Analysis of the radial and tangential soil pressure on pipes installed by the pipe jacking method in the Aburrá Valley

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2019

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In Partial Fulfillment of the Requirements for the Degree
Mcs. Engineering (Soil mechanics and foundations)

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2019

ACKNOWLEDGMENTS

I want to dedicate this work to the most important people in my life, my parents, my wife and our expected daughter.

I would like to thank to Empresas Públicas de Medellín and their professionals who helped me with the information about pipe jacking projects developed in the Aburrá Valley and several technical aspects of this work.

Special thanks to professor Silvana Montoya Noguera for her support and recommendations during the development of this work.

Summary

In the pipe jacking design, it is required the estimation of radial and tangential pressures exerted by soil on pipes, to determine the jacking forces that need to be achieve to overcome the frictional resistance during the installation process.

This document presents theoretical methods that can be used to compute the external pressures on pipes and jacking forces, especially in cohesionless soils where the frictional component is determinant in the jacking process. In addition, several analyses are presented of how these calculations are affected by the variability of the geotechnical parameters of soils in the Aburrá Valley.

Additionally, a comparison between computed values and field data recorded from two projects developed in the Aburrá Valley are presented, in order to analyze how well the computed values, approximate the real data.

The comparison showed that theoretical models tend to overestimate the real behavior recorded in the projects, however, this overestimation provide a safety factor that must be optimized by the designer of a future pipe jacking project.

Keywords:

Pipe jacking, radial and tangential soil pressures, jacking force, geotechnical parameters, frictional resistance, Aburrá Valley.

Table of Contents

List of tables

List of figures

List of symbols and abbreviations

Symbol	Definition	Units	Equation
A_c	Contact area between pipe and soil	m	
b	Width of the contact area between pipe and soil	m	2.2
B	Width of affected ground	m	2.12, 2.13
c'	Effective cohesion of the soil	kPa	
D_c	Internal diameter of the tunnel	m	
D_e	External diameter of the pipe	m	
D_i	Internal diameter of the pipe	m	
C_e	Ratio between Poisson ratio for the soil/pipe and Elastic moduli of the soil/pipe	1/kPa	2.4
E_p	Elastic moduli of the pipe	kPa	
E_s	Elastic moduli of the soil	kPa	
$f'c$	Compressive strength of concrete	MPa	
F_r	Frictional resistance per unit length	kN/m	2.1, 2.7
g	Gap between pipe and soil (in diameter)	m	
H	Depth of the pipe crown	m	
H_1	Depth of the water table	m	
K_o	Coefficient of earth pressure		
L_u	Length of a single pipe	m	
L_t	Total length of pipe between shafts	m	
k_d	Ratio between external diameter of the pipe and internal diameter of the tunnel	m	2.3
P_r	Normal load on pipe induced by the soil	kN/m	2.6
P_u	Contact force between pipe and soil per unit length	kN/m	
q	Surcharge on the ground surface	kPa	
R	External radius of the pipe	m	
S_u	Undrained strength of the soil	kPa	
t	Pipe thickness	m	
W	Weight of the pipe per unit length	kN/m	
z	Depth from the ground surface	m	

Symbol	Definition	Units	Equation
α	Angle at any point of the circumference of the pipe	º	
γ	Bulk unit weight of the soil	kN/m^3	
γ'	Submerged unit weight of the soil	kN/m^3	
γ_w	Unit weight of water	kN/m^3	
δ	Angle friction between pipe and soil	º	
λ	Reduction factor for effective vertical stress on pipe caused by soil		2.15
λ_o	Reduction factor for surcharge		2.16
ν_p	Poisson ratio for pipe		
ν_s	Poisson ratio for soil		
σ_r'	Effective radial stress on pipe by the soil	kPa	2.5
σ_h'	Effective horizontal stress in soil	kPa	2.11
σ_v'	Effective vertical stress in soil	kPa	2.9
τ	Shear stress in soil	kPa	
Φ'	Effective internal angle of friction of the soil	°	

INTRODUCTION

The installation of new pipelines in urban environments is a complex activity because not only there is not enough space in the cities, which brings problems with the pedestrian and vehicular traffic, but also because of the numerous intersections of different utilities in the underground (pipes of water, sewer, gas, electrical conduits and optical fiber, etc.). For this reason, engineers have been developing new methods like trenchless technologies. This development includes manufacturing new materials for pipes and their joints, sophisticated microtunneling equipment, inspection technologies and renewal methods in all areas of water, sewer, gas and other applications.

The International Society for Trenchless Technology (ISTT, 2018), defines trenchless as "*all the techniques that involves the installation, replacement or renewal of underground utilities with minimum excavation and surface disruption*". Those techniques can be used for all underground utilities.

One of the most interesting methods of trenchless is "pipe jacking". This technique is a good option for new installation of pipes in big cities because it causes minimum disturbance during construction that represents an advantage for the traffic flow causing small or no impact to the local economy in the project area. Furthermore, pipe jacking is an environmental friendly method compared with the conventional trench excavation for pipe installation, because it reduces the quantity of the excavated material and it has a low impact on the existing infrastructure. In addition, the pipe jacking method has a minimum noise impact in the project area (ISTT, 2018).

Due to the technical, environmental and social advantages of pipe jacking, this method is becoming increasingly important for the installation of new service pipes and sewer pipes throughout the world. The Aburrá Valley in Antioquia department of Colombia is one of the metropolitan areas where this technology is being used.

The pipe jacking method requires a good knowledge of the geological and geotechnical conditions, because the tunnel excavation can produce, among others, soil deformations, settlements, tunnel instability, ground closure and local interface stresses between the pipe and soil (radial and tangential soil pressure on pipes). These problems need to be studied and must be properly solved especially in urban areas, where the excessive deformation and settlements in the soil could cause building foundation damages, rupture of existent water pipes, sewer, gas or electrical conduits, damages in road pavements and impacts on the pipe jacking installation process (Milligan & Norris, 1993).

The purpose of this document is to analyses how the radial and tangential soil pressure impact the pipe jacking installation process, especially for projects that could be planned in the future in different areas in the Aburrá Valley. This topic is important for a good planning, design and construction process.

The document is divided in five chapters. The first one presents the objectives, the general context of the pipe jacking method and its construction procedure. In chapter two, a literature review is presented about the distribution of normal and tangential stresses on pipes due to the soil pressure. The third chapter is about the Aburrá Valley geotechnical soil conditions and the pipe jacking projects developed in the Aburrá Valley in recent years. Chapter four presents the effect and influence of geotechnical parameters in the calculation of soil pressures on pipes and the jacking forces required for installation. The fifth chapter summarizes the results of chapter four and presents some practical examples in the Aburrá Valley for pipe jacking projects using geotechnical information presented in chapter three. Finally, conclusions and recommendations are presented in order to have information that help designers and planners to compute an approximated value of jacking forces required in a future pipe jacking project in the Aburrá Valley.

CHAPTER 1

1.1 Objectives

General objective:

Analyze how the geotechnical parameters of soils in the Aburrá Valley impacts the calculation of radial and tangential pressures on pipes installed by the pipe jacking method.

Specific objectives:

- Study theoretical models to calculate the radial and tangential stresses on pipes due to the soil pressure during pipe jacking installation.
- Review of geotechnical soil conditions in the Aburrá Valley and the pipe jacking projects developed in recent years in this metropolitan area.
- Analyze the influence and variability of geotechnical parameters on the pressure distributions on pipes installed by the pipe jacking method and the jacking forces required for installation.
- Present a methodology for the calculation of the jacking forces for different soil types, pipe diameters and installation depths in future projects of pipe jacking in the Aburrá Valley.
- Conclusions and perspectives for future projects and investigation about the pipe jacking in the Aburrá Valley.

1.2 Motivation for study

Medellín is located in the Aburrá Valley in the center of the Antioquia department of Colombia. This city is part of a group of ten municipalities called "the metropolitan area of the Aburrá Valley" (Área Metropolitana del Valle de Aburrá - AMVA, 2006). The municipalities are: Caldas, La Estrella, Sabaneta, Itagüí, Envigado, Bello, Copacabana, Girardota, Barbosa and Medellín.

The total area of the metropolitan area is about 1165 km^2 (including rural and urban areas), its population in 2018 was 3.72 million and the projected population in 2030 is about 4.5 million people (Departamento Administrativo Nacional de Estadística (DANE) , 2018). In the last 15 years, the local public service company for the Aburrá Valley called *Empresas Públicas de Medellín* (EPM) has been working in the use of trenchless for water and sewer projects. EPM has received many suppliers of these technologies and has developed projects like Centro Parrilla, which is a rehabilitation and renovation project of the Medellin downtown networks of sewerage and potable water, using trenchless techniques like: horizontal directional drilling, pipe ramming, cured in place pipe (CIPP), pipe bursting, close fit sliplining and pipe jacking (Montoya, 2017).

Rehabilitation and new installation of pipes in Aburrá Valley using trenchless technologies, especially pipe jacking, requires a good estimation of ground deformations and soils pressures on pipes, because the tunnel stability can cause sudden collapses that may damage tunneling machinery, ground movements above the pipe may cause damage in existent building foundations, road pavements and other services pipes (Milligan & Norris, 1993). In addition, a ground collapse onto the pipe will increase the resistance to jacking and lead excessive jacking forces during construction.

Due to the increasing use of these trenchless technologies in the Aburrá Valley, there is a necessity to understand different technical aspects of trenchless technologies, in order to have valuable information that help planners and designers of water and sewer services to use these techniques in future projects in the metropolitan area.

This document is focused in a specific topic of the pipe jacking method applied to the local conditions of soils in the Aburrá Valley, which is the evaluation and analysis of the radial and tangential soil pressures for different soil types.

It is important to evaluate the variation of the soil pressures in different soil types because the stability of the tunnel depends on the type of material excavated. For example, granular materials whether in dry or fully saturated condition are considered unstable as the hole will tend to close and collapse onto the pipe. In contrast, in fine soils the cohesion between particles can help to maintain temporary stability of the hole during the construction process (Milligan and Norris, 1999).

The estimation of the radial and tangential soil pressures allows determining the jacking forces involved in the installation process. In turn, the quantification of these forces is essential in the different phases of pipe jacking projects – planning, designing and constructing (Staheli, 2006). For instance, the definition of the distance between the launch and the receiving shaft depends on the jacking forces. In addition, the hydraulic jacks capacity for pushing the pipes depends on those forces. Finally, the need of intermediate jacking stations depends on the magnitude of the jacking forces and the distance between shafts.

1.3 General context of pipe jacking method

According to the classification of The International Society for Trenchless Technology (ISTT, 2018), pipe jacking is a new installation of trenchless technique, as is shown in Figure 1.

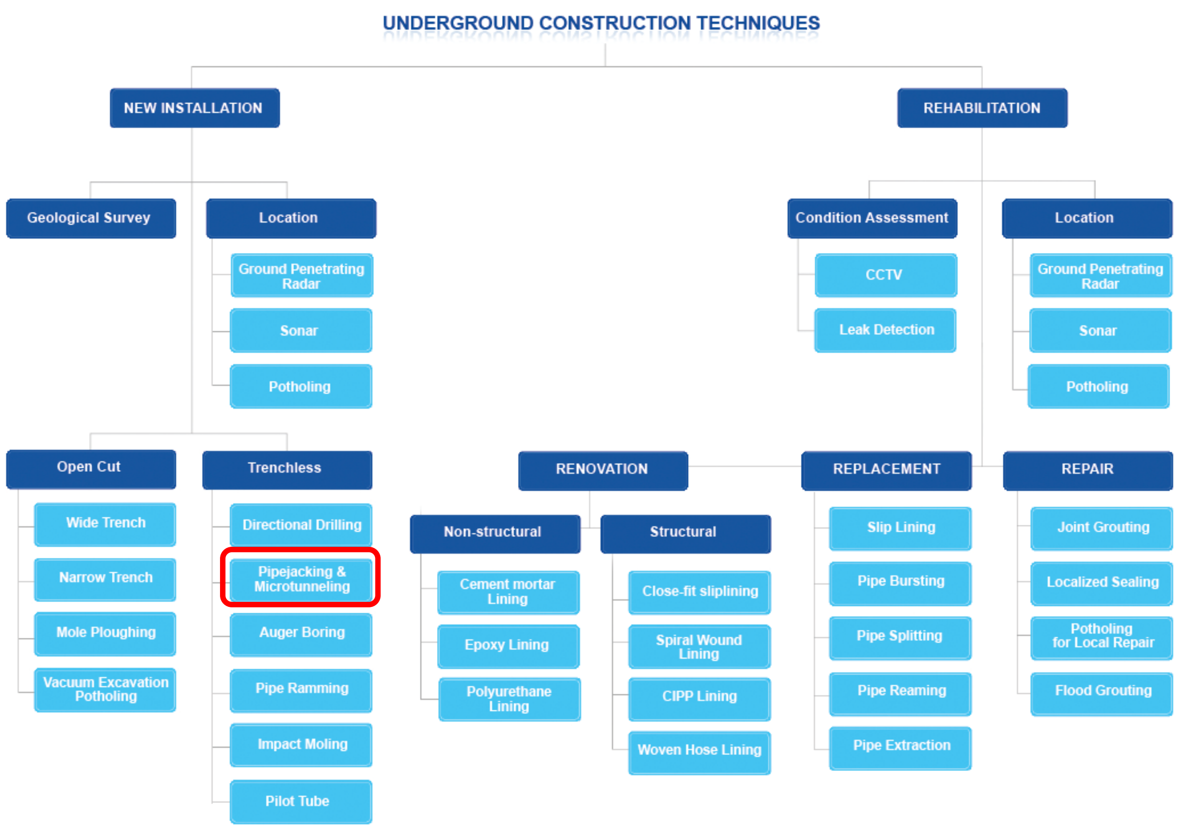


Figure 1. Underground construction techniques, modified from: ISTT (2018)

Pipe jacking is a trenchless method for installation of new pipes, and it is differentiated from conventional tunneling by the soil support structure. In pipe jacking methods, the pipes are installed at the same time the soil is excavated. In contrast, in a conventional tunnel, first, the tunnel is excavated and the internal perimeter of the hole is lined with a liner system (prefabricated concrete plates, steel ribs, etc). After the excavation and installation of the liner system, the pipe sections are transported one by one and installed into the tunnel. Both pipe jacking method and conventional tunneling can use the same excavation system such as the tunnel boring machine (TBM) or the earth-pressure balance machine (EPBM) (Najafi, 2010).

1.4 Pipe jacking general procedure

In this method of installation, a prefabricated pipe sections are jacked or pushed behind the TBM or EPBM, and one of its principal advantages is that during the excavation process no personnel are required into the tunnel, therefore the risk of accidents is minimized considerably.

Pipe jacking starts with the construction of a launch shaft and a receiving shaft. These are constructed to the designed pipeline depth. The distance between shafts depends on the pipe diameter and the force required to overcome the forces associated with face pressures on the machine and friction between soil and pipe (Staheli, 2006).

In the launch shaft, a jacking frame and the hydraulic jacks for pushing the pipes are installed. Additionally, a thrust block in concrete is built on the wall of the shaft for receiving the stress from the hydraulic jacks.

Some auxiliary components like a discharge pump, a laser guidance system, conveyor and supply pipes are placed into the launch shaft. Some components for the work are placed outside of the launch shaft, for instance: a control container where the operator controls the alignment of the boring machine and the hydraulic jack, a spoil removal system (conveyor belt and haul units), and a sedimentation tank for excavated material storage.

The pipes are introduced into the launch shaft and the hydraulic jacks push them into the tunnel behind the boring machine (see Figure 2). One common practice to help the pipe slip easily through the tunnel is to place grout plugs in each section of pipe, these plugs are used to pump lubricants around the outside of the pipe during the jacking operation.

Sometimes it is possible to use intermediate jacking stations that can be incorporated into the pipe at a distance behind the microtunneling machine. This intermediate jacking station can help to reduce the total jacking force of the main hydraulic jack located in the launch shaft. The pipes sections are then pushed between the intermediate station and the tunneling boring machine.

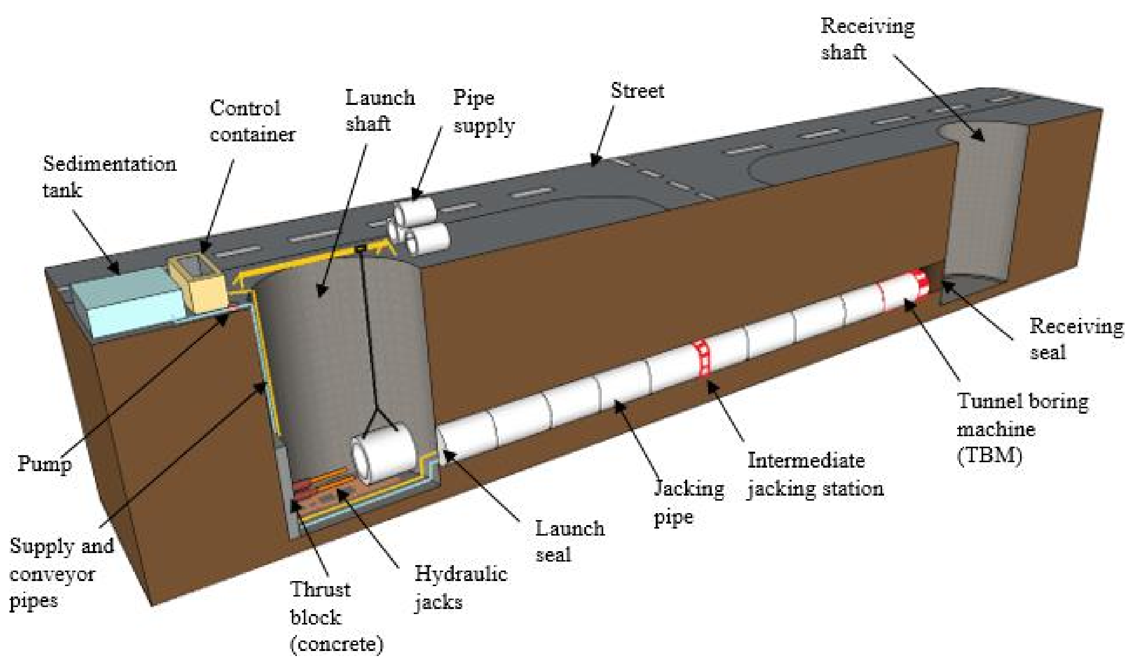


Figure 2. Pipe jacking scheme. Modified from (Staheli, 2006)

The external diameter of the intermediate jacking station must be equal to the external diameter of the pipe, and the number of these stations needs to be computed according to the jacking forces to overcome (Staheli, 2006). Normally, there is a gap between the external diameter of the pipe and the face of the excavated hole made by the (TBM or EPBM). This gap is referred to the over cut or annular space (Reyes, 2017).

Pipe jacking typical diameters of installation range between 0.40 m and 3.05 m. However, diameters are not limited to those values, because this type of technology is being developed every day and can be adapted according to the need. Some authors (Ueki, M; Hass, C; Seo, J, 1999) have proposed general criteria based in economic aspects to select the most convenient method of pipe installation. Figure 3, shows the combination of pipe diameter and installation depth that make the trenchless technology like pipe jacking economically more advantageous.

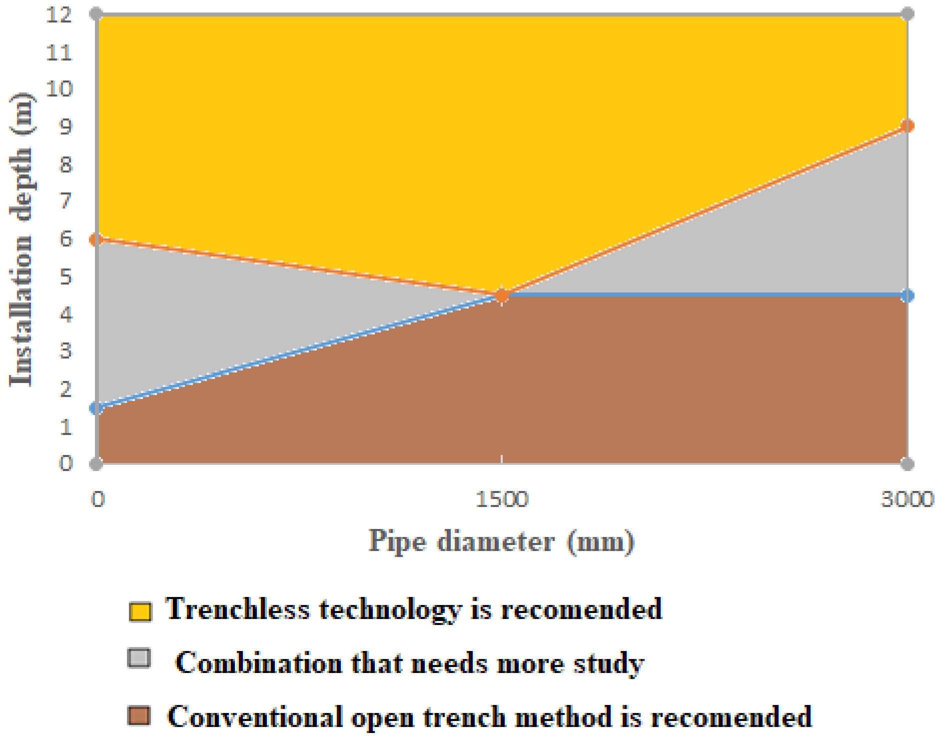


Figure 3. Criteria to select conventional open trench or trenchless installation. Modified from (Balairón Pérez, 2002)

CHAPTER 2

2.1 Theoretical models

2.1.1 The jacking forces

The jacking force that needs to be applied in pipe jacking is composed by two components: the face pressure acting in the front of the excavation due to the soil and water (if the tunnel is under the water table), and the second component is the tangential pressure due to the friction between the soil and pipe (see Figure 4).

