tunnel.

The flow chart presented in Fig. 1 shows the procedure followed. In the figure, shaded boxes are related to the conditions relevant for the INM. Back analysis allows for the variation of the Elastic Modulus in a cyclic process involving comparison of the results obtained from the numerical simulation ($Disp._{Mod}$) and the measured actual displacements ($Disp._{Meas.}$). After several calibration cycles, a Calibrated Numerical Model (CNM) is obtained. It allows for the adequate analysis of the construction process and the selection of any additional complementary works.

3. The Bogota-Villavicencio road: Its beginning and upgrading

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. 2 shows the project location. With a length of approximately 86 km, the road starts at the border between the urban and the rural area in the south side of Bogota and ends at the entrance of Villavicencio city.

The corridor in which the road is located, is characterized by a highly heterogeneous geology mainly composed of different soil deposits and sedimentary and metamorphic rocks, immerged in 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 2004), to tragic consequences as recorded in June 1974 when a landslide in Quebrada Blanca buried approximately 300 people (El Tiempo 1999).

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 and the presence of soft sedimentary rocks (sand, mud and clay stones) and thick colluvial deposits. The second third section of the road, which is the focus of this paper, has an extension of approximately 29 km (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.

Fig. 2 Location of the road Bogota-Villavicencio on the Map of Colombia (Central Intelligence Agency 2017)

An initial major road intervention took place between 1995 and 2002. During this period the Colombian government signed two 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 4.4 km tunnel (Buenavista tunnel). Additionally, the government signed another concession contract with CoviAndes for the first third, which included the construction of a 2.4 km tunnel (Boqueron tunnel). The road intervention 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 has experienced a traffic volume increase of approximately 6% every year since 2005. This may also be due to the more stable social and political conditions on the area and the oil production increase 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 15.4 km) and 46 bridges (total length of 5.2 km).

Terrace deposits along the road were found to have 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 230 m, as it was found on exploratory drillings performed near the place called the Santandereana Ridge.

Fig. 3 Tunnelling in terrace soil deposits, tunnel 13 Bogota-Villavicencio road

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 mainly consisted of sand-clay soil, showing low signs of consolidation. Block sizes vary from 0.4 m to 0.8 m in diameter, and occasionally blocks with an average size of above 1.5 m (see Fig. 3) can be found. Blocks are mainly formed of phyllites, sandstone or metasandstone, with different degrees of compaction and weathering, depending on the age and origin of the deposit.

Due to the presence of steep slopes and the tectonism characteristic found in the area, a combination of geological events is often found overlapping terrace and colluvium deposits with a higher clay content 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). 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 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 to the initial design and construction method (Sakurai *et al.* 2003).

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 tunnel collapses (Button *et al.* 2002). Typically, during the tunnel design, 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, in the area of intervention; 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, were initially investigated through geological mapping, followed by a second stage in which core drilling was executed, with the aim 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

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

SG	2.773
LL (%)	NL
PL (%)	NP
Particle size distribution	
Sieve	Percentage passing (%)
3” (75 mm)	100.00
4 (4.75 mm)	43.60
10 (2 mm)	36.30
40 (0.425 mm)	26.00
200 (0.075 mm)	16.60
Shape Index of soil particles	
EI (%)	28.43
FI (%)	31.88

*SG: Specific gravity of soils; LL: liquid Limit; PL: Plastic Limit; EI: Elongation Index; FI: Flattening Index

Table 2 Summary of tested sample characteristics in one dimensional consolidation

Characteristics	C1	C2	C3	C4
FWC (%)	17.64	17.40	16.97	17.50
IUW (kN/m3)	13.47	16.26	16.70	17.75
FUW (kN/m3)	21.52	21.15	21.67	22.79
IVR	1.04	0.69	0.65	0.55

FWC: Final Water Content; IUW: Initial Unit Weight; FUW: Final Unit Weight; IVR: Initial Void Ratio