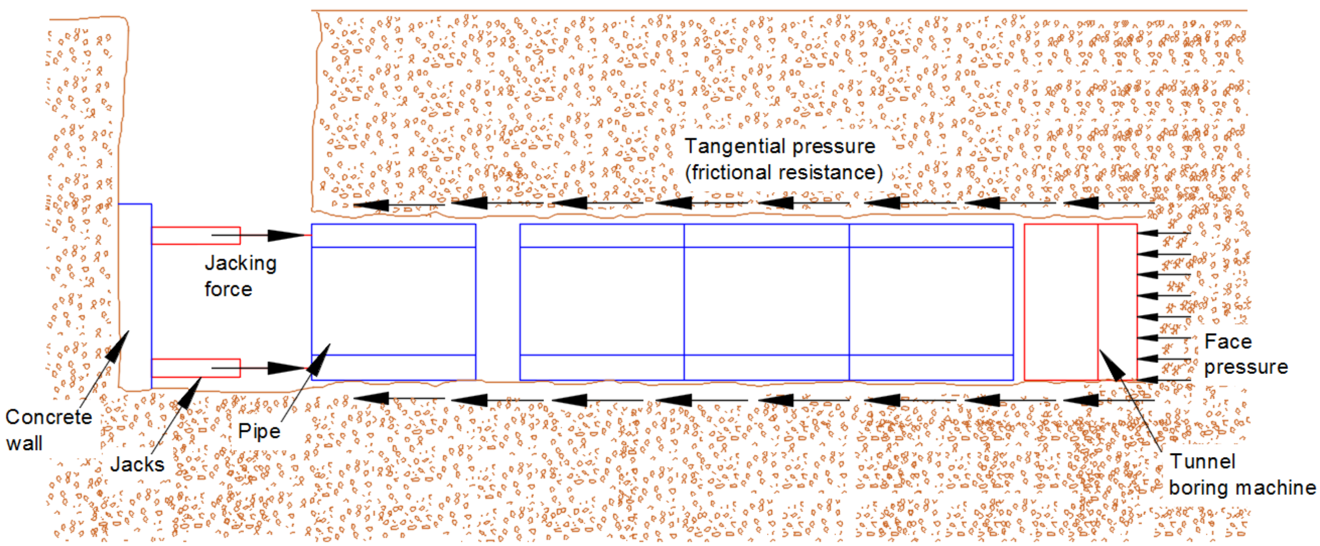


Figure 4. Jacking forces during pipe Jacking. Modified from (Staheli, 2006)

The face pressure is balanced by pumping slurry to the front of the tunnel-boring machine to contain the groundwater and earth pressure. This equilibrium depends on the machine operation and its advance speed. If the operational speed is high, the face pressure will increase and the machine may be damaged. On the other hand, if the speed is too slow the material at the front may slide into the excavation. Because of this, the face pressure acting on the TBM or EPBM shield remains between the active and passive earth pressure.

In general, projects experiences around the world has shown that face pressure represents a low contribution in the total jacking force, and this condition is adjusted during construction by the machine operation, who is responsible for the speed and advance rates (Staheli, 2006).

Tangential pressure or frictional resistance is the second component in the jacking force evaluation. This is the resistance caused by the friction between the soil and pipe and its magnitude depends on the tunnel stability.

2.1.2 Tunnel stability

Depending on the material excavated, the opening in the soil may be stable or unstable. If the soil is stable, the excavation will stay open temporarily and the pipes can slide into the tunnel, but if the soil is unstable, the soil can collapse onto the pipe. As a result, a radial soil pressure is generated and the resistance to jacking will increase (see Figure 5).

In pipe jacking installation, it is convenient to have a larger diameter in the excavation hole respect to the external pipe diameter, the reason of this is to allow the pipe to slide easily into the tunnel. This gap usually is about 10-20 mm in diameter (Milligan and Norris, 1999).

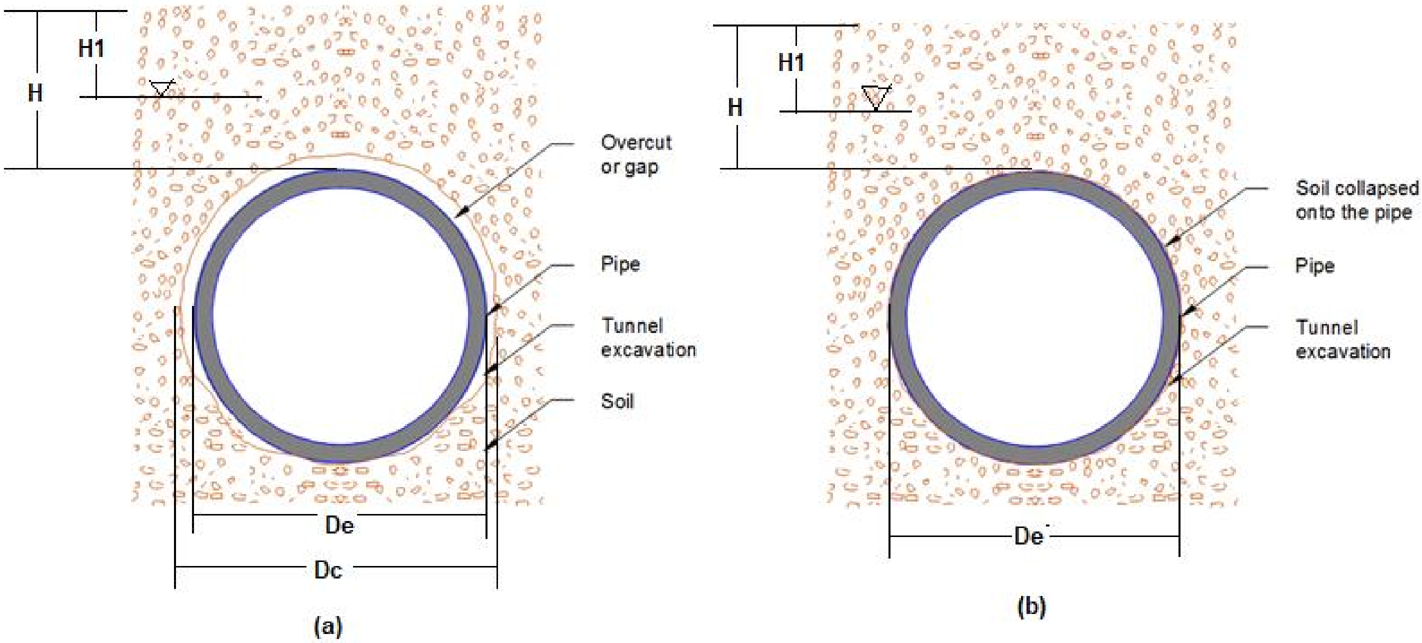


Figure 5. (a) Pipe on stable soil. (b) Pipe on unstable soil. Modified from (Milligan and Norris, 1999)

Notation:

D_c: Internal diameter of the tunnel

D_e: External diameter of the pipe

H: Depth of the pipe crown

H_1: Depth of the water table

Bentonite is injected for lubrication in stable cases, to facilitate the slide of the pipe surface into the tunnel. This bentonite must be design and its injection pressure needs to be controlled to ensure the stability of the excavation.

When the soil collapses onto the pipe, the use of bentonite has less effect in lubrication, and depending on the soil type it may cause problems for the tunnel stability.

2.1.3 Stable soils

Tunnels excavated in clay soils tend to be stable due to cohesion, and it will stay open temporally, so the pipes can slide easily along the tunnel. In this case, there are two models of analysis to calculate the frictional resistance.

The first model, and the most accepted, assumes that the contact between the pipe and the soil is purely frictional and the resistance is given by the Newton's law of friction:

$$F_r = W \cdot \tan(\delta) \qquad (2.1)$$

Where:

Fr: Frictional resistance per unit length (kN/m)

W: Weight of the pipe per unit length (kN/m)

δ: Angle of friction between pipe and soil (º)

To illustrate this, consider a project with the following information:

Table 1. Pipe and soil information for a case in stable soils using the Newton´s law of friction

Pipe		
Material	Reinforced concrete (f'c= 42 MPa)	
Internal diameter	D_i	1.50 m
Pipe thickness	t	0.15 m
Weight per unit length of pipe	W	17.88 kN/m
Length of a single pipe	L_u	2.50 m
Length of pipe between shafts	L_t	100 m
Soil		
Type of soil along the pipe alignment	Medium-stiff clay	
Angle of friction between pipe and soil	δ	19°

Frictional resistance is computed according to equation (2.1):

$$F_r = W \cdot \tan(\delta)$$

$$F_r = 17.88\ kN/m \cdot \tan(19°) = 6.16\ kN/m$$

The force required to move the pipe in the total length between shafts will be:

$$F_{rt} = (6.16\ kN/m) \cdot (100\ m) = 616\ kN$$

In cohesive soils, a portion of the resistance to the movement of the pipe is the adhesion between the pipe and the soil. There is another model proposed by (Haslem, 1986) which assumes a cohesive interaction between pipe and soil, with the resistance per unit contact area related to the undrained strength of soil. Haslem suggested that the width (b) of the contact area between pipe and soil should be the solution for elastic contact between two curved surfaces:

$$b = 1.6(P_u k_d C_e)^{0.5} \qquad (2.2)$$

where:

b: Width of the contact area between pipe and soil (m)

P_u: Contact force per unit length between pipe and soil per unit length (kN/m)

k_d: Ratio between external diameter of the pipe and internal diameter of the tunnel (m) (equation 2.3)

C_e: Ratio between Poisson ratio for the soil/pipe and Elastic moduli of the soil/pipe (1/kPa) (equation 2.4)

D_e: External diameter of the pipe (m)

D_c: Internal diameter of the tunnel (m)

E_s: Elastic moduli of the soil (kPa)

E_p: Elastic moduli of the pipe (kPa)

ν_s: Poisson ratio for soil

ν_p: Poisson ratio for pipe

$$k_d = \frac{D_c D_e}{(D_c - D_e)} \qquad (2.3)$$

$$C_e = \frac{(1-\nu_s{}^2)}{Es} + \frac{(1-\nu_p{}^2)}{E_p} \qquad (2.4)$$

To illustrate the second method, consider the same project described in Table 1. Additionally, consider the following information:

Table 2. Pipe and soil information for a case in stable soils using the Haslem method (Haslem, 1986)

Pipe data		
Material	Reinforced concrete	
Elastic moduli of the pipe (concrete f'c= 42 MPa)	E_p	30500 MPa
Poisson ratio for pipe	ν_p	0.2
Internal diameter of the pipe	D_i	1.50 m
Pipe thickness	t	0.15 m
External diameter of the pipe	D_e	1.80 m
Contact force between pipe and soil per unit length (equal to the weight per unit length of pipe)	P_u	17.88 kN/m
Length of a single pipe	L_u	2.50 m
Length of pipe between shafts	L_t	100 m
Gap between pipe and soil (in diameter)	g	0.05 m
Soil data		
Type of soil along the pipe alignment	Medium-stiff clay	
Elastic moduli of the soil	E_s	15000 kPa
Poisson ratio for soil	ν_s	0.4
Undrained strength of the soil	S_u	20 kPa
Internal diameter of the tunnel	D_c	1.85 m

Computing the parameters k_d and C_e:

$$k_d = \frac{D_c D_e}{(D_c - D_e)} = \frac{(1.85\ m\ )\cdot(1.80m)}{(1.85m - 1.80m)} = 66.60m$$

$$C_e = \frac{(1-{\nu_s}^2)}{Es} + \frac{(1-{\nu_p}^2)}{E_p} = \frac{(1-0.40^2)}{15000kPa} + \frac{(1-0.20^2)}{30500000\ kPa} = {}^{0.000056}/_{kPa}$$

This model assumes that the interaction between pipe and soil is cohesive, with the resistance per unit contact area related to the undrained strength of the soil (Milligan & Norris, 1999).

Then:

$$b = 1.6(P_u k_d C_e)^{0.5} = 1.6\left[(17.88kN/m)\ \cdot(66.6m)\cdot\left({}^{0.000056}/_{kPa}\right)\right]^{0.5} = 0.41m$$

The contact area between the pipe and soil along the entire length between shafts is:

$$Ac = b \cdot L_t = (0.41m)(100m) = 41m^2$$

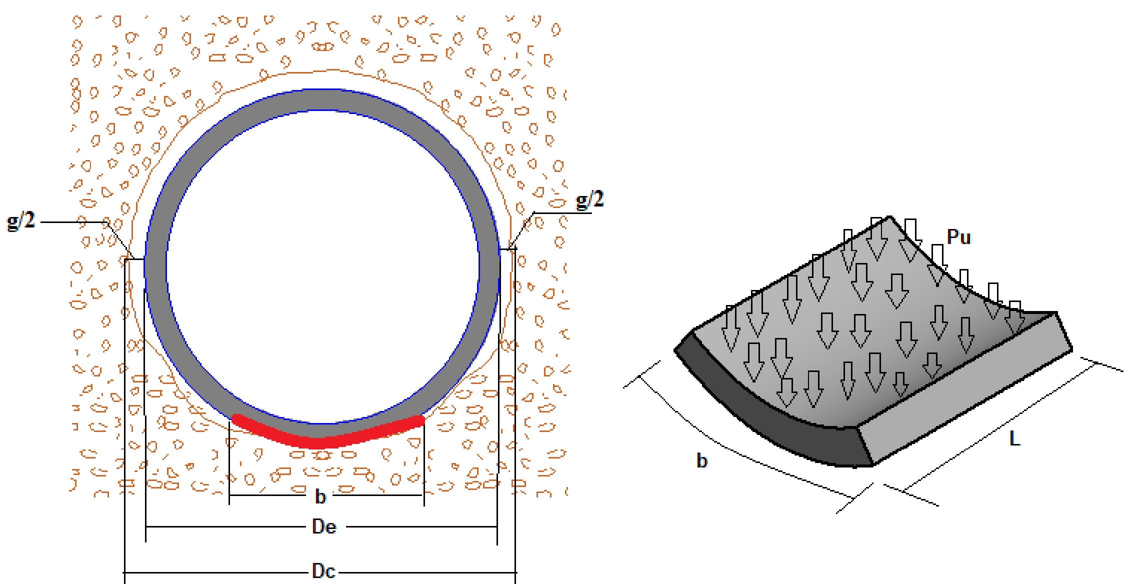


Figure 6. Contact area between the pipe and soil in Haslem method (Haslem, 1986)

The undrained resistance S_u is assumed equal to the cohesion value in the Mohr-Coulomb envelope for total stress. For this example, the value of (S_u) will be 20 kPa.

Then, the force required to move the pipe in the total length between shafts will be:

$$F_{rt} = Ac \cdot S_u = (41m^2)(20\ kPa) = 820\ kN$$

The difference between the two methods for stable soils is because the first model assumes that the contact between pipe and soil is purely frictional and the second assumes that the interaction is cohesive.

It is a normal construction practice to fill the gap between the pipe and soil with bentonite, to reduce the frictional resistance to jacking. Under this condition, the pipe will be partially buoyant and its weight will become smaller than the normal pipe weight.

2.1.4 Unstable soils

When a tunnel is excavated in dry or completely saturated cohesionless soils, the excavation does not have the capability to be stable and the material tends to close around the pipe generating a radial stress, which is normal to the pipe surface. This stress will increase the frictional resistance and the jacking force (Milligan and Norris, 1999).

Radial effective stresses on a pipe due to the imposed soil stresses are given by:

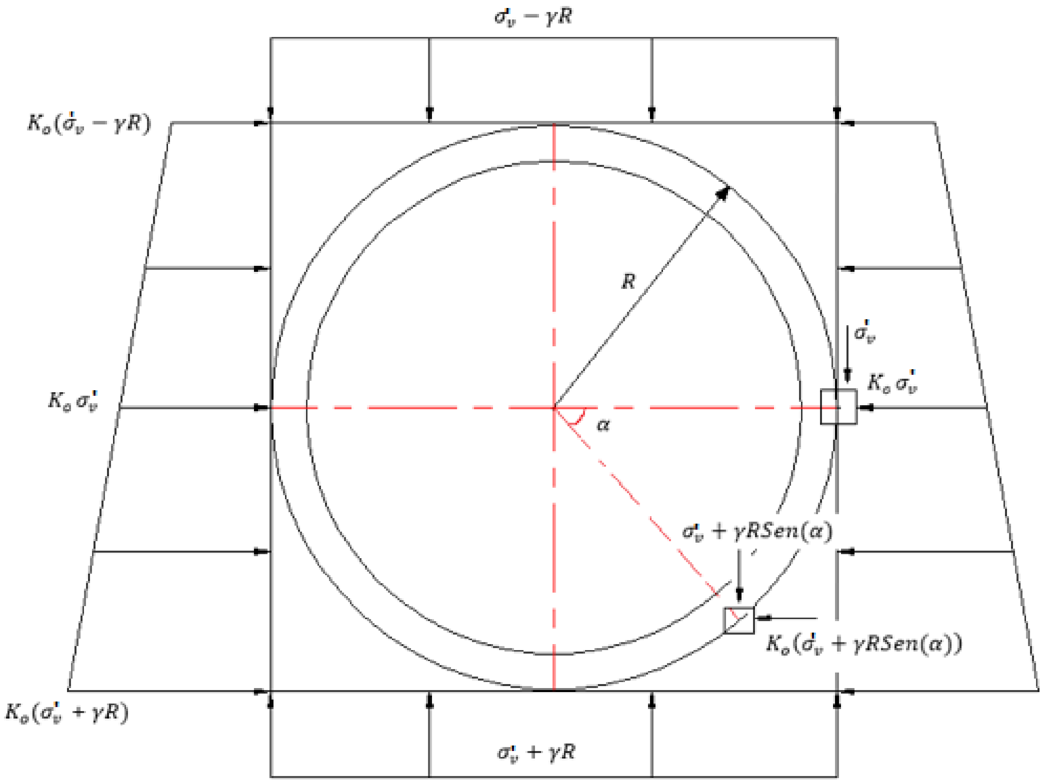


Figure 7. Soil stresses on a pipe. Modified from (Ripley, 1989)

Where:

R: External radius of the pipe (m)

α: Angle at any point of the circumference of the pipe (°)

γ: Bulk unit weight of the soil (kN/m^3)

σ'_v: Vertical effective stress at centroid of the pipe (kPa)

K_o: Coefficient of earth pressure

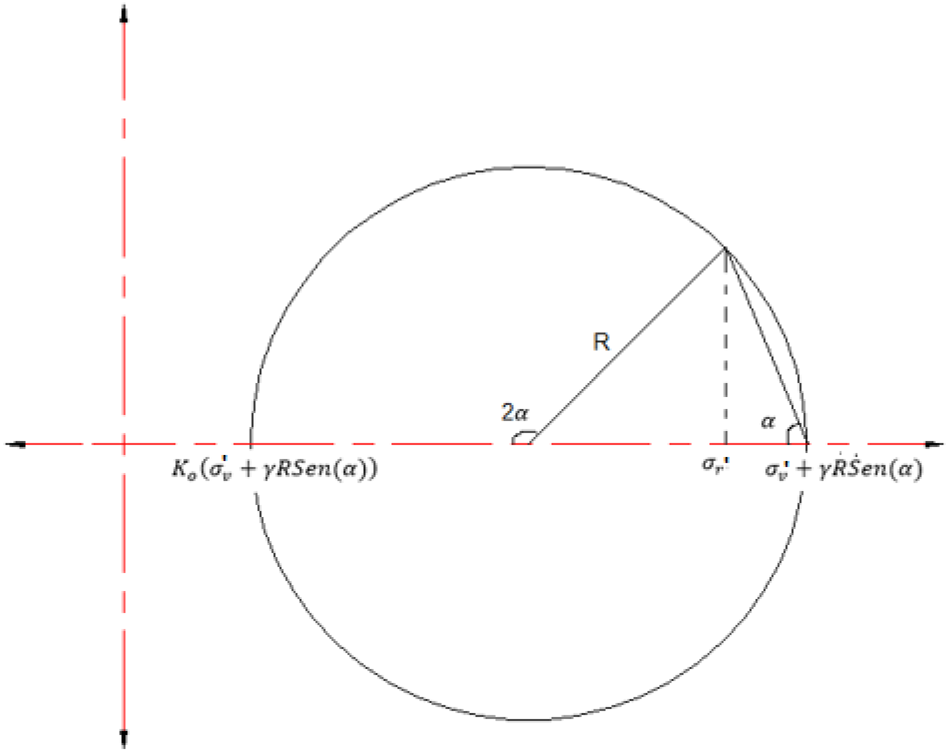


Figure 8. Mohr's circle of stresses on the pipe. Modified from (Ripley, 1989)

Using Mohr's circle to calculate the radial stress at any point:

$$\sigma' r = \left[\frac{(\sigma'_v + \gamma RSen\ \alpha) + K_o(\sigma'_v + \gamma RSen\ \alpha)}{2}\right] + \left\{\left[\frac{(\sigma'_v + \gamma RSen\ \alpha) - K_o(\sigma'_v + \gamma RSen\ \alpha)}{2}\right] Cos(\pi - 2\alpha)\right\}$$

$$\sigma' r = \left(\frac{\sigma'_v + \gamma RSen\ \alpha}{2}\right)(1 + K_o - (1 - K_o)\ Cos\ 2\alpha) \qquad (2.5)$$

Evaluating:

α = π/2, $\sigma'_r = \sigma'_v + \gamma R$

α = 0, $\sigma'_r = K_o \sigma'_v$

α = -π/2, $\sigma'_r = \sigma'_v - \gamma R$

The normal load induced on pipe by the radial stress is given by:

$$P_r = 2\int_{-\pi/2}^{\pi/2} \sigma'_r R d\alpha$$

$$P_r = 2\int_{-\pi/2}^{\pi/2} R\left(\frac{\sigma'_v + \gamma RSen\,\alpha}{2}\right)(1 + K_o - (1 - K_o)\,Cos\,2\alpha)d\alpha$$

$$P_r = R\int_{-\pi/2}^{\pi/2} \sigma'_v(1 + K_o - (1 - K_o)\,Cos\,2\alpha) + \gamma RSen\,\alpha(1 + K_o) - \gamma R(1 - K_o)(Sen\,\alpha\,Cos\,2\alpha)d\alpha$$

$$P_r = R\left[\sigma'_v(1 + K_o)\alpha - \sigma'_v(1 - K_o)\frac{Sen\,2\alpha}{2} - \gamma R\,Cos\,\alpha(1 + K_o) + \gamma R(1 - K_o)\left(\frac{Cos\,3\alpha}{6} - \frac{Cos\,\alpha}{2}\right)\right]$$

$$P_r = \pi R\sigma'_v(1 + K_o)$$

Considering:

$$\sigma'_h = K_o\sigma'_v \therefore R = De/2$$

$$P_r = \tfrac{\pi De}{2}(\sigma'_v + \sigma'_h) \qquad (2.6)$$

P_r is de total force due to the radial stresses caused by soil, and the jacking force required to push the pipe under this radial force is given by:

$$F_r = \tfrac{\pi De}{2}(\sigma'_v + \sigma'_h)\tan\delta \qquad (2.7)$$

Where (δ) is the angle of friction between the pipe and soil.

2.1.5 Prediction of the vertical and horizontal effective stresses on the pipe

Many researchers have been studying the calculation of the stresses acting on the pipe. One of the most accepted theories of how the soil stresses are distributed on the pipe was presented by Auld (1982). Auld presented a model based in the Terzaghi arching theory.

Terzaghi developed a test in which he observed the behavior of a sand in a large box that contained a small trap door in the base. If the trap door is closed, the vertical stress per unit of area on the horizontal support is equal to the depth of the layer of sand times the bulk unit weight of the sand. However, when the trap door is slowly removed at the base of the box, the sand located above the strip started to yield. As a result, the vertical stress decreases an amount equal to the vertical component of the shearing resistance which acts on the boundaries of the yielding zone. Then, the adjoining stationary parts of that yielding zone increases the vertical stress in the same amount (see Figure 9). This transfer of pressure from a yielding mass of soil to the adjoining stationary parts is commonly called the arching effect (Terzaghi, 1943).

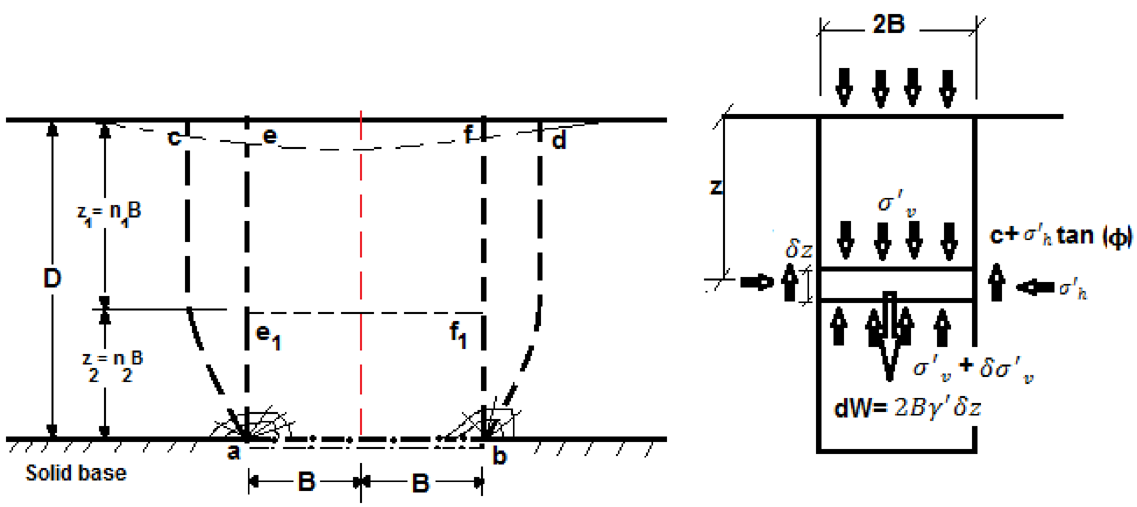


Figure 9. Arching in cohesionless soils. Modified from (Terzaghi, 1943)

Based on the Terzaghi's model, many researchers have used the arching theory to calculate the normal stress acting on the pipe during pipe jacking. Most of the researchers correlated the trap door width to the pipe diameter (Staheli, 2006).

Auld (1982) represented a pipe installed in cohesionless soil that collapsed onto the pipe exerting a normal stress on it (see Figure 10).

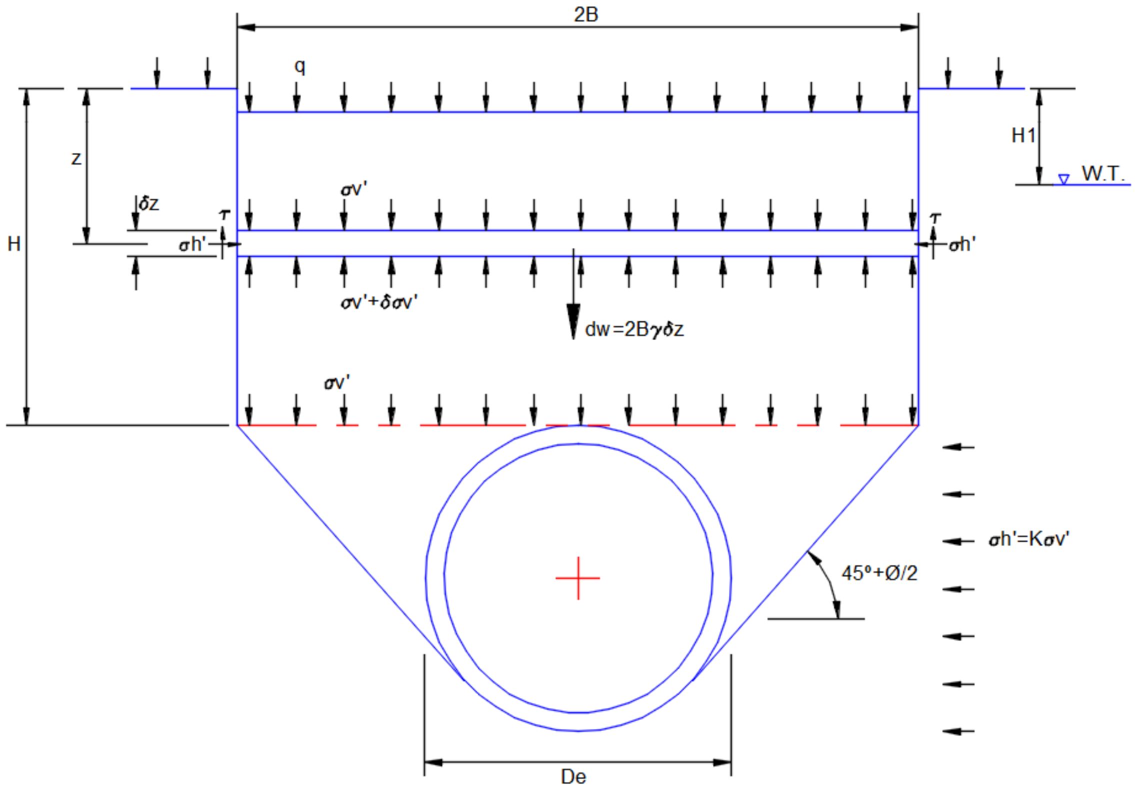


Figure 10. Ground loading from Auld model. Modified from (Milligan and Norris, 1999)

Where:

$\tau = c' + \sigma'_h \tan(\phi')$

$\sigma'_h = K\sigma'_v$

c'= Efective cohesion of the soil (kPa)

q: Surcharge at the ground surface (kPa)

B: Width of the affected ground (m)

Φ': internal angle of friction for soil (°)

γ': Submerged unit weight = $\gamma - \gamma_w$ (kN/m^3)

γ: Bulk unit weight of soil (kN/m^3)

γ_w: Unit weight of water (kN/m^3)

K_o: Coefficient of earth pressure

D_e: External diameter of the pipe (m)

The total force due to the radial stresses is given by equation 2.7:

$$F_r = \frac{\pi De}{2}(\sigma'_v + \sigma'_h)\tan\delta \qquad (2.7)$$

To determine σ_v' and σ_h', a vertical equilibrium of a differential layer of soil below the water table can be done:

$$\sum Vertical\ forces = 0$$

$$2B\sigma'_v + 2B\gamma'\delta z - 2B(\sigma'_v + \delta\sigma'_v) - 2\tau\delta z = 0 \qquad (2.8)$$

Rearranging and substituting equation (2.8):

$$2B\sigma'_v + 2B\gamma'\delta z - 2B(\sigma'_v + \delta\sigma'_v) - 2c'\delta z - 2K_o\sigma'_v\tan(\phi')\,\delta z = 0$$

$$\sigma'_v + \gamma'\delta z - \sigma'_v - \delta\sigma'_v - \frac{c'}{B}\delta z - \frac{K_o}{B}\sigma'_v\tan(\phi')\,\delta z = 0$$

$$\gamma'\delta z - \frac{c'}{B}\delta z - \frac{K_o}{B}\sigma'_v\tan(\phi')\,\delta z = \delta\sigma'_v$$

$$\left(\gamma' - \frac{c'}{B} - \frac{K_o}{B}\sigma'_v\tan(\phi')\right)\delta z = \delta\sigma'_v$$

Solving this equation and integrating:

$$\sigma'_v = \frac{B(\gamma' - c'/B)}{K_o \cdot tan(\phi')}\left(1 - e^{-\frac{(z-H_1)K_o tan(\phi')}{B}}\right) + \sigma'_{v1}\cdot e^{-\frac{(z-H_1)K_o tan(\phi')}{B}} \qquad (2.9)$$

The value of σ'_{v1} is found from a similar analysis for the soil above the water table, using the bulk unit weight of the soil (ϒ) rather than the submerged unit weight (ϒ'):

$$\sigma'_{v1} = \frac{B(\gamma - c'/B)}{K_o \cdot tan(\phi')}\left(1 - e^{-\frac{H_1 K_o tan(\phi')}{B}}\right) + q \cdot e^{-\frac{H_1 K_o tan(\phi')}{B}} \qquad (2.10)$$

σ'_h is given by:

$$\sigma'_h = K_o(\sigma'_v + 0.5\gamma' De) \qquad (2.11)$$

The width of the affected ground at surface is given by:

$$B = \frac{De \cdot \tan(45° - \phi'/2)}{2} + \frac{De}{2 \cdot sen(45° + \phi'/2)} \qquad (2.12)$$

Some other authors have proposed alternative methods for the calculation of the vertical and horizontal effective stresses on the pipe, most of them based on the Terzaghi arching theory, the difference between these methods are the magnitude of the width of the affected ground "B", which varies from one to other researcher (Staheli, 2006).

For instance, the German Standard DWA-A 161 Statische Berechnung von Vortriebsrohren (2014), recommends:

$$B = De \cdot \sqrt{3} \qquad (2.13)$$

$$\sigma'_v = \lambda \cdot \gamma' \cdot (H - H_1 + D_e) + \lambda \cdot \gamma \cdot (H_1 - D_e) + \lambda_o \cdot q \qquad (2.14)$$

$$\lambda = \frac{1 - e^{-2\frac{H}{B}K_0\, tan(\delta)}}{2\frac{H}{B}K\, tan(\delta)} \qquad (2.15)$$

$$\lambda_o = e^{-2\frac{H}{B}K_0 tan(\delta)} \qquad (2.16)$$

$$\sigma'_h = K_o(\sigma'_v + 0.5\gamma' De) \quad (2.11)$$

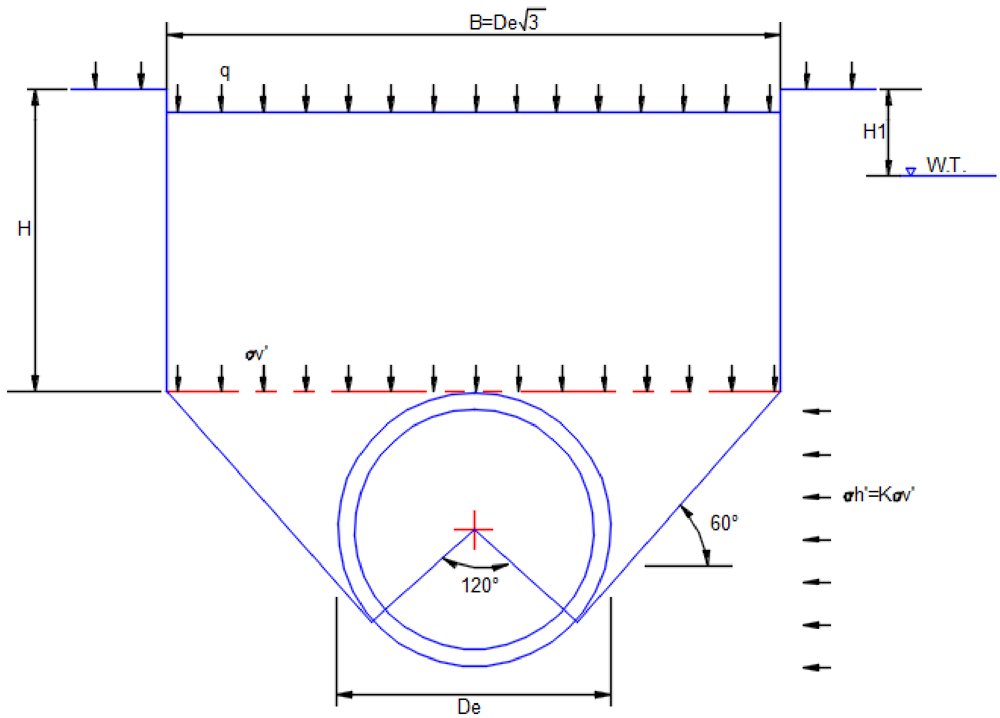


Figure 11. Stress model in DWA-A 161 Standard. Modified from DWA-A 161 (2014)

2.1.6 Angle of friction between pipe and soil (δ)

The value of the angle of interface friction depends on the soil type and the pipe material. There are different materials that are used for pipe jacking around the world: Glass fiber reinforced plastic pipes (GRP), reinforced concrete pipes (RCP), polymer concrete pipe, vitrified clay pipes and rolled steel pipes. Each material has a different surface roughness that affect the interface friction angle and the interaction mechanism between pipe and soil.

It is important to say that the most commonly used materials for pipe jacking in Colombia are GRP pipes and reinforced concrete pipes (RCP) because the availability of manufacturers in the country.

The ground conditions determine the material selection of the pipe, for example, the best soil for RCP pipes is clay, because the internal angel of friction (Φ') is lower than the value of (Φ') in the sand. The higher the percentage of granular materials in the clay (silt or sand), the higher the value of friction and jacking loads on the pipe.

Some researchers have studied the value for the angle of interface friction (δ) between concrete pipe and soils. Tomlinson (1969) presented a table relating the angle of interface friction (δ) at the concrete pipe surface with the soil friction angle in sands (Φ'):

Table 3. Relation ($\boldsymbol{\delta/\Phi'}$) for concrete pipe surface and sand. Tomlinson (1969)

Surface finish	**(δ/Φ`) for dry sand**
Smooth (made in metal formwork)	0.76
Grained (made in timber formwork)	0.88
Rough (cast on ground)	0.98

GRP pipe has an external surface with less roughness than the concrete pipe, because the composition of the materials and the external surface finish of pipe. This condition allows the GRP pipe to slide more easily in granular soils, decreasing the friction resistance force between soil and pipe.

Other researchers argue that the value of (Φ') varies depending on the soil stress changes, and they use a reducing factor to calculate (δ) typically between 1/4 ϕ and 3/4 ϕ (Staheli, 2006).

In general, the selection of the angle of interface friction depends on the type of soil and the pipe material, and a practical value of (δ/Φ') of 0.70 is usually assume by the pipejackers and designers (Ripley, 1989).

Also, the use of bentonite or grout injection into the gap between excavation and pipe is a common practice to improve the lubrication and decrease the friction force. In that case the value of (δ) is affected and in some cases the pipes become buoyant within the fluid.

CHAPTER 3

3.1 Aburrá Valley Geography

The metropolitan area of the Aburrá Valley is an integration of 10 municipalities: Caldas, La Estrella, Envigado, Itagüí, Sabaneta, Bello, Copacabana, Girardota, Barbosa and Medellín. The total area of this zone is about 1165 km^2 (including rural and urban areas).

The Aburrá Valley is in the north of the central mountain range of Colombia. The valley is a natural basin of the Medellín river and can be divided in two principal zones: the first one which is in the north -south direction between Caldas and Bello, and the second one which is in the north-east direction between Bello and Barbosa. The valley is narrow at the south and progressively become wider in the middle. It is surrounded by mountains that vary their altitude approximately between 1300 and 2800 meters above sea level.

In table 2 it is shown the main geographic data of the Aburrá Valley:

Table 4. Aburrá Valley Geographic data. Source: http://datosabiertos.metropol.gov.co/

<table>
<tr><td>Country</td><td colspan="2">Colombia</td></tr>
<tr><td>Division</td><td colspan="2">Antioquia</td></tr>
<tr><td>Name of region</td><td colspan="2">Aburrá Valley</td></tr>
<tr><td rowspan="2">Coordinates</td><td>Latitude</td><td>6° 15' N</td></tr>
<tr><td>Longitude</td><td>75° 33' W</td></tr>
<tr><td>Altitude (average)</td><td>1300 - 2800</td><td>m.a.s.l.</td></tr>
<tr><td>Temperature range</td><td>12 - 30</td><td>˚C</td></tr>
<tr><td>Precipitation (annual)</td><td>1554</td><td>mm</td></tr>
<tr><td>Length (average)</td><td>60</td><td>km</td></tr>
<tr><td>Width</td><td>Max. 30
Min. 8</td><td>km</td></tr>
<tr><td>Area</td><td>1165</td><td>km^2</td></tr>
<tr><td>River name</td><td colspan="2">Medellín River</td></tr>
</table>

Figures 12 and 13 shown the configuration and relief of the valley.

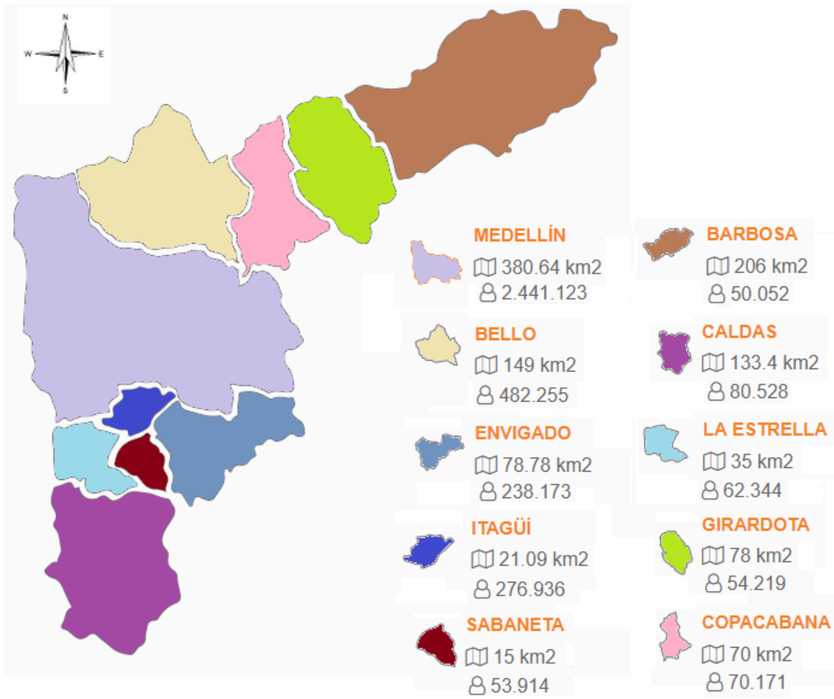


Figure 12. Aburrá Valley configuration. Modified from: http://datosabiertos.metropol.gov.co/

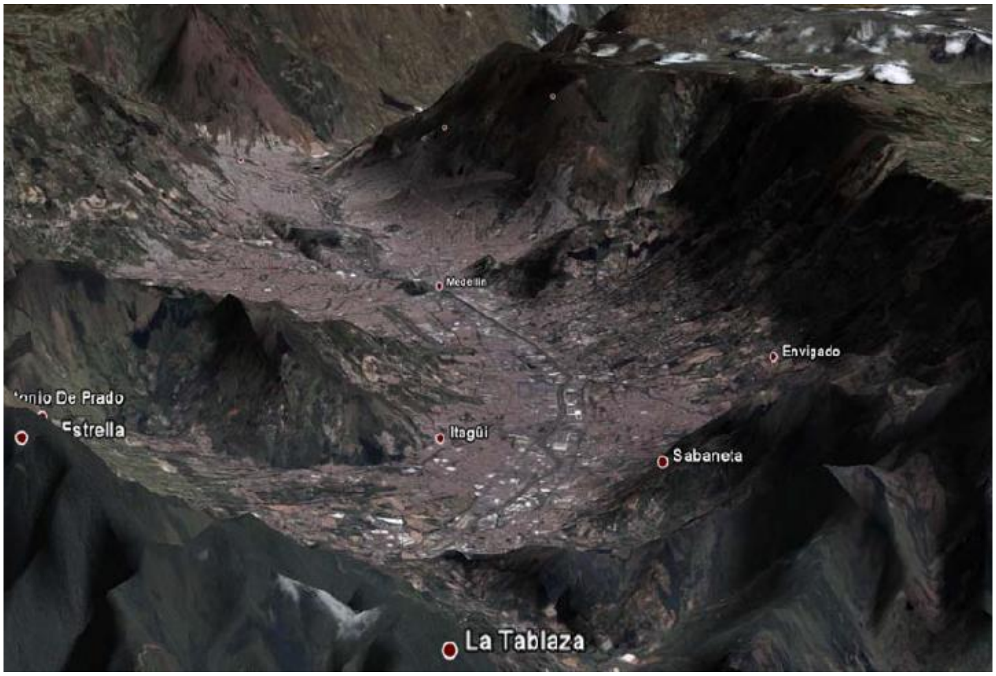


Figure 13. Aburrá Valley relief. Source: (Google maps, 2019)

3.2 Population growth of the Aburrá Valley

Due to the difficult economic and violence conditions suffered by the region of Antioquia in recent years, there has been a phenomenon of migration from rural areas to the metropolitan area of the Aburrá Valley, this is added to the growth of the local industries and the normal increase of the population.