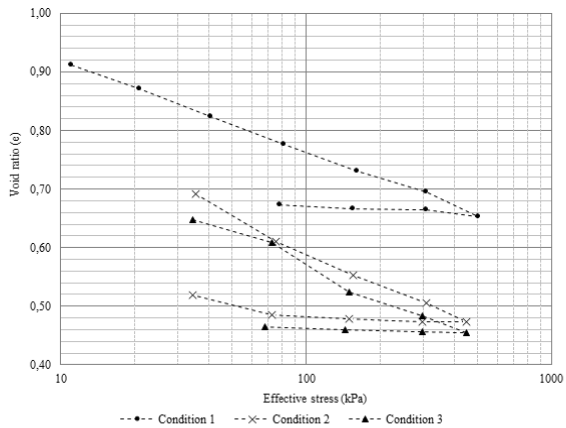


Fig. 4 Compressibility curves

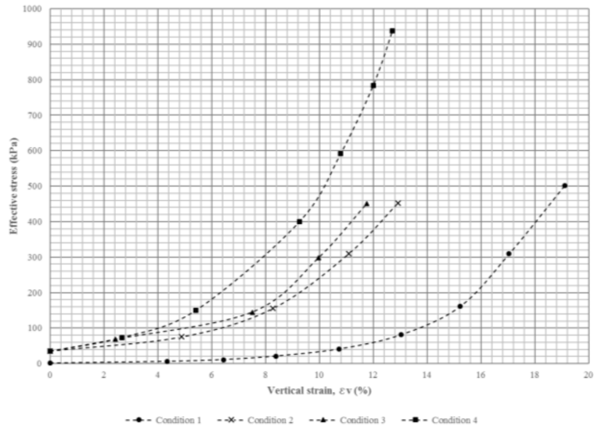


Fig. 5 Stress-Strain relationship of the samples studied

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 types of grounds, taking into consideration the high stiffness variation within the deposit and radical recorded variations in results due to the geophone arrangement and source location.

4.2 Geomechanical characterization

Representative matrix samples of the terrace soil deposits were taken from tunnel portals on the second third of the road. They correspond to a brown-olive silty gravel containing some blocks that measure higher than 3” in size. Particle size distribution of the matrix sample was determined. Additionally, shape index, specific gravity, and Atterberg limits were measured. The results for the materials corresponding to Tunnel 12 are presented in Table 1.

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 in internal diameter, were used. Samples with particles smaller than 1” 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). The maximum vertical stresses selected to establish the deformation modulus correspond to Tunnel 12 where there is a low ground cover level. Initial and final Particle size distribution of the material was evaluated in order to verify particle crushing. Results confirmed that no significant crushing was taking place under this level of confinement.

Fig. 4 shows the compressibility curves evaluated from the samples tested. It can be seen that the initial density has a major effect on the compressibility of the granular material. The sample prepared to the lowest density (condition 1) presents a higher void ratio in comparison with the other samples. The compressibility curve in all cases displays a straight path both in loading and unloading paths. This is the result of particle reorganization while a new fabric is developed. Such behaviour is typical of materials with high permeability as those found in this project.

Fig. 5 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.

Considering the oedometer testing results and assuming Poisson’s ratio values between 0.20 and 0.35, the Elasticity Modulus was estimated for each load increment. The effect

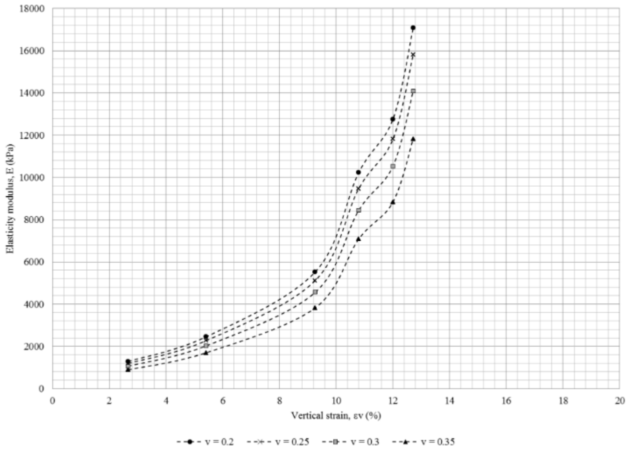


Fig. 6 Estimation of the influence of the Poisson's ratio (condition 4)

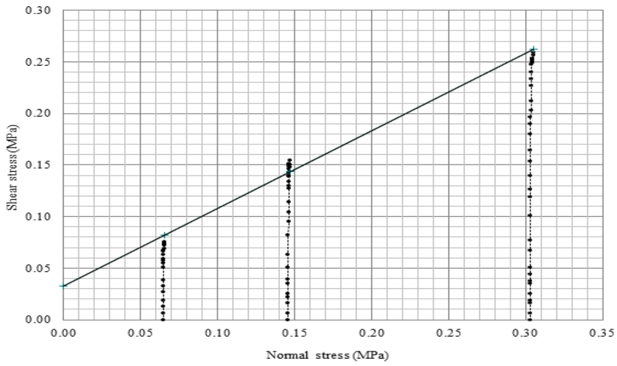


Fig. 7 Stress paths on a σ-τ diagram (condition 1)

Table 3 Direct shear testing

Sample type	Direct shear test		
	IUW (kN/m^3)	c' (MPa)	ϕ (°)
1	13.02	0.033	36.9
2	16.45	0.013	35.7
3	16.66	0.010	42.3
4	17.33	0.000	39.1

IUW: Initial Unit Weight

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. 6, for the densest condition tested.