According to Departamento Administrativo Nacional de Estadistica de Colombia (DANE, 2018), the current population of the Aburrá Valley is 3.72 million people. Projections suggest that population will increase to 4.5 million people in 2030.

Taking into account that the population will increase in the next years, and considering the topography conditions, it is necessary to look for alternatives such as trenchless technologies to carry out extensions of the water and sewerage systems for the metropolitan area.

3.3 Geological and geotechnical conditions in the Aburrá Valley

Surface formations of the Aburrá Valley include diverse fluvial and slope deposits in a stepped structure. In the valley, there are several geological and geomorphological units as can be seen in Figure 14.

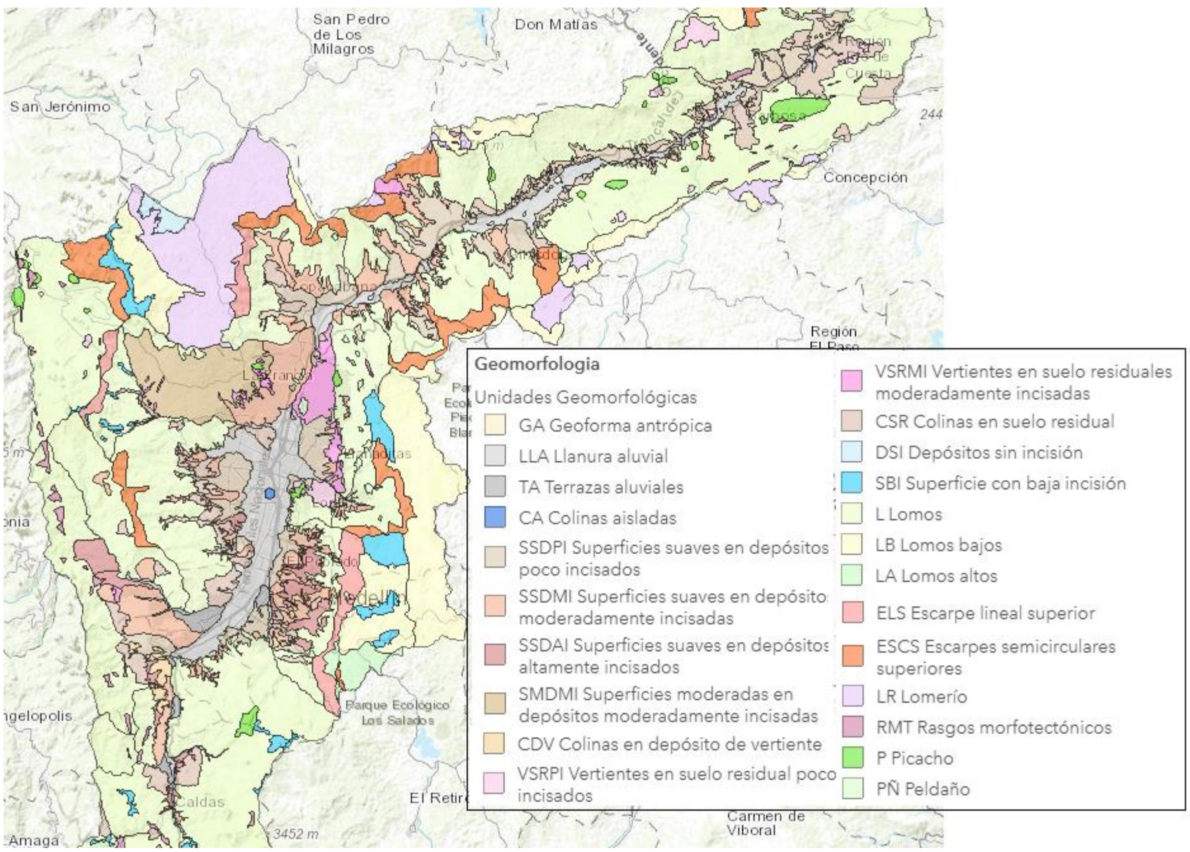


Figure 14. Geomorphology of the Aburrá Valley. Modified from: http://datosabiertos.metropol.gov.co/

According to the figure 14 and considering the regional geological context, the superficial materials in the valley corresponds to a sequence of alluvial deposits which are associated with torrential events of the Medellín river and several tributary streams. These torrential events transport the material in the areas where the streams follow a turbulent behavior, until reaching lower slopes where they are deposited.

Additionally, a considerable part of the surface materials in the valley correspond to slope deposits. Most of those deposits are mudflows conformed by a mix of fine materials (sand, clay and silt) and granular materials (gravel and rock blocs). The water content in the slopes of the valley is variable generating landslides that are moved down in the slope by gravity effect. The older layers of this old landslides have placed in the upper part of the slopes, and the newer layers are in the lower part of the slopes, close to the Medellín river.

Surface formations have variable thickness in the valley and usually in the first 10 m - 20 m there are soils composed by gravel, sand and silt with embedded rock balls (García, 2006).

3.3.1 Geotechnical parameters in Aburrá Valley

For analyzing the radial and tangential soil pressure induced by soil on pipes installed by the jacking pipe method in Aburrá Valley, it is necessary to know the variation of the geotechnical parameters in this area, those parameters are: the internal angle of friction of the soil (Φ’), the bulk unit weight of soil (γ), the water table level (H_1) and cohesion of soil (c´). In figures 15 to 18 are presented typical values of those parameters in the Aburrá Valley taken from: Universidad de Los Andes, 2016. *Armonización de la microzonificación sísmica de los municipios del Valle de Aburrá.*

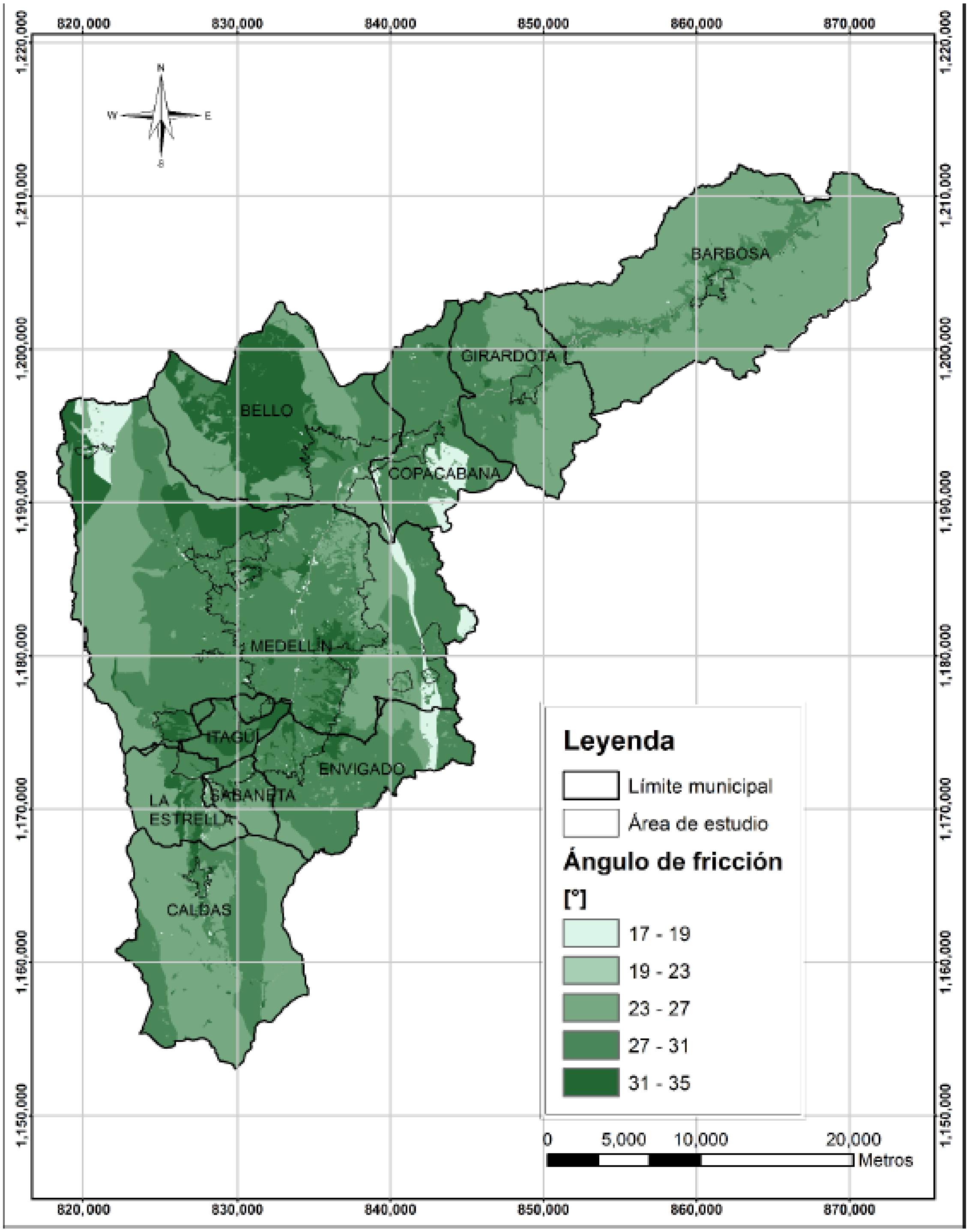


Figure 15. Angle of friction variation in Aburrá Valley. Modified from: "Armonización de la microzonificación sísmica de los municipios del Valle de Aburrá (Universidad de Los Andes, 2016)"

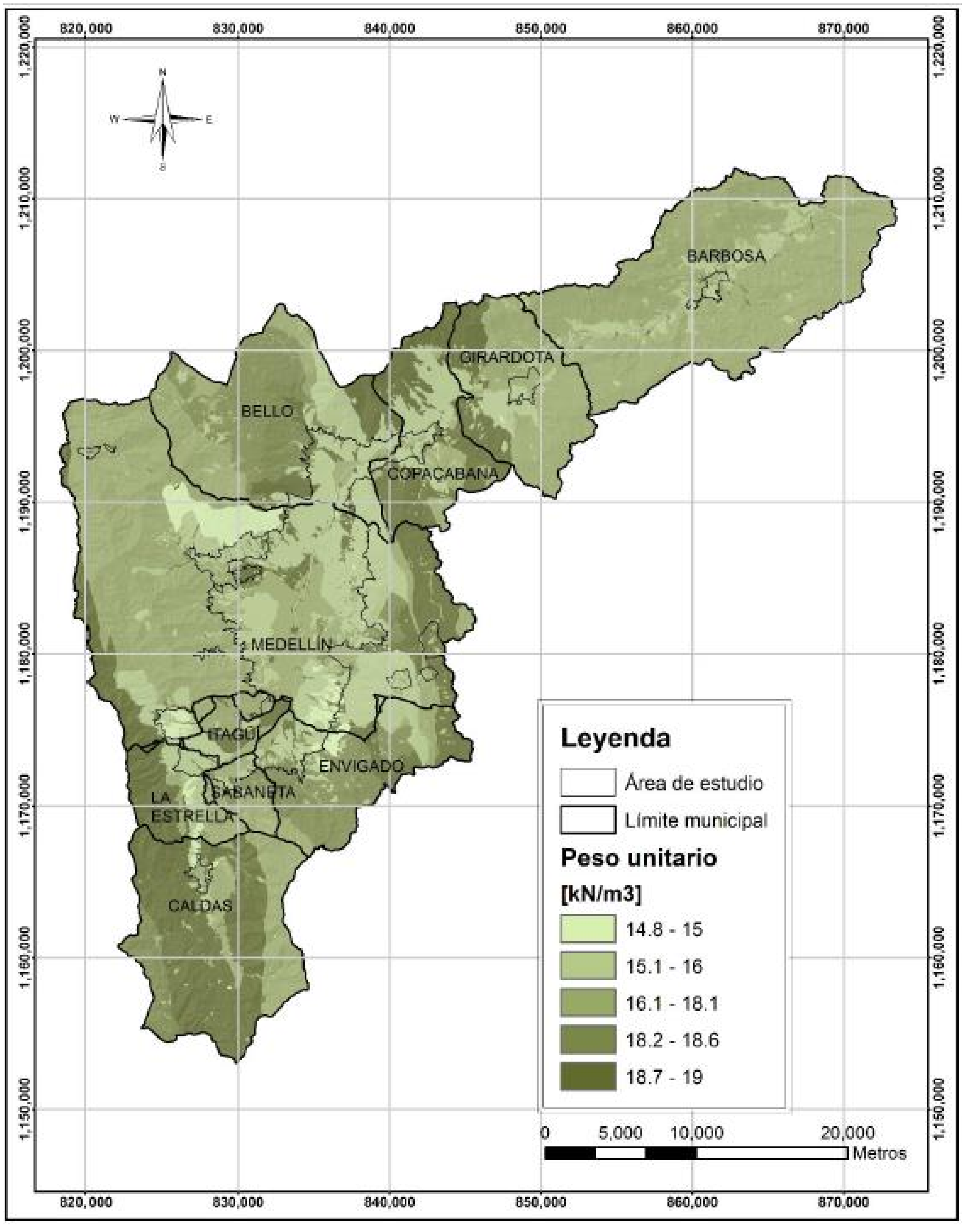


Figure 16. Bulk unit weight of soil in Aburrá Valley. Modified from: "Armonización de la microzonificación sísmica de los municipios del Valle de Aburrá, (Universidad de Los Andes, 2016)"

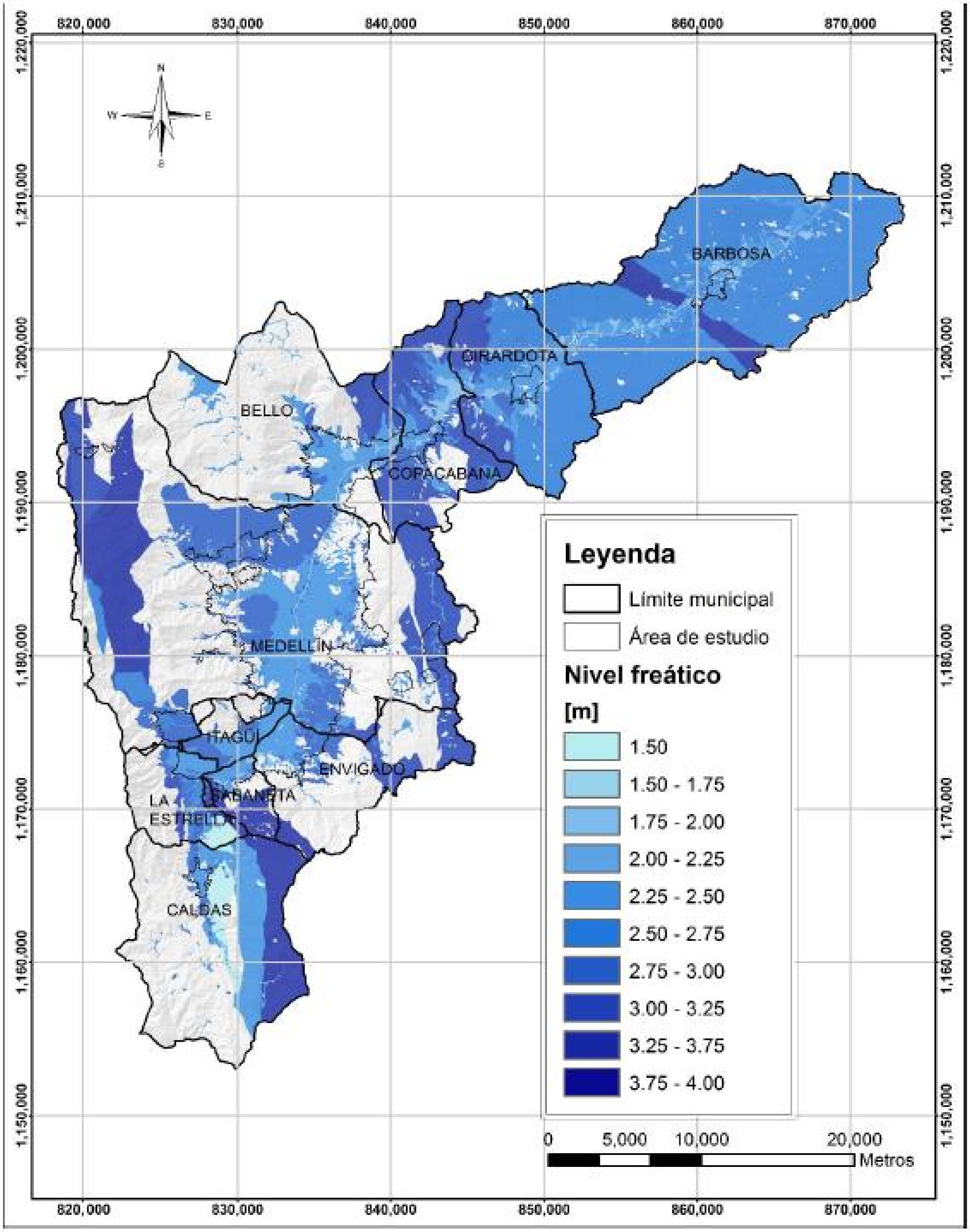


Figure 17. Water table variation in Aburrá Valley. Modified from: “Armonización de la microzonificación sísmica de los municipios del Valle de Aburrá, (Universidad de Los Andes, 2016)”

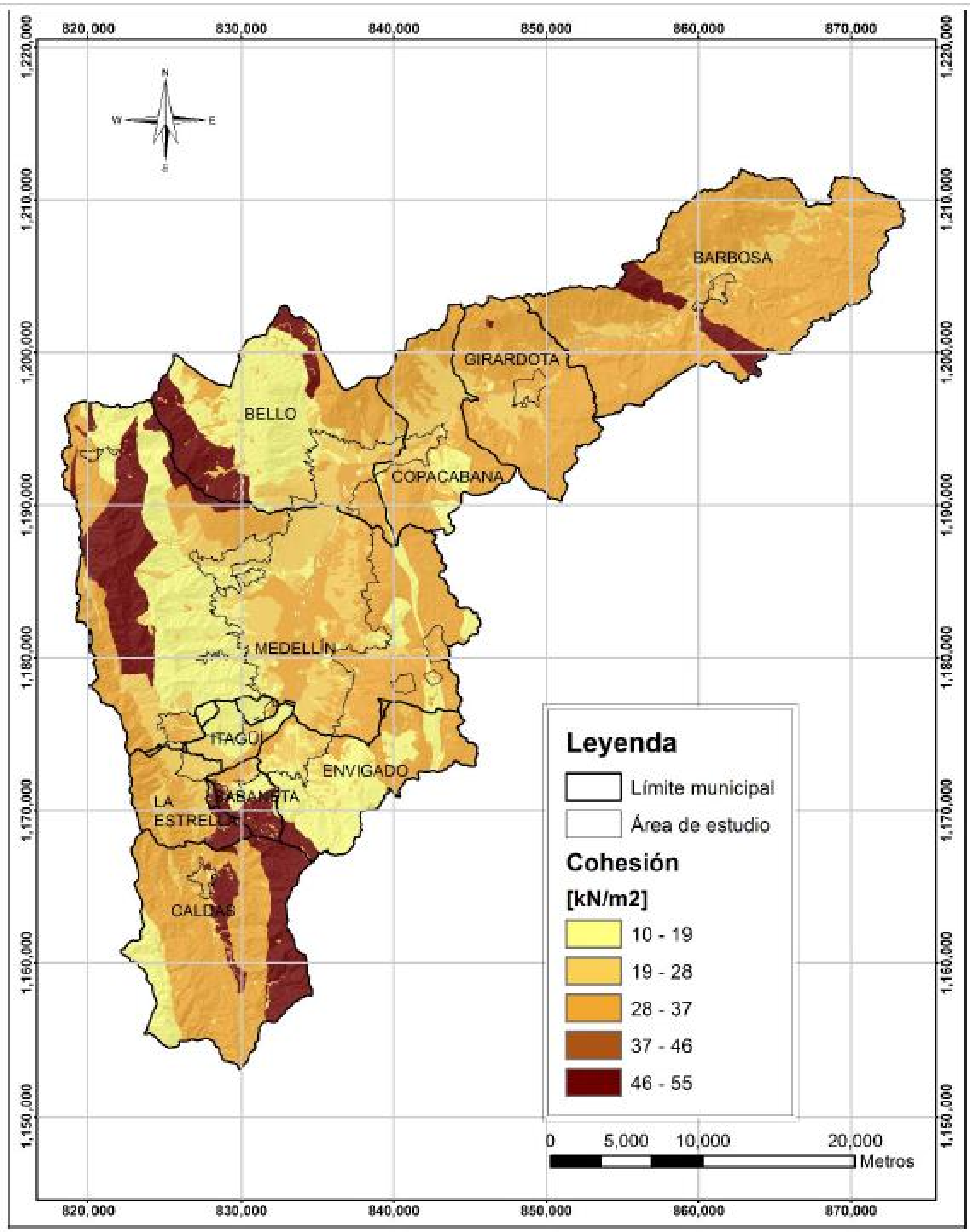


Figure 18. Cohesion of soil in Aburrá Valley. Modified from: "Armonización de la microzonificación sísmica de los municipios del Valle de Aburrá, (Universidad de Los Andes, 2016)"

3.4 Field case histories of pipe jacking in Medellin

Two projects have been constructed by the local public service company for the Aburrá Valley using pipe jacking method.

During construction of these projects, valuable information was collected such jacking forces measurement, face pressure, cutter wheel torque measurement and slurry flow rates. This information can help to understand the behavior of several parameters involved in the pipe jacking process.

3.4.1 Centro Parrilla project

3.4.1.1 Description of the project

In 2015 Empresas Públicas de Medellín (EPM) began the renovation and rehabilitation of potable water and sewer services in the downtown of Medellín city. This network is located in the urban center of Medellín, from San Juan Avenue to 62nd Street, and between 38th Street and Regional Avenue near to Medellín river (see Figure 19).

The project called "Centro Parrilla" had to solve different problems associated to vehicular traffic, numerous intersections of existing pipes and several social, economic and environmental problems.

In this project was implemented different trenchless methods including: pipe jacking, cured in place pipe (CIPP), horizontal directional drilling, pipe bursting and close fit sliplining.

The total length of pipes for rehabilitation and renovation was approximately 75 km (40 km for potable water and 35 km for sewerage networks). Trenchless was implemented in 67%

of potable water network length and 71% for sewerage network length, the rest of length was constructed by the open-cut traditional method (Arenas, 2017).

Figure 19. Google map view (GICA, 2011)of “Centro Parrilla” Project. Green: Metro lines. Blue: Santa Elena Creek. Dotted-red: Project borders. Modified from: (Gutierrez, 2015)

In “Centro Parrilla” project was constructed sewerage pipes with diameters from 600 mm to 1500 mm by pipe jacking method. The pipe used was reinforced concrete pipes with a polyethylene inner liner. This sewerage pipes transport the waste water that in the past was discharged directly to Santa Elena creek to the “Interceptor Norte” which is a big pipe that transport the fluid to the water treatment plant “PTAR Aguas Claras” constructed by Empresas Públicas de Medellín in Bello municipality.

Launch and receptions shafts were constructed along the alignment of the pipe with variable separation lengths between them. Shafts were approximately between 3 m and 21 m deep and their geometry was rectangular and circular.

The excavation lining used in shafts was shotcrete and concrete dowels. In some cases, steel and wood beams were used.

The microtunneling machine used in “Centro Parrilla” project was earth-pressure balance machine (EPBM) with a shield adapted for a mixed ground (soft and hard soils).

Figure 20. Launch shaft, concrete pipes and hydraulic jacks used in “Centro Parrilla” Project. Modified from: EPM photographic record

3.4.1.2 Geological and geotechnical conditions

A complete geotechnical characterization of the zone was made for designing phase of the project by the design contractor (INGETEC – Ingenieros Consultores). As can be seen in Figure 21, there are four types of geological units presents in the zone: Qal, QFIII, KgSD and TRmPP.

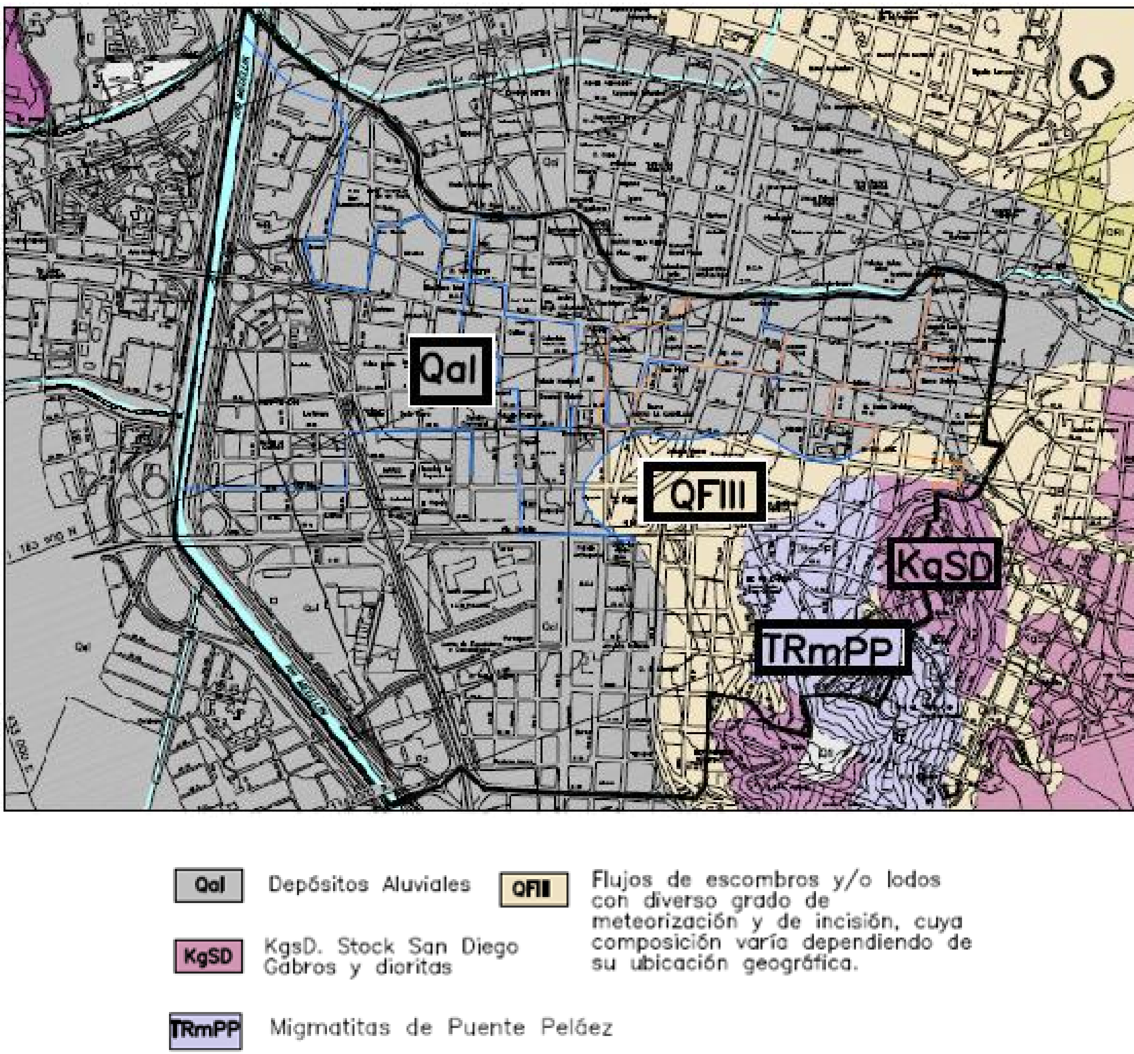


Figure 21. Geology of “Centro Parrilla” Project (Ingetec SA - Ingenieros Consultores, 2013).

Qal correspond to alluvial deposits which are present in nearly flat areas along the course of a Medellín River and their streams which are naturally subject to flooding. In general, this material is composed by a mix of gravel, sand and silt with embedded rounded rocks of different sizes. As can be seen in Figure 21 this is the predominant formation in the project area.

QFII correspond to mudflows that was deposited in the lower part of the slopes by old landslides that are moved down by gravity effect. In general, this material is characterized by presenting rock blocks of different sizes on a sandy-silt matrix.

KgSD is an igneous formation which present a strong physical weathering profile near the surface. The residual material product of the rock disaggregation corresponds to silty-sand soils.

TrmPP correspond to a migmatite formation present a strong physical weathering. The residual material product of the rock disaggregation has different characteristics (silty-sands and clay-sandy soils).

Figure 22 shows a division of the project area into 7 zones: 1, 2, 3a, 3b, 4, 5a, 5b. This division was made based in 36 exploratory perforations with depths between 1.80 m – 20 m. With the information collected from the perforations made, the ground resistance parameters were obtained for each of the 7 zones.

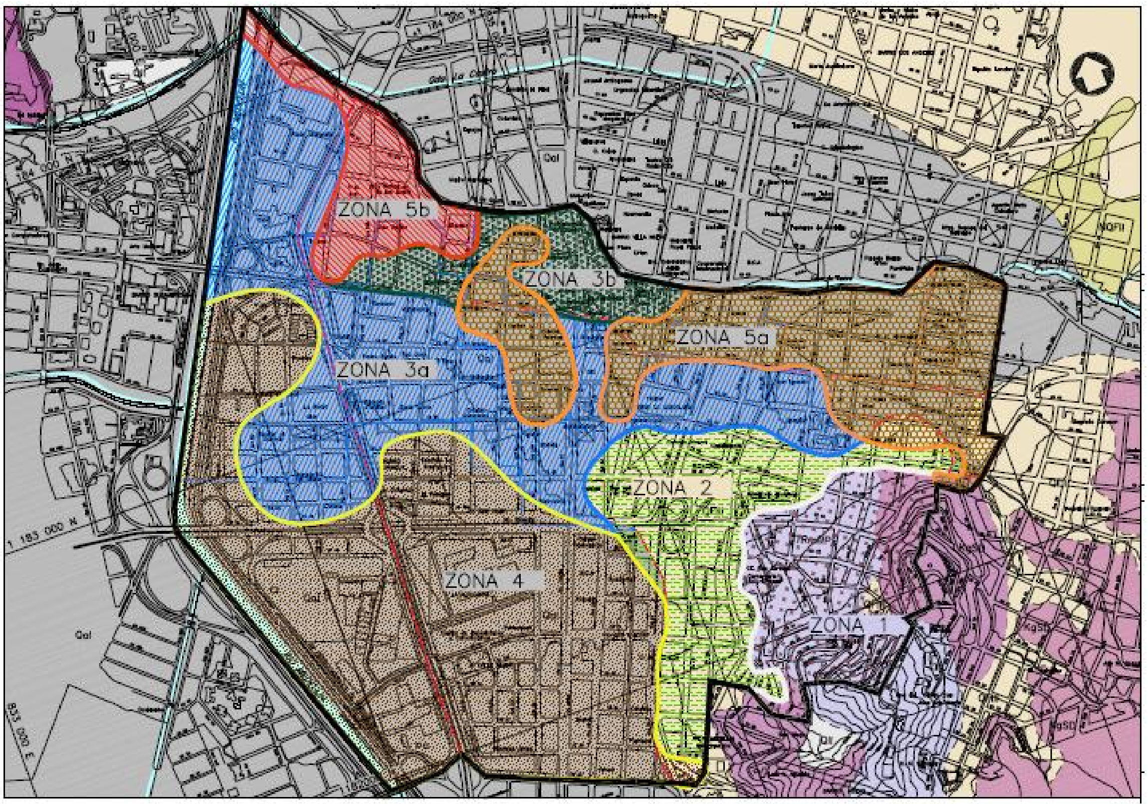


Figure 22. Geotechnical zones of “Centro Parrilla” Project (Ingetec SA - Ingenieros Consultores, 2013)

In Table 5 is presented the geotechnical parameters of the project for every zone.

Table 5. Geotechnical parameters of “Centro Parrilla” project (Ingetec SA - Ingenieros Consultores, 2013)

PARÁMEROS DE RESISTENCIA								
Zona geotécnica	Descripción	Profundidad [m]	Drenados			No drenados	Módulo de elasticidad	Nivel freático
			γ [kN/m3]	c´ [kPa]	Φ´ [°]	Su [kPa]	E [MPa]	[m]
1	Suelos de meteorización de TrmPP y KgsD	0.00 - 1.50	18.30	14	27	91	19	1.80
	Suelos de meteorización de TrmPP y KgsD	1.50 - 3.50	18.10	13	34	127	27	
	Suelos de meteorización de TrmPP y KgsD	3.50 - 6.00	18.20	15	32	114	29	
2	Flujos de escombros y/o lodos QFIII	0.00 - 1.50	18.8	15	31	75	19	1.80
	Flujos de escombros y/o lodos QFIII	1.50 - 4.00	17.6	28	26	127	23	
	Flujos de escombros y/o lodos QFIII	4.00 - 6.00	17.6	13	31	143	25	
3a y 3b	Depósitos aluviales Qal	0.00 - 1.40	19.2	20	31	66	19	2.50 (3a) 4.00 (3b)
	Depósitos aluviales Qal	1.40 - 4.50	18.3	18	27	99	25	
	Depósitos aluviales Qal	4.50 - 6.30	19.1	6	42	137	29	
4	Depósitos aluviales Qal	0.00 - 1.50	19.8	27	36	71	25	1.80
	Depósitos aluviales Qal	1.50 - 2.85	19	11	32	89	32	
	Depósitos aluviales Qal	2.85 - 6.00	20	18	25	121	50	
5a	Depósitos aluviales Qal	0.00 - 1.40	19.6	20	31	95	26	4.00
	Depósitos aluviales Qal	1.40 - 4.70	18.7	14	33	137	40	
	Depósitos aluviales Qal	4.70 - 6.00	17.3	14	23	115	36	
5b	Depósitos aluviales Qal	0.00 - 1.40	21.1	15	34	-	35	4.00
	Depósitos aluviales Qal	1.40 - 6.30	21.2	15	42	-	65	

3.4.1.3 Construction stage

In Figure 23 is presented a plan view of a typical segment of pipe jacking installation in “Centro Parrilla” project. On this segment for sewer pipe renovation, there are 5 shafts (red dots), those launching and receiving shafts allowed to install 280 m of a new concrete sewer pipe of 600 mm and 700 mm diameter.

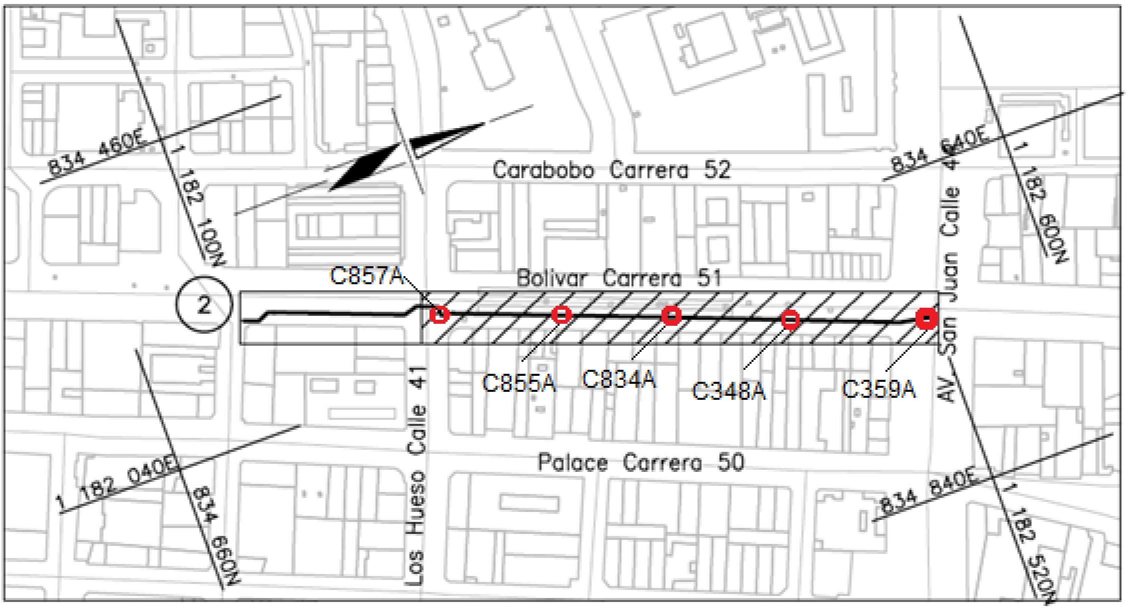


Figure 23. Typical segment of pipe jacking in "Centro Parrilla" Project (EPM-51MED23-07RE-0337, 2013)

In order to analyze the pipe jacking installation process, in figure 24 is shown a longitudinal section between shafts C348A and C359A.

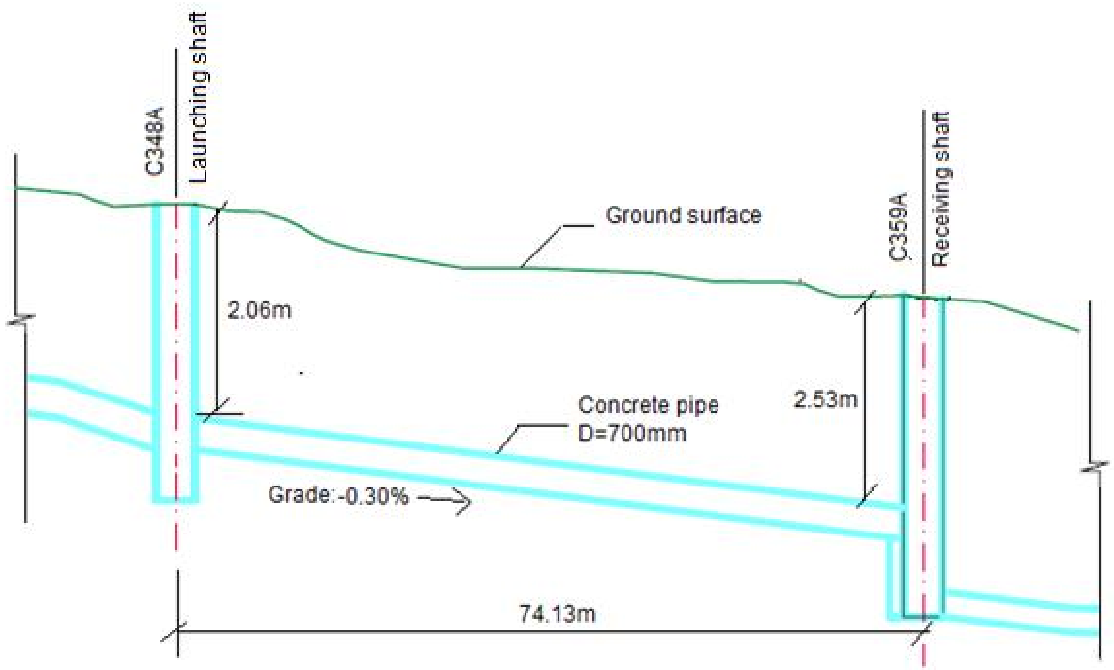


Figure 24. Segment between C348A and C359A shafts in “Centro Parrilla” Project. (EPM-51MED23-07RE-0337, 2013)

In Table 6 is presented the basic information about the segment between C348A and C359A shafts.

Table 6. Information about the segment between C348A and C359A shafts.

(Ingetec SA - Ingenieros Consultores, 2013)

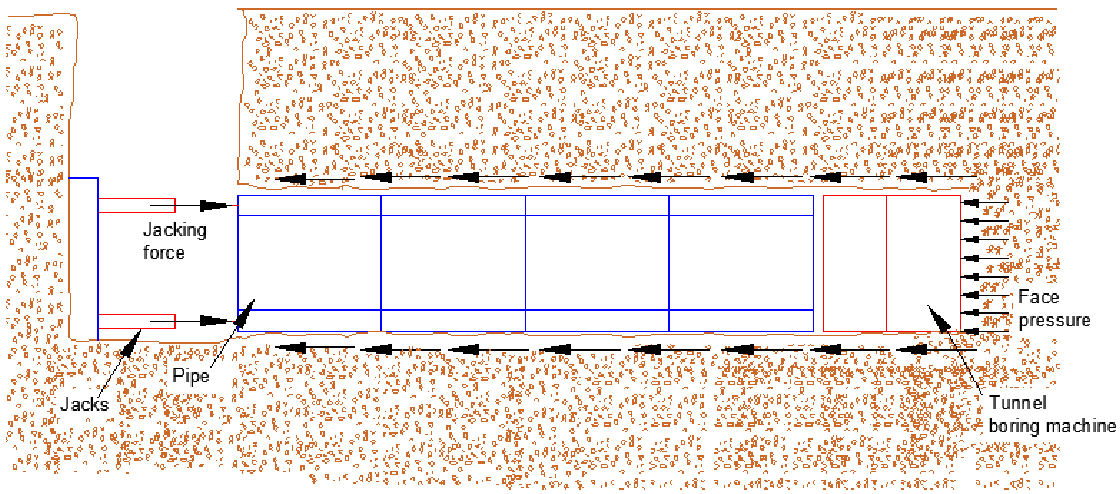
Parameter	**Value**
Pipe material	Reinforced concrete with a polyethylene inner liner
Pipe diameter	0.70 m
Length between shaft axes	74.13 m
Effective jacked length	56.08 m
Average depth of soil cover over the pipe crown	2.30 m

Parameter	**Value**
Geotechnical zone (according to figure 22)	Zona 4 - Depósitos aluviales (Qal)
Efective internal angle of friction of the soil (Φ’) (according to table 5)	32°
Bulk unit weight of the soil (γ) (according to table 5)	19 kN/m^3
Depth of the water table from surface (H_1) (according to table 5)	1.80 m

During construction stage, it was possible to collect information that is useful to understand the interaction between pipe and soil. Parameters such the jacking force needed to push the pipes between shafts, the advance rates, the cutter wheel torque and the face pressure were measured.

It is very important to say that during the jacking operation the microtunneling machine excavated a larger diameter hole than the external pipe diameter. In this case the overcut was about 3 and 5 cm. In this annular space it was injected bentonite through the lubrication ports located on the pipe walls (every two pipes).

In order to illustrate and analyze the information collected in the field, in Figure 25 is presented a scheme where can be seen a longitudinal view of pipe that is pushed by the main jacks. In the same figure is shown the “time effects” and “lubrication effects” during the pipe jacking process.