Additionally, direct shear tests, using medium size shear boxes, were performed on samples prepared according to the characteristics shown in 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. 7 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 from 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.

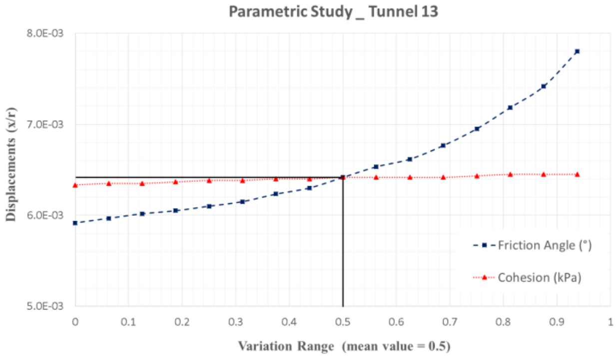
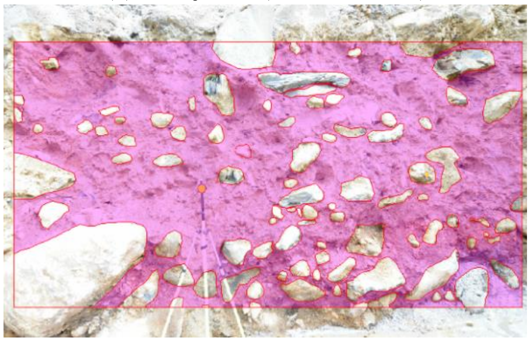


Fig. 8 Tunnel 13, Influence of the block proportion on the shear strength parameters and tunnel deformation

(a) Analysis 1 (VBP = 40.6%)



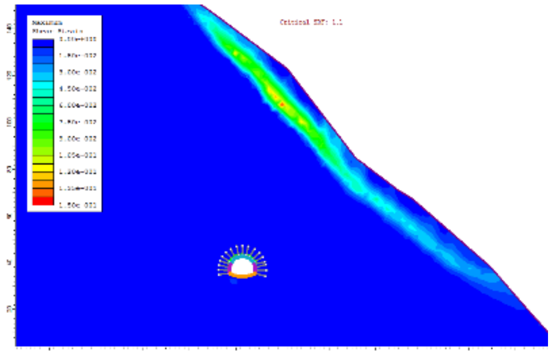
(b) Analysis 2 (VBP = 30.3%)

Fig. 9 Volumetric block proportion (VBP) at the entrance of tunnel 12

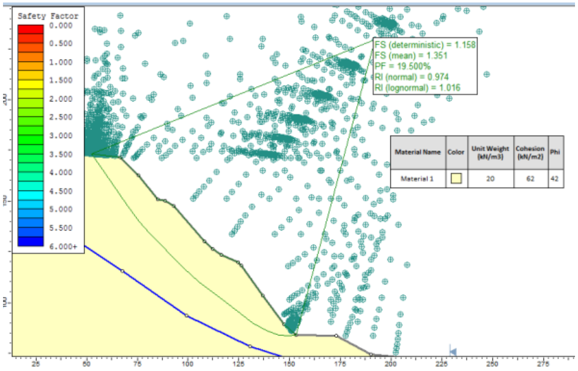
Presumably the presence of rock blocks also influence the compressibility as it was found from the back analysis reported below where the Elasticity modulus was estimated to be 1 to 2 orders of magnitude higher than the measured for the granular matrix. The Poisson's ratio effect was found not to be significant (from a practical engineering point of view), as there is no significant variation in 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 with the one expected for granular materials, considering the particle size distribution.

4.3 Strength parameters

Strength parameters were assessed following Medley's



(a) Tunnel 13, back analysis for deformability parameters



(b) Tunnel 12, back analysis to check strength properties

Fig. 10 Back analysis performed at the Bogota-Villavicencio project

approach (1994). 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 as the characteristic engineering dimension. Block proportion was determined using the software ShapeMetrix3D, which acquires surfaces with three-dimensional images. Fig. 9 shows an example of the surfaces taken from tunnel 13.

For this example, it was determined that tunnel 13 had a 35% volumetric block proportion, which according to Lindquist (1994), represented and increment of 5 degrees on the friction angle, resulting in an increase from 37° to 42°. The elasticity modulus was assessed based on a back analysis (see section 4.4) approach resulting in an E-modulus equal to 750MPa for the material related to this tunnel.