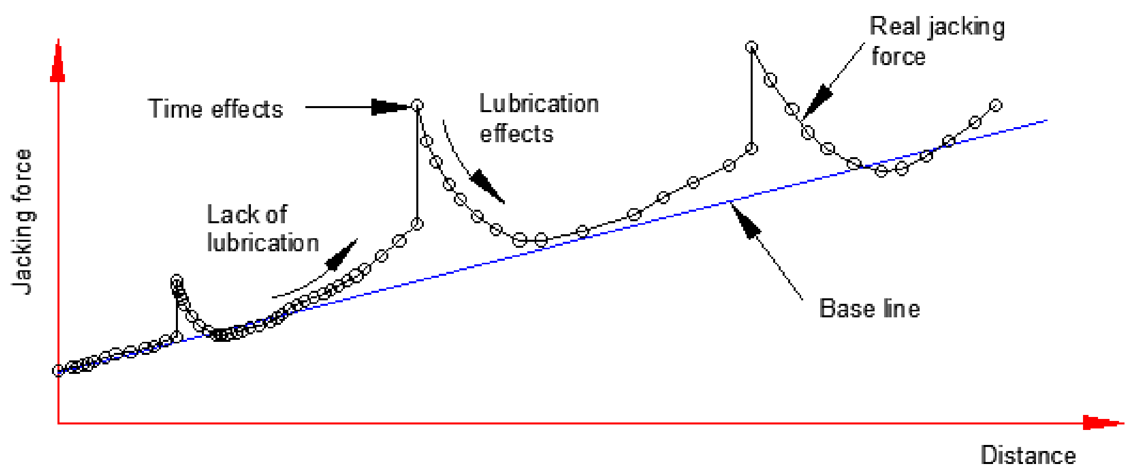


Figure 25. Behavior of the predicted and real jacking forces. Modified form (Milligan & Norris, Pipe-soil interaction during pipe jacking, 1999)

As can be seen in Figure 25, there is a “base line” that represents the frictional component of the jacking force. The frictional component increase while the distance is larger due to the increasing contact surface between pipe and soil and the radial stresses induced by the soil on the pipe.

The real jacking force behavior is affected by two principal effects. The first effect that increase rapidly the jacking force is the time effect. It is observed during periods when the pipeline is not moving. During construction it is common to stop the advance for any technical reason and after the stoppage the jacking force jumps up and the force presents values higher than the values of the base line (Milligan & Norris, 1999).

The second effect that affects the increasing or decreasing in jacking force is lubrication. When there is a lack of lubrication between the pipe and soil, the jacking force gradually increases due to the direct frictional interaction between pipe and soil. When using lubrication, the jacking force decreases and in some cases could generate a buoyant effect on the pipe.

This effects can be seen in Figure 26, where is presented the jacking forces measured between C348A and C359A shafts in Centro Parrilla project.

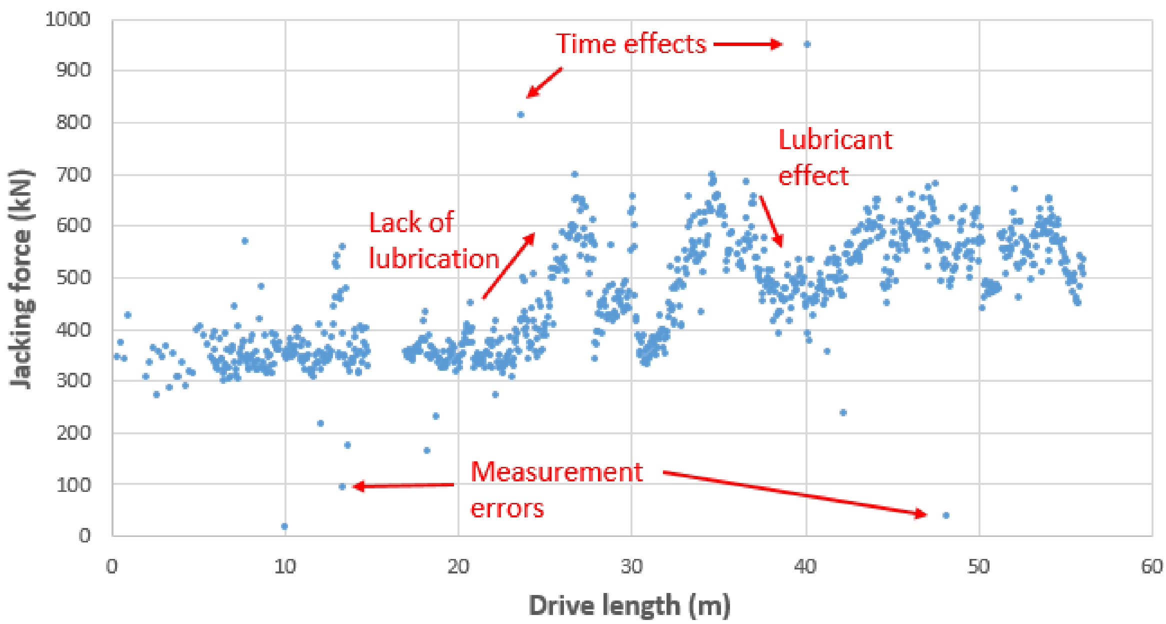


Figure 26. Jacking forces between C348A and C359A shafts in “Centro Parrilla” Project.

Modified from: (Ingenieria y Contratos S.A.S, 2013)

As can be seen in Figure 26, the time effect increase abruptly the force after a stop of the work, for example when the drive length of the tunnel was 24 m, the force in the main jacks registered was 380 kN, but after a stop of several hours the jacking force jumps up to 810 kN. After that, the lubrication using bentonite helps to reduce the force to values of 450 kN approximately.

The lubrication effect can be observed too. Note the bentonite effect in Figure 26, when no lubricant is used the interface friction coefficient between pipe and soil is reduced considerably respect to the non-lubricated interface friction coefficient. As a consequence, the jacking forces increase gradually when there is no lubricant in the gap between the pipe and soil.

The minimum values that can be seen in Figure 26 corresponds to measurement errors. This occurs because the measuring device consisting of a wheel that makes contact with the pipe loses contact with the pipe wall and wrong force values are recorded.

3.4.2 Interceptor Norte project

3.4.2.1 Description of the project

The “Interceptor Norte” is a project conducted by EPM (Empresas Públicas de Medellín) for the sanitation of Medellín river and their affluent creeks. The project consists in a big pipe that transport the waste water of the north side of Medellín city and Bello municipality to the treatment plant “PTAR Aguas Claras”.

In Figure 27 is presented the project location. The pipe alignment is parallel to Medellin river from Moravia neighborhood as the initial point, to the final point in Niquia neighborhood where is located the treatment plant.

Figure 27. Google map view of “Interceptor Norte” Project. Modified from: (GICA, 2011)

Launch and receptions shafts were constructed along the pipe alignment with variable separation lengths between them. Shafts were constructed approximately between 2 m and 20 m of depth.

Table 7. General information about the “interceptor Norte” project (Consorcio CISE, 2011)

Shaft	**Pipe diameter (m)**	**Pipe Length (m)**	**Installation method**
C1 to C14	2.20	4747	Pipe jacking
C14-C22	2.40	2006	Pipe jacking
C22-C23	2.40	511	Open cut trench
C23-C26E	2.20	397	Pipe jacking
Cross points between shafts: C27-C 5 C28-C8 C30-C10A C31-C15	1.20	579	Pipe jacking

As can be seen in Figure 28, along the alignment the pipe had to cross the Metro line, the Medellín river from the east side to the west side and existing infrastructure.

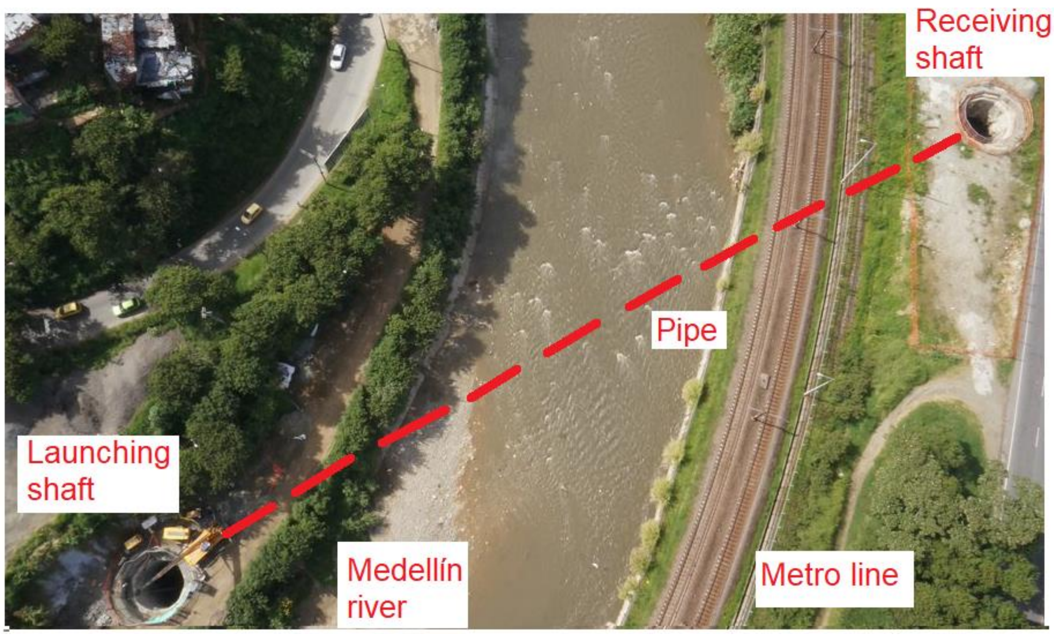


Figure 28. River and Metro line crossing "Interceptor Norte" Modified from: EPM photographic record

The excavation for pipe jacking installation was carried out by a slurry tunnel boring machine with pressure balance in the front of the excavation. The face pressure was balanced by pumping slurry (bentonite) to the front of the tunnel-boring machine to contain the face pressure. The machine shield used was adapted for cutting soft grounds and rock.

In the "Interceptor Norte" project was used sewerage pipes with diameters from 1200 mm to 2400 mm by pipe the pipe jacking method

Figure 29. Typical shaft, TBM, hydraulic jacks and concrete pipe used in the “Interceptor Norte” project

Modified from: EPM photographic record

3.4.2.2 Geotechnical conditions along the alignment

In order to know the geological and geotechnical conditions along the “Interceptor” alignment a complete exploration campaign was executed by the contractor. The exploration campaign consisted in a several number of boreholes located in each shaft and different points along the pipe alignment. In Figure 30 is shown a stratigraphic profile along the interceptor axis (red line) and perforations that were made which are denoted with the letter “P”.

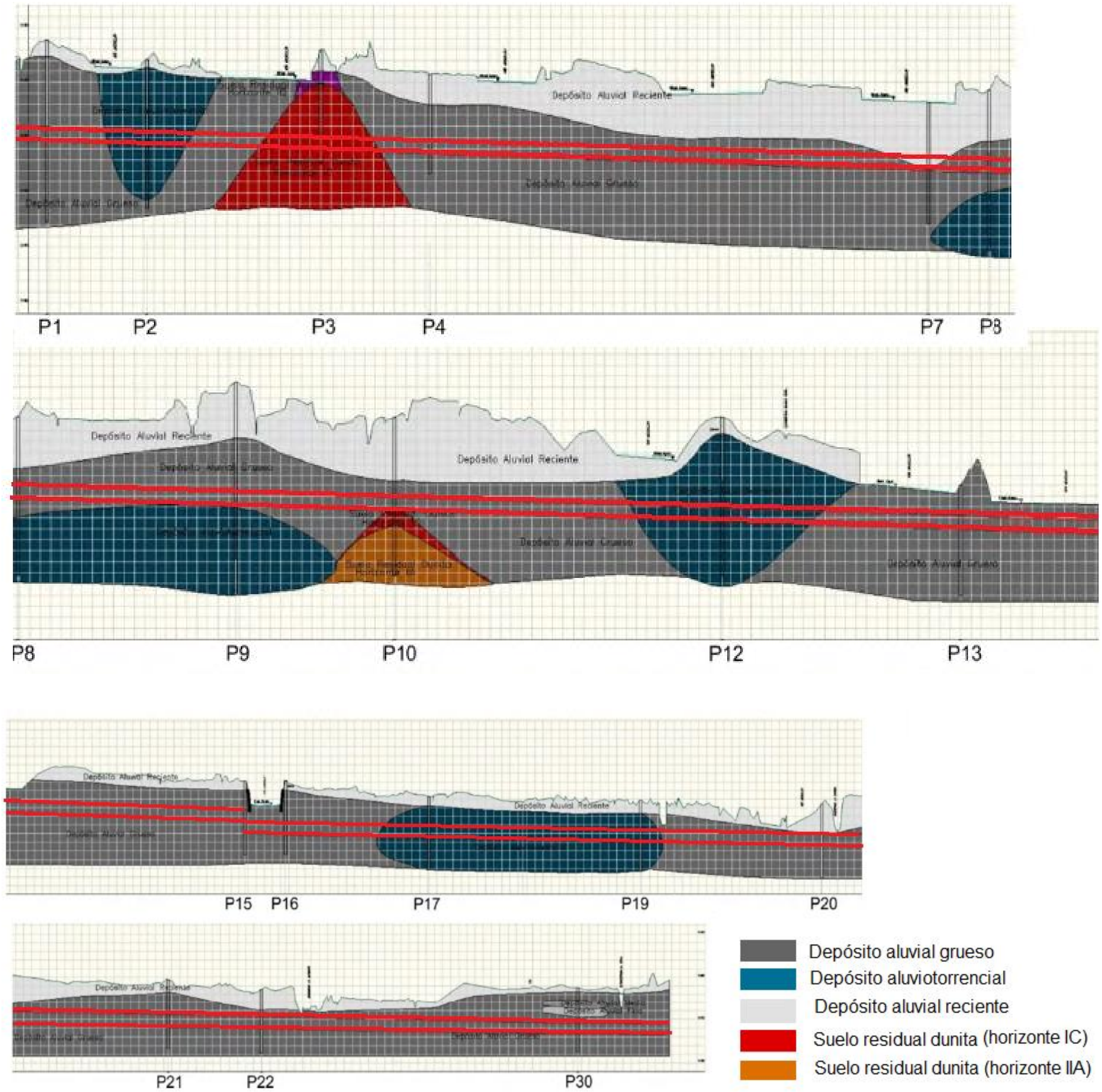


Figure 30. Geotechnical profile along the “Interceptor Norte” project (GICA, 2011)

According to the final inform for the geological and geotechnical characterization of Interceptor Norte (GICA, 2011), the following geotechnical characterization was made:

Thick alluvial deposit (Depósito aluvial grueso - Qal): This material is a mix of gravel, sand and silt. This is a well-graded material in which the smaller soil particles fill the space between the large particles. Also the material has rounded rock particles larger than 25 mm (1 in). As can be seen in Figure 30 this is the predominant material along the pipe alignment.

Alluvial-torrential deposit (Depósito aluviotorrencial – QAT): These are alluvial deposits which are associated with old torrential events of the Medellín river. These torrential events transported the material in the areas where the streams follow a turbulent behavior, until reaching lower slopes where they were deposited. The material is composed by angular fragments of dunite and gabbro rocks embedded in a clayey-silt matrix.

Recent alluvial deposit (Depósito aluvial reciente - Qal): Correspond to silts and clays with variable plasticity (CL, ML, CH, MH).

Dunite residual soil (Suelo residual dunita -KuM): Correspond to silty and sandy soils with high plasticity.

In Table 8 is presented the average geotechnical parameters for the predominant materials along the interceptor alignment.

Table 8. Average geotechnical parameters of "Interceptor Norte" project (GICA, 2011)

Depósito aluvial grueso		Depósito aluvial medio		Depósito aluviotorrencial		Depósito residual de dunita	
Parámetro	**Valor**	**Parámetro**	**Valor**	**Parámetro**	**Valor**	**Parámetro**	**Valor**
γ(kN/m³)	19	γ(kN/m³)	16	γ(kN/m³)	20	γ(kN/m³)	18
C(Kpa)	*	C(Kpa)	*	C(Kpa)	*	C(Kpa)	23
ϕ (°)	42	ϕ (°)	35	ϕ (°)	38	ϕ (°)	35
k_s (kN/m³)	66593	k_s (kN/m³)	21477	k_s (kN/m³)	83959	k_s (kN/m³)	54779
G_o(kPa)	1847462	G_o(kPa)	179470	G_o(kPa)	2693072	G_o(kPa)	1218937
G_{20} (kPa)	369492	G_{20} (kPa)	93324	G_{20} (kPa)	538614	G_{20} (kPa)	243787
E_o(kPa)	4618655	E_o(kPa)	466621	E_o(kPa)	6894265	E_o(kPa)	3169235
E_{20}(kPa)	923731	E_{20}(kPa)	93324	E_{20}(kPa)	1378853	E_{20}(kPa)	633847
ν	0.25	ν	0.30	ν	0.28	ν	0.3
Ko	0.33	Ko	0.50	Ko	0.39	Ko	0.43
Ka	0.20	Ka	0.27	Ka	0.24	Ka	0.27
Kp	5.00	Kp	3.67	Kp	4.14	Kp	3.67

3.4.2.3 Construction stage

In Figure 31 is presented a profile view of a typical pipe jacking installation on "Interceptor Norte" project. On this segment, C14 and C16 are the launching and receiving shafts and there is an intermediate inspection shaft C15.

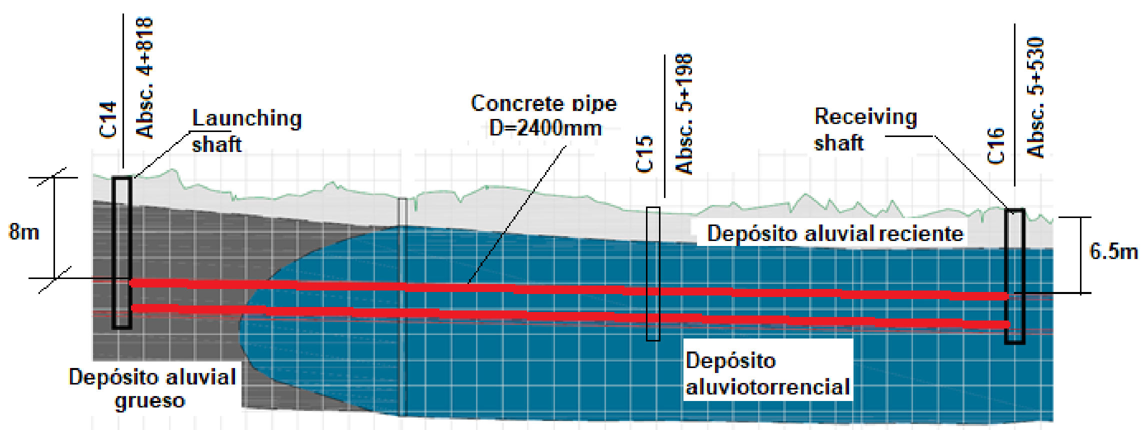


Figure 31. Profile view of the segment between C14 and C16 shafts in the "Interceptor Norte" project.

Modified from: (GICA, 2011)

In order to decrease the jacking loads, the use of bentonite for lubrication was implemented every two pipes. Additionally, it was used intermediate jacking stations inside the tunnel with a maximum spacing of 120 m or in zones where the frictional component of the jacking force was excessive and could cause a structural damage of the pipe.

The intermediate jacks propelled the TBM and the pipes between the machine and the intermediate station and the rest of the pipe length behind the intermediate station was propelled by the main jacks in the launching shaft or by another intermediate station.

The intermediate jacks were located inside a cylindrical steel section that was fabricated with the same external diameter of the pipe. The steel section connected two concrete pipes with special joints in order to prevent a local collapse of the ground when the jacks were activated.

In Figure 32 is presented one scheme and picture of the intermediate jacking stations.

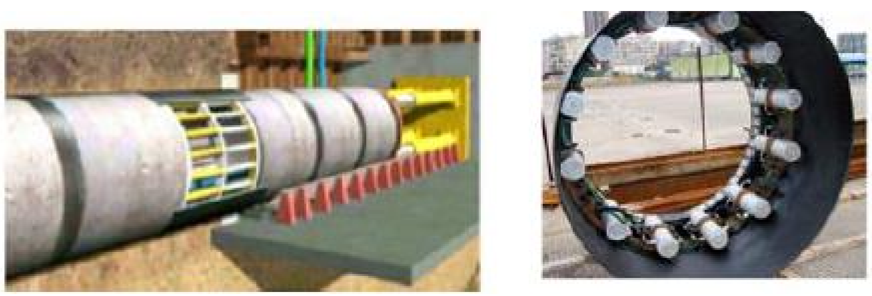


Figure 32. Intermediate jacking stations in the Interceptor Norte project.

Modified from: (GICA, 2011)

In Table 9 is presented the basic information about the segment between C14 and C16 shafts.

Table 9. Information about the segment between C14 and C16 shafts of the "Interceptor Norte" project (GICA, 2011)

Parameter	**Value**
Pipe material	Reinforced concrete
Pipe diameter	2.40 m
Length between shafts	712 m
Average depth of soil cover over the pipe crown	7 m
Soil materials	Alluvial deposit (200 m) Alluvial-torrential deposit (512 m)
Internal angle of friction of the soil	Alluvial deposit (Φ'=42°) Alluvial-torrential deposit (φ'=38°)
Bulk unit weight of the soil	Alluvial deposit (γ =19 kN/m^3) Alluvial-torrential deposit (γ =20 kN/m^3)
Depth of the water table from surface	Alluvial deposit (H_1 =5.3 m) Alluvial-torrential deposit (H_1=5.4 m)

In Figure 33 is shown the jacking forces measured between C14 and C16 shafts on the Interceptor Norte project.

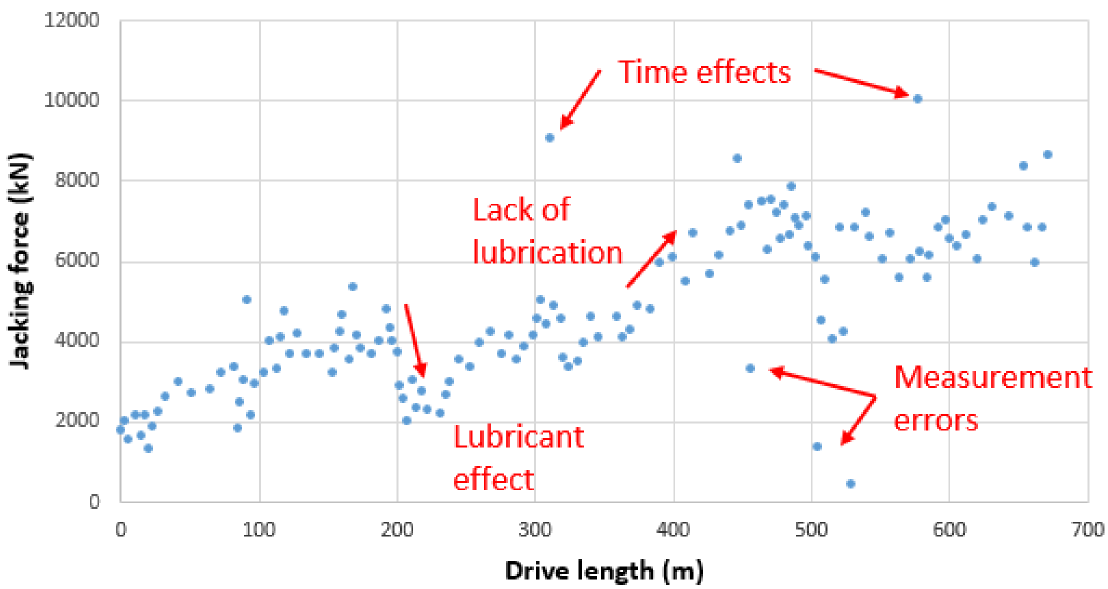


Figure 33. Jacking forces between C14 and C16 shafts in "Interceptor Norte" Project.

Modified from: (Ingetec SA, 2013)

As can be seen in Figure 33, the time effect increase abruptly the force after a stop (example: 9000 kN at 311 m), but after and before that pipe, the jacking force behavior shows values of 5000 kN approximately. That means that probably on that point of the interceptor alignment there occurred a stop during some period of time.

The lubrication effect can be observed too. When there was a lack of bentonite between the pipe and soil the jacking force gradually increases, and when there was a good lubrication the jacking force gradually decreased.

An additional effect that decreases the jacking forces in the main jacks happened because the use of intermediate jacking stations. In the "Interceptor Norte" project it was define that every 120 m should be located intermediate jacking stations. This is a very important aspect to take into account in Figure 33, because the force recorded corresponds to the main jack station which was reduced by the effect of the intermediate stations. This machines helped to push some segments between the TBM and the point where they were installed, as a consequence, the main jacks decreased the force needed to push the pipes.

The minimum values that can be seen in Figure 33 corresponds to measurement errors.

From information presented in chapter 3, it is possible to resume the following aspects:

Soils in the Aburrá Valley corresponds to alluvial and slope deposits, which in general are composed by gravel, sand and silt with embedded rock balls. This type of materials can be classified as unstable for a tunnel excavation and pipe jacking. That means that the excavation does not have the capability to be stable and the material tends to close around the pipe generating a radial stress, which increases the frictional resistance for pipe installation.

CHAPTER 4

4.1 Analysis of soil pressure on pipes installed by pipe jacking method in Aburrá Valley

Geotechnical parameters of soils have impacts in calculation of radial and tangential stresses on pipes installed by the pipe jacking method. The effect of these parameters and their influence on jacking forces are analyzed in this chapter using equations that were presented in section 2.1.5 and information of soils in Aburrá Valley presented in chapter 3.

4.2 Variation of radial stress around the pipe

4.2.1 Radial stress Vs. Bulk unit weight of soil

In order to evaluate the variation of radial stress induced by soil on pipe, consider a 1.00 m pipe (external diameter), installed 10.00 m depth from surface level. The bulk unit weight of soil (γ) vary from 15 kN/m^3 to 19 kN/m^3, according to figure 16 in the Aburrá Valley. The internal friction angle of soil was assumed in 30° and the water table 5.00 m from surface level.

Radial stress variation was evaluated according to equation (2.5)

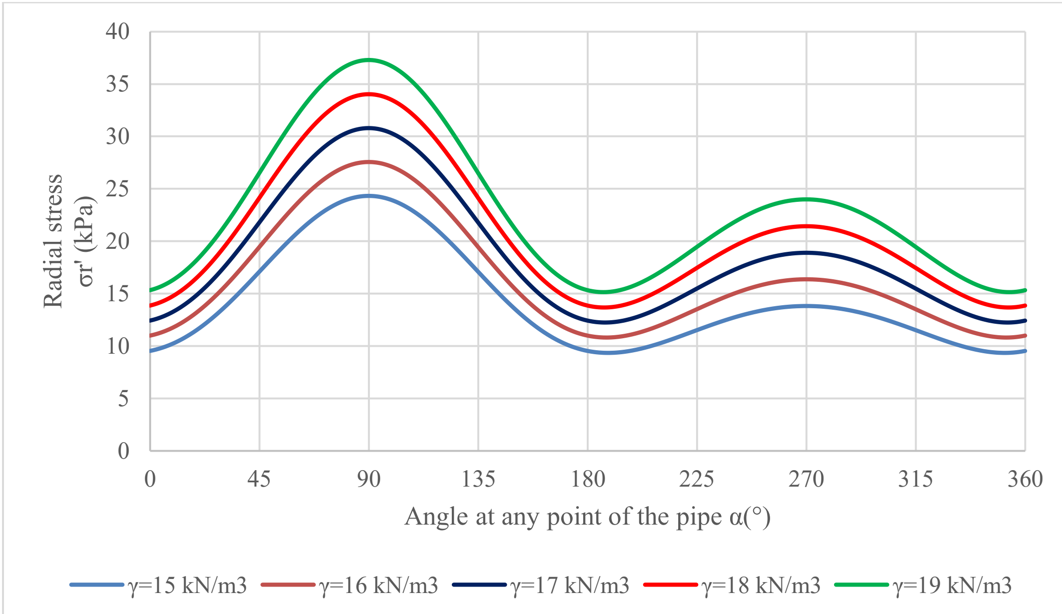


Figure 34. Typical radial stress variation around the pipe during pipe jacking (unstable soils).

Figure 34 allows to visualize the magnitude variation of radial stresses and their distribution around the external surface of the pipe. In general, the pattern and the shape of the radial stresses around the pipe keeps the distribution presented, regardless of diameter and geotechnical parameter values assumed.

The radial stress can be illustrated around the pipe as can be seen in Figure 35:

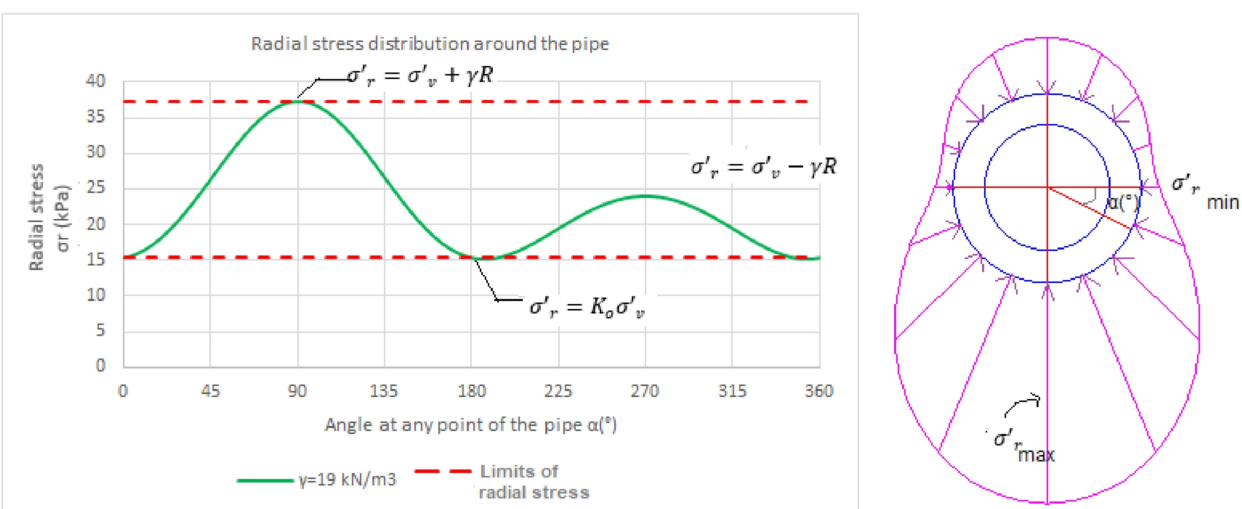


Figure 35. Radial stress distribution around the pipe.

4.2.2 Radial stress Vs. Internal friction angle of soil

In Figure 36 is presented the variation of radial stresses with the internal friction angle of the soil for the same conditions exposed in numeral 4.2.1. Values of the friction angle was evaluated for the range of this geotechnical parameter in the Aburrá Valley according to Figure 15.

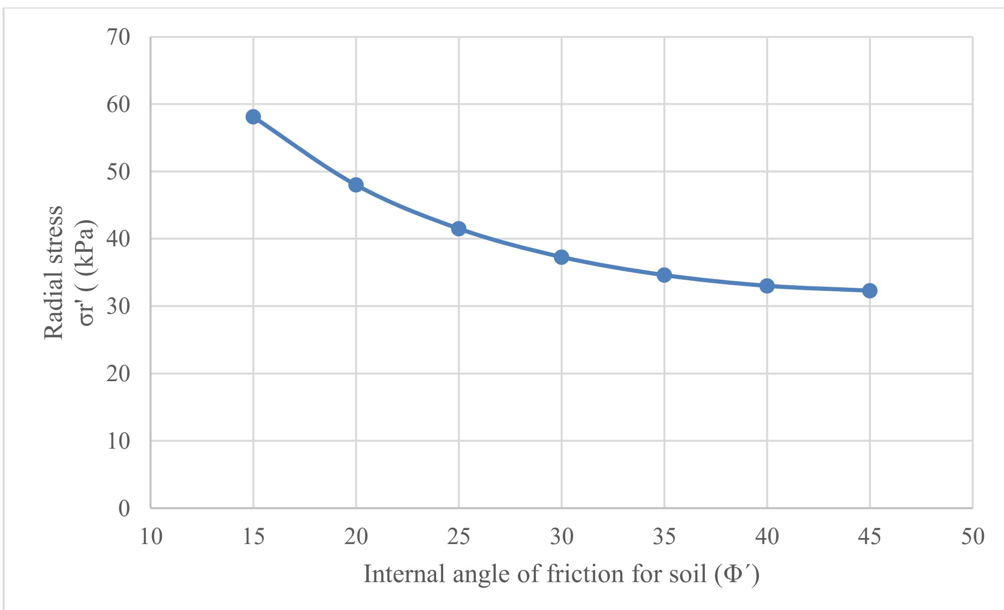


Figure 36. Radial stress Vs Internal angle of friction for soil

As can be seen in figure 36, soils with higher values of the friction angle will induce less radial stresses on pipes due to their higher shear resistance to the Terzaghi's arching effect (see 2.1.5).

Note: The radial stress was computed at $\alpha = 90°$.

4.3 Variation of tangential resistance in pipe jacking

Resistance to movement of pipe as it is pushed into the tunnel in a typical pipe jacking installation is because the tangential resistance or tangential pressure, which is related to the frictional interaction between pipe and soil.

Jacking forces that need to be achieve to overcome the tangential resistance is analyzed in this section, taking into account the variation of the geotechnical parameters in the Aburrá Valley.

4.3.1 Variation between jacking force and the bulk unit weight of soil

As a first step, it was calculated the vertical and horizontal effective stresses on pipe from equations (2.9) and (2.11), then it was computed the jacking force using equation (2.7).

Values of the bulk unit weight of soil (γ) were taken from Figure 16 (section 3.3.1) for the Aburrá Valley. Also, for the analysis purpose, it was assumed Φ'= 32°, D_e = 0.70 m, H = 2.30 m and H_1=1.8 m. These values were assumed equal to "Centro Parrilla" project data presented in section 3.4.1.3 (see table 6). A practical value of (δ/Φ') of 0.70 was assume for analysis (see section 2.1.6).

In Figure 37 is presented the jacking forces along a segment between shafts for typical values of (γ) in the Aburrá Valley soils.

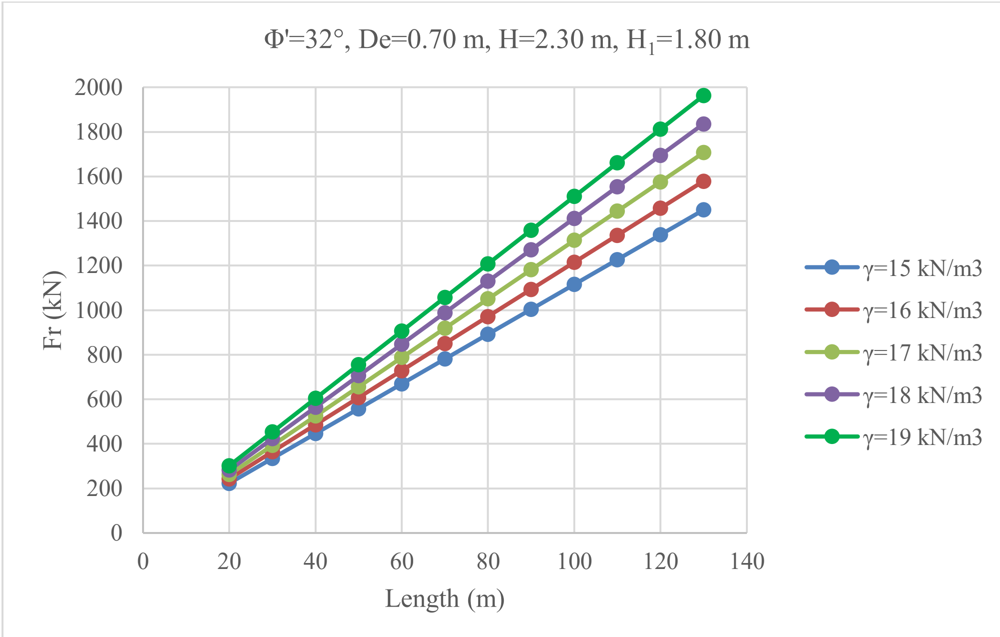


Figure 37. Jacking force Vs length between shafts, for typical values of the bulk unit weight of soil in Aburrá Valley

As can be seen in Figure 37, the jacking force has proportional relation with (γ), it means that if (γ) is increased, also the jacking forces will be increase. In addition, as the length between shafts increases, the effect of (γ) on the jacking forces is greater.

Figure 38 presents the variability of (γ) in a fixed length. For this exercise it can be seen that in 100 m length the jacking force vary from 1100 kN to 1511 kN, which represents a variation of 37%.

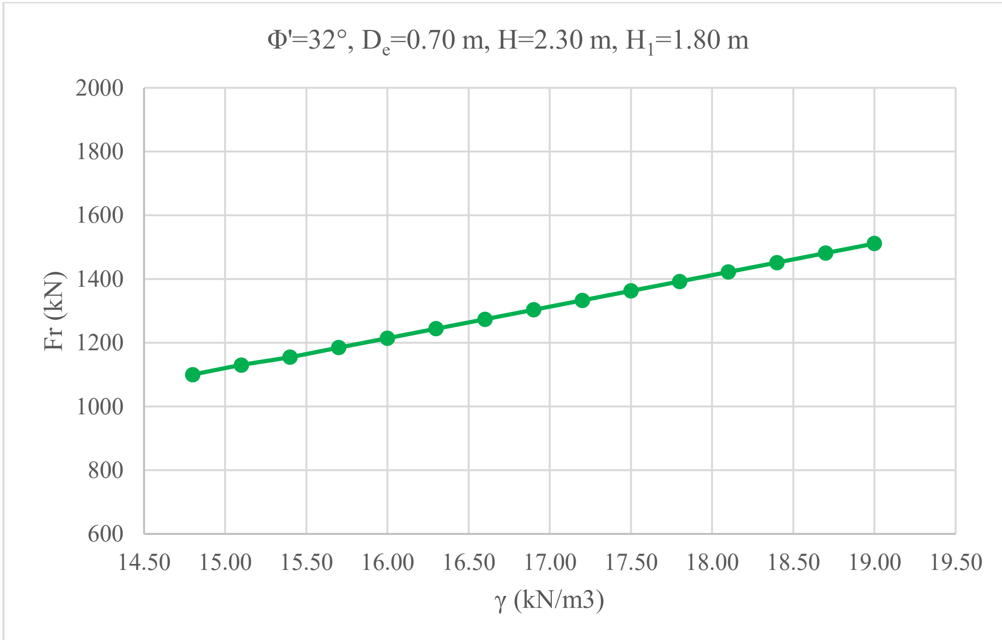


Figure 38. Jacking force (Fr) Vs Bulk unit weight of soil (γ)

The variability of (γ) along a segment between shafts has a very important implication when computing jacking forces. For this reason, is important to have field investigations data, in order to know the variability of this geotechnical parameter.

4.3.2 Variation between jacking force and installation depth

The same procedure described in section 4.3.1 was performed to observe the variation of jacking force with installation depth (H).

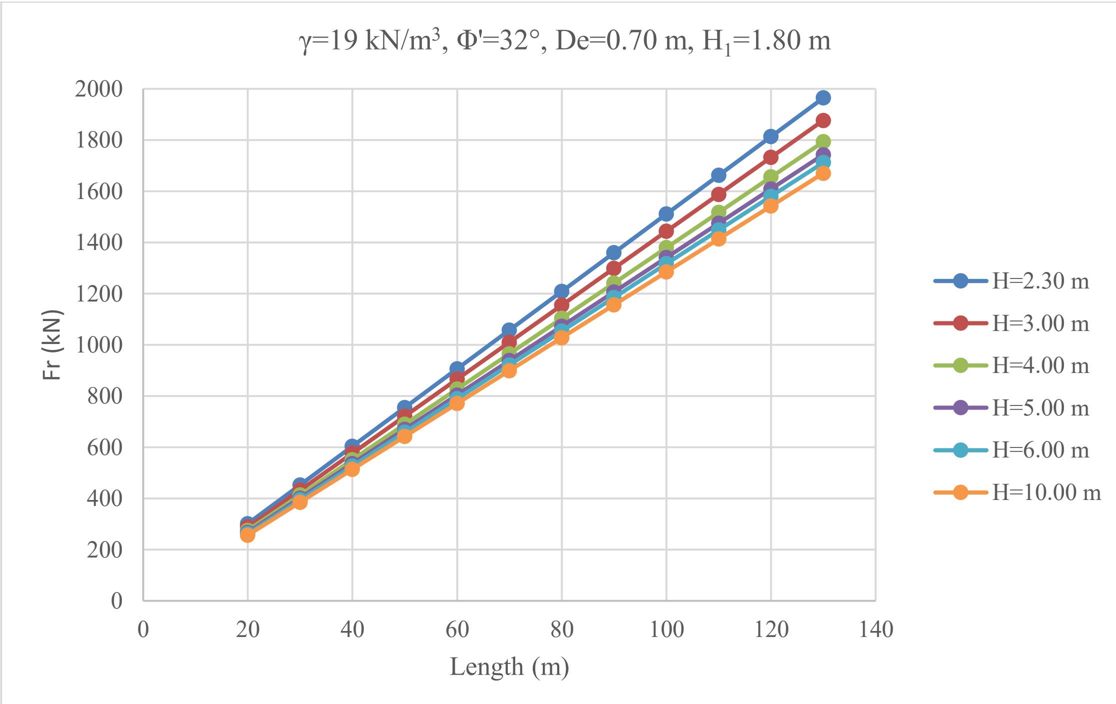


Figure 39. Jacking force variation with the installation depth of pipe

As can be seen in Figure 39, if geotechnical parameters remain relatively constant between shafts, there is no significant variation of the jacking forces. For instance, if the length is 100 m, the force is 1317 kN for H=6 m and 1285 kN for H= 10 m, this represents a difference of 2.4% in the jacking force. Additionally, comparing the force for H=3 m and H=6 m there is a difference of 8.7%. Thus, in a particular project the installation depth of pipe could be changed in a reasonable range without needing to require higher capacity of hydraulic jacks.

4.3.3 Variation between jacking force and the pipe diameter

The same procedure described in section 4.3.1 was performed to observe the variation of jacking force with pipe diameter (D_e).

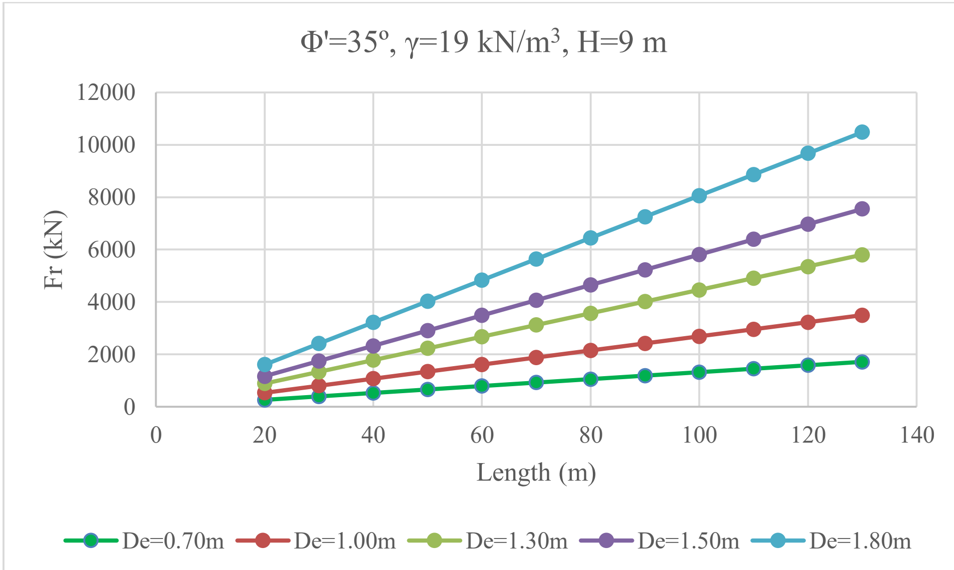


Figure 40. Jacking force variation for different pipe diameters

In this case, the variability of jacking force is significant, since the diameter increases the contact surface between soil and pipe.