4.4 Parametric study

A parametric study was conducted for the tunnels to evaluate sensibility results when varying the input parameters. Fig. 8 shows the assessment for tunnel 13.

Lower and upper limits were set according to laboratory test results performed on the matrix and the approach shown in section 4.3. Strength parameters were calculated, in which the friction angle varies depending on the Volumetric Block Proportion (VBP between 20-75%). The parametric study had as a reference value, the on-site measured displacement at the tunnel crown and the mean Volumetric Block Proportion, measured in the tunnel. The figure shows a large influence of the friction angle, complying with the theory proposed by Medley (2007).

4.5 Numerical simulations

Defining ground properties that accurately represents the ground behaviour within a Mohr Coulomb constitutive model, was a challenge. The approach was based on the characterization proposed by Medley (2007) and Medley and Lindquist (1995). 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, as explained in section 2.1. 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 by the use of finite element numerical modelling or limit equilibrium analysis (see Fig. 10). The procedure followed was previously explained in section 2.3. Fig. 10(a) shows the back analysis methodology used, in order to determine the deformability parameters and Fig. 10(b) shows the back analysis for shear strength parameters corresponding to Tunnel 12 portal analysed by limit equilibrium methods.

4.6 Designed support systems and their implementation during construction stage

Following the selected design approach (OeGG 2010), two main requirements were established, based on the material characterization and expected ground behaviour; (a) low deformability capability of the ground, and (b) proper behaviour during seismic events, which are important for tunnels with low overburden. Therefore, the support system was conservatively designed relying mainly on support elements and not on the ground contribution to stability. The designed support system for Ground Class "Terrace deposits", included:

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

• Excavated cross section: 102 m^2 to 106 m^2.

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

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

• Monitoring during construction: it played a major role and was divided into geological documentation and convergence stations installed every 15-20 m. Convergences were measured using extensometers tapes and high accuracy total stations (angle accuracy: +/- 2''_0,3 mgon-distance accuracy +/- 2 mm + 2 ppm), such accuracy was needed considering that convergences were on a 1 to 3cm range. A strict control was made in order to perform a proper cero reading, highly important in order to register proper time vs. displacement and face distance vs. displacements curves.

Four tunnels where excavated completely in soil terrace

Fig. 11 Low overburden on entrance portal. Tunnel 7, Bogota-Villavicencio road

Fig. 12 Location of tunnel 11 ant tunnel 12, Bogota-Villavicencio road

(a) Entrance portal

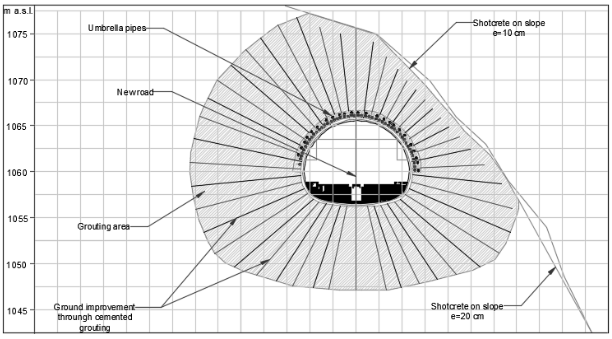
(b) Implemented support system

Fig. 13 Tunnel 11, Bogota-Villavicencio road

deposits. Below, is a brief summary of each tunnels' characteristics and relevant remarks documented during construction.

4.6.1 Tunnel 7

(a) Entrance portal



(b) Systematic ground improvement

Fig. 14 Tunnel 12, Bogota-Villavicencio road

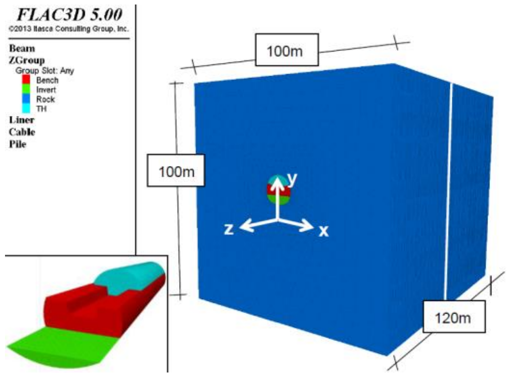
The 197 m long tunnel with a maximum overburden of 65 m, was the first tunnel constructed in terrace deposits at the Bogota-Villavicencio road. High uncertainty on the ground behaviour and lack of experience for tunnelling in this ground led to important support system changes during construction. Implementation of heavier steel sets (HEB160) took place at the beginning of the excavation with a mean separation between steel sets of 0.75 cm; this change consequently increased the excavation area from 106 m^2 to 112 m^2 and a 25 cm shotcrete layer accordingly to the steel set depth. Additionally, implemented 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 were accepted by the client, considering a low lateral overburden, deriving on increasing stresses on the sidewall with low confinement (see Fig. 11). Later, technical discussions and a back analysis study of the ground behaviour, according to the above explained methodology leaded to a successful implementation of the original design over approximately 70 m of the tunnel. Pre-support on tunnel 7 was completed after 12 months.

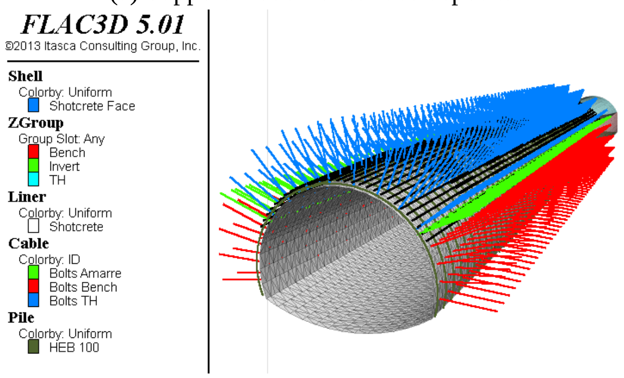
4.6.2 Tunnel 11 and tunnel 12

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 412 m and 45 m, respectively. Both tunnels faced different challenges during construction, as shown in Fig. 12.

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



(a) Support model for terrace deposits



(b) Calibration based on ground displacements

Fig. 15 Back analysis performed during construction (tunnel 13, km 57+302)

and ground improvement were only used in portal areas, as seen in Fig. 12(a) and 12(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 Tunnel 11's length, no grouting was used through the umbrella pipes, HEB 100 arches were used to replace the effect of grouting effect, and excavation lengths greater than 1.5 m 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 a very low overburden on the right sidewall of the excavation (max. overburden 11m). As seen in Fig. 14(a), there was a high risk of road closure, if the tunnel's adjacent slope fails. Considerations regarding high concentration of stresses between the tunnels sidewall and the adjacent slope and seismic effects due to the low overburden (Hashash *et al.* 2001), were assessed for the design.

Before the tunnel excavation started, the adjacent slope was stabilized through 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. 14(b)).

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.

4.6.3 Tunnel 13

During the design, a large portion of tunnel 13 was classified as rock, however, during construction it was found that the geophysics results and their interpretation, mislead the characterization of the material. This 680 m long tunnel had a particularity within the terrace deposits documented on the Bogota-Villavicencio road. During the tunnel construction, 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 of rocks, however, during a post analysis it was found 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 and due to a complex topographical portal situation the tunnel only had one drift for the excavation. Previous experiences played an important role in completing the tunnel in approximately 13 months. Considering the drift situation, tunnel 13 recorded the fastest excavation rate under terrace deposit conditions. Additionally and in contrast to other tunnels, this tunnel was constructed with a shotcrete final lining This was accepted by the owner based on a technical report which included a complete back analysis of the whole structure, as was previously explained, considering the support system (ground improvement, support elements, excavation sequences, etc.) implemented during construction and calibrated through displacements and strains measured on the shotcrete. Fig. 15 shows some displays of the 3D numerical modelling related to the back analysis performed for the system in Tunnel 13 in order to fit the measured displacements. It is worth mentioning that the back analysed E-modulus complied with the one used during the design stage (back analysed E-modulus ~750MPa).

5. Conclusions

Throughout this document, it has been demonstrated how the initial designs and construction methods can easily become inappropriate and adjustments are needed when tunnelling 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 throughout 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 thoroughly investigated information 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 a less conservative design.

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 prior to 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, influencing factors, and system behaviour with those assessed during the design stage. Here is when numerical simulations (i.e., back analysis) allow the designs and some details in the construction method to be calibrated. Clearly, accuracy during the design stage must be as high as possible, always keeping in mind the uncertainties that the tunnelling engineer is dealing with.

Acknowledgements

This paper was supported by the National Research Foundation (NRF) of Korea under Research Project 2015R1A2A1A05001627 and the Korea Agency for Infrastructure Technology Advancement under the Ministry of Land, Infrastructure and Transport of the Korean government (17SCIP-B066321-05). The Colombian Consulting firm EDL SAS and the concessionaire Coviandes SAS supplied all reported Geological and excavation information for the tunnels. The authors greatly appreciate the support provided.

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