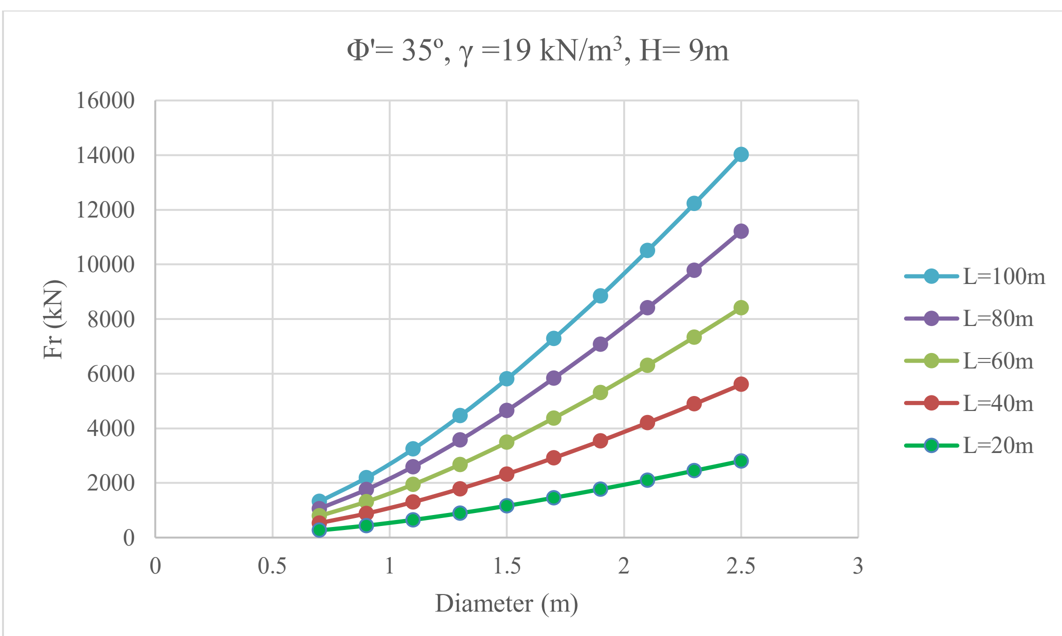


Figure 41. Jacking force Vs Pipe diameter

From Figure 41, it can be seen that the relationship between the pipe diameter and the jacking force is not a linear relation. The pipe diameter is the variable that most affects the calculation of jacking forces compared with the other parameters in equations (2.7), (2.9) and (2.11). However, the pipe diameter is normally determined by the hydraulic necessities in a particular project, and usually there is no variation of this parameter along a segment between shafts in pipe jacking operations.

4.3.4 Comparing computed jacking forces to field data

4.3.4.1 Comparison in "Centro Parrilla" project

Computed jacking forces were compared with field jacking forces obtained from "Centro Parrilla" project. Value of γ was taken as 19 kN/m^3 and Φ' as 32° (see Table 5). Pipe diameter D_e = 0.70 m, H = 2.30 m and H_1 = 1.80 m. These values were taken from "Centro Parrilla" project data presented in section 3.4.1.3. A practical value of (δ/Φ') of 0.70 was assume for analysis (see section 2.1.6).

The type of soil between C348A and C359A shafts in "Centro Parrilla" corresponds to alluvial deposits, which are composed by a mix of gravel, sand and silt with embedded rounded rocks of different sizes. This soil can be characterized as a cohesionless or unstable soil for a tunnel construction (see section 2.1.4).

The calculation was made using Auld´s and the German Standard DWA-A 161 according to equations presented in section 2.1.5.

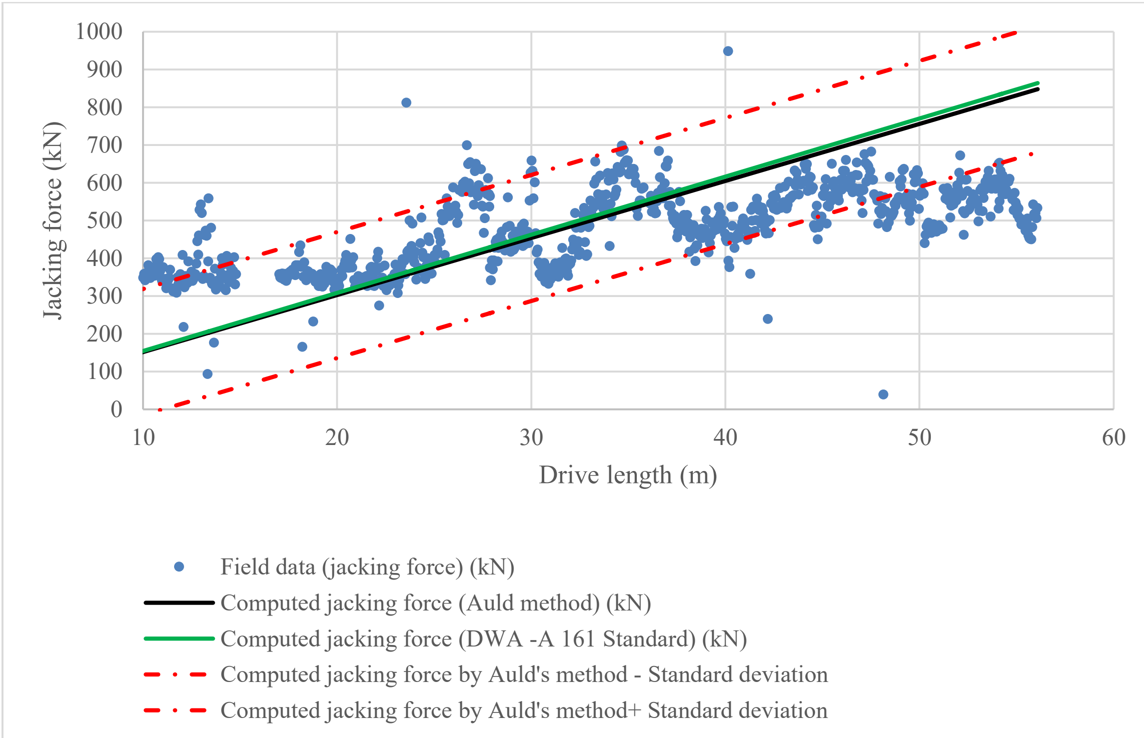


Figure 42. Field jacking force and computed jacking force in "Centro Parrilla" project

Figure 42 shows that the computed values have an incremental tendency with the drive length of the pipe. This is because the frictional component increases with distance due to the increasing contact surface between pipe and soil and the radial stresses induced by the soil on the pipe.

In addition, Figure 42 shows that computed values of jacking forces using equations from Auld's method and German standard DWA-A 161 are very similar. The difference between these methods was 2% approximately.

As it was mentioned in section 3.4.1.3 the highest values of the field jacking forces shown in Figure 41 are because the time effect. This effect occurs during construction periods when the pipeline is not moving. During construction it is common to stop the advance for any technical reason, or just because the work stops at nights and restart at mornings, and after the stoppage the jacking force jumps up and presents higher values than before the stop.

Computed values do not take into account lubrication and time effects that are two components that affect the real jacking forces, but this prediction is a good approximation when a designer have to evaluate the magnitude of jacking forces in early stages of a pipe jacking projects.

4.3.4.2 Comparison in the “Interceptor Norte” project

To compare computed jacking forces with the field data in the “Interceptor Norte” project, it is necessary to take into account the use of intermediate jacking stations during construction. In this project, it was used intermediate jacking stations inside the tunnel with a maximum spacing of 120 m in order to protect the pipes from a structural damage. In addition, a value of 18300 kN was defined in the project as a maximum allowable load for the concrete pipe (Consorcio CICE, 2011).

The force recorded corresponds to the main jack station which was reduced by the effect of the intermediate stations (see Figure 43).

The type of soil for the first 120 m between C14 and C16 shafts in the “Interceptor Norte” corresponds to thick alluvial deposits. This material is a mix of gravel, sand and silt, and it has rounded rock particles larger than 25 mm (see figure 31 and Table 8, section 3.4.2.2 and 3.4.2.3).

For calculations of jacking forces, it was considered two cases. In the first one it was assumed a cohesionless soil taking into account the geotechnical information of the project. The value of γ was taken as 19 kN/m^3, an equivalent internal angle of friction (Φ_{eq}') was taken as 42° (see Table 9). Pipe diameter D_e = 2.40 m, H = 8.00 m and H_1 = 5.30 m. These values were assumed equal to “Interceptor Norte” project data presented in section 3.4.2.3. A practical value of (δ/Φ') of 0.70 was assume for analysis (see section 2.1.6).

In the second case it was assumed that soil has a cohesion according to the information of geotechnical parameters in the Aburrá Valley presented in section 3.3.1. The value of Φ'= 27º and c’= 25 kPa was taken from figures 15 and 18.

In this case, the computed jacking force will not be analyzed by the DWA- A 161 German Standard, because there is not a big difference with the Auld’s method as evidenced in Figure 42.

In Figures 43 and 44 is presented the comparison for the segment between the main jack and the first intermediate station for the two cases analyzed (the first 120 m of the segment between the C14 and C16 shafts).

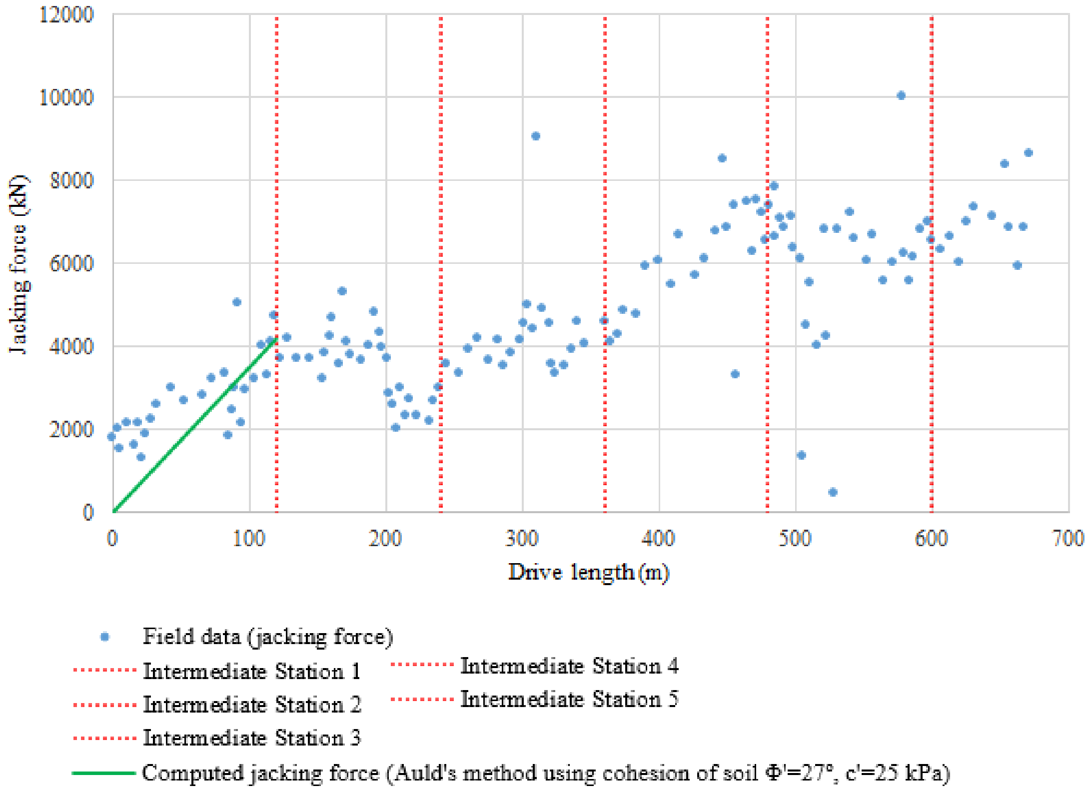


Figure 43. Field jacking force of the main jack (segment between C14 and C16 shafts). Interceptor Norte project (Ingetec SA, 2013)

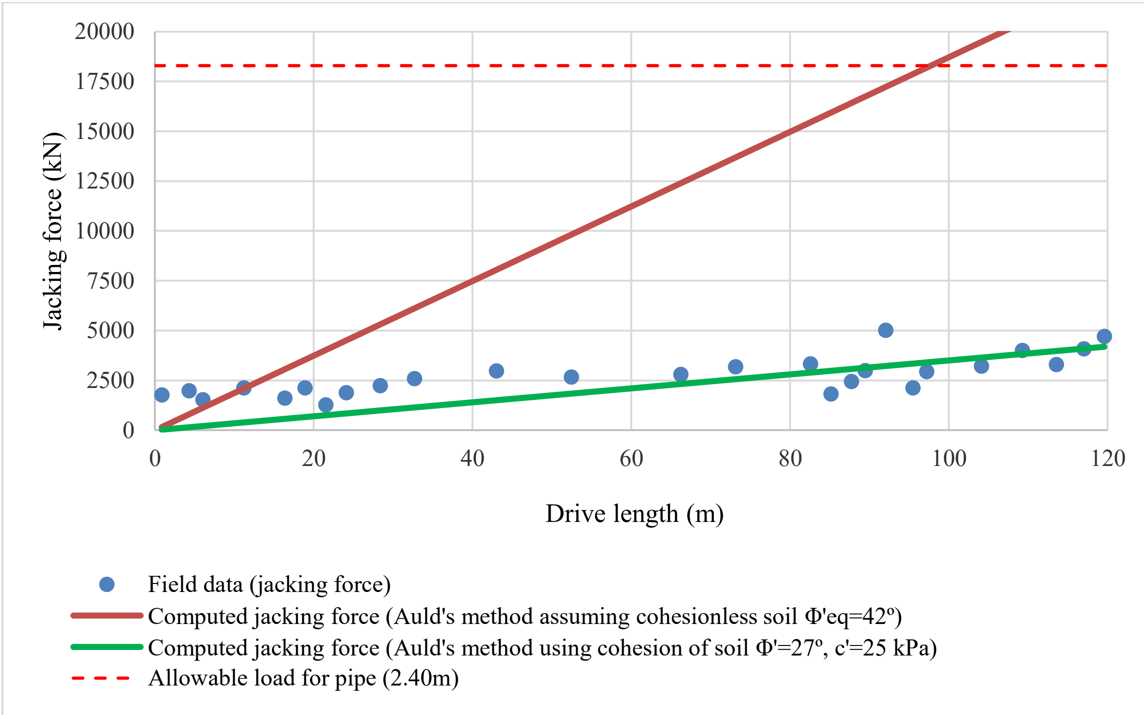


Figure 44. Field jacking force and computed jacking force in the "Interceptor Norte" project

As can be seen in Figure 44, the values calculated by the Auld's method considering a cohesionless soil are much higher than those observed for the field data. However, in the second case when cohesion of soil is considered, the values computed by Auld's method has a better approximation to the field data of jacking forces.

It is not possible to compare the total segment between shafts because the presence of intermediate jacking stations. This stations push the segments between the TBM and the point where they were installed, as a consequence, the main jacks decreased the force needed to push the pipes. Additionally, the use of bentonite for lubrication was implemented every two pipes along the entire length between shafts (Consorcio CISE, 2011).

From the analysis presented in chapter 4, it is possible to resume the following aspects:

Radial and tangential pressures on pipes installed by the pipe jacking method are affected by the variability of geotechnical parameters, the pipe diameter and the installation depth. The pipe diameter is the variable that most affects the radial and tangential pressures associated to jacking forces, and the pipe installation depth is the variable that less affects them as was presented in Figures 39 and 41. However, the pipe diameter is controlled by the hydraulic design and usually there is no variation of this parameter in a segment of pipe. In addition, among geotechnical parameters analyzed, it was found that the bulk unit weight of soil (γ) was the value that most affects the jacking forces followed by the internal angle of friction (Φ'). For this reason, it is very important to have field investigations, in order to analyze the variability of this parameters in soils where a pipe jacking project is projected.

Calculation of jacking forces using theoretical models were made and compared with field data recorded from "Centro Parrilla" and "Interceptor Norte" projects developed in the Aburrá Valley. In "Centro Parrilla" the computed values had a reasonable agreement with the field data. In the "Interceptor Norte" the computed values by Auld's method considering a cohesionless soil were much higher than those observed for the field data, but when cohesion of soil is considered, the values computed has a better approximation.

It is important to say that theoretical models tend to overestimate the jacking forces, however this overestimation provide a safety factor that can be optimized by the designer of a future pipe jacking project. In addition, theoretical methods do not take into account the use of lubrication and intermediate jacking stations and its influence on reduction of jacking forces.

In detailed design stage, designers can do numerical simulations using a finite element software, considering more information to accurate the value of the jacking forces (J. Yen, K. Shou, 2015).

Calculation of jacking forces in unstable soils by Auld´s and German standard DWA-A 161 models have shown a difference of 2%. The reason of this is because both methods are based on the Terzaghi arching theory. Difference between these methods are basically the magnitude of the width of the affected ground (B), which is 2% greater in the German Standard compared with the calculated by Auld's equations. In engineering practice, values of jacking forces computed by the German Standard are more conservatives than values of Auld´s method.

CHAPTER 5

Public services companies, planners and designers have to evaluate the feasibility of projects where there is no space for a conventional pipe installation using a typical construction of a trench, and the study of an alternative solution of a trenchless method like pipe jacking is required. It is common that designers and planners of water and sewer projects do not take into account or simply discard trenchless technologies such pipe jacking due to the limited information available and the lack of experience in this type of projects in the Aburrá Valley.

This chapter summarizes the results of chapter 4 and presents some application cases in the Aburrá Valley for a pipe jacking project using geotechnical information presented in chapter 3.

In addition, some examples will be presented with the objective to help designers and planners to compute approximated values of the jacking forces required in a project, in order to establish requirements of civil works and hydraulic jacks capacity. Quantification of these forces is essential in the different phases of pipe jacking projects

5.1 Methodology to compute the jacking force

A procedure to determine the magnitude of jacking forces in a pipe jacking project is presented:

1. Input data:
    a. Type of project (sewer or water supply)
    b. Pipe diameter and pipe material.

   c. Pipe alignment (plan and profile)

2. Geotechnical parameters

   a. Effective angle of friction of the soil (Φ')

   b. Bulk and submerged unit weight of the soil (γ, γ')

   c. Water table (H_1)

   d. Angle of friction between pipe and soil (δ)

3. Computed jacking forces

4. Variation of jacking forces due to variability of geotechnical parameters. In this step is studied the uncertainty of the results obtained due to the input data and geotechnical parameters used.

5. Recommendations for future steps of the project (detailed design).

In numerals 5.1.1 and 5.1.2 some examples of application will be presented in order to illustrate this methodology.

5.1.1 Practical exaple #1

A sewer project located in the Aburrá Valley is planned to be developed in Itagui municipality. A hydraulic design was made taking into account the growth of the population and the demand of water and sewer in the area. The information of the hydraulic design is as follows:

1. Input data:

Table 10. Information for example #1

Pipe material	Internal pipe diameter (Hydraulic)	Pipe thickness (t)	External pipe diameter (De)	Pipe strength
	(m)	(m)	(m)	(kN/m)
Reinforced concrete	1.20	0.136	1.47	175

Pipe alignment according to figures 45 and 46.

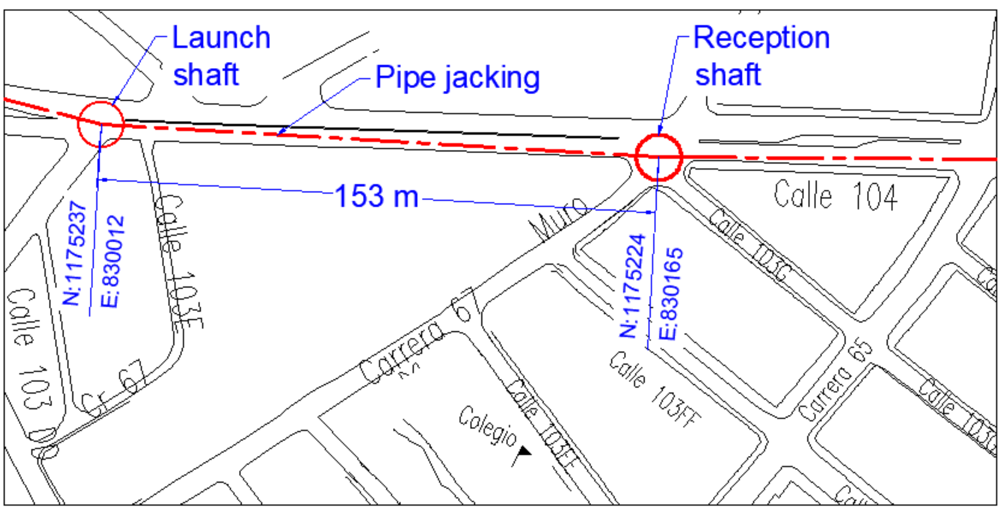


Figure 45. Plan view of the project for example #1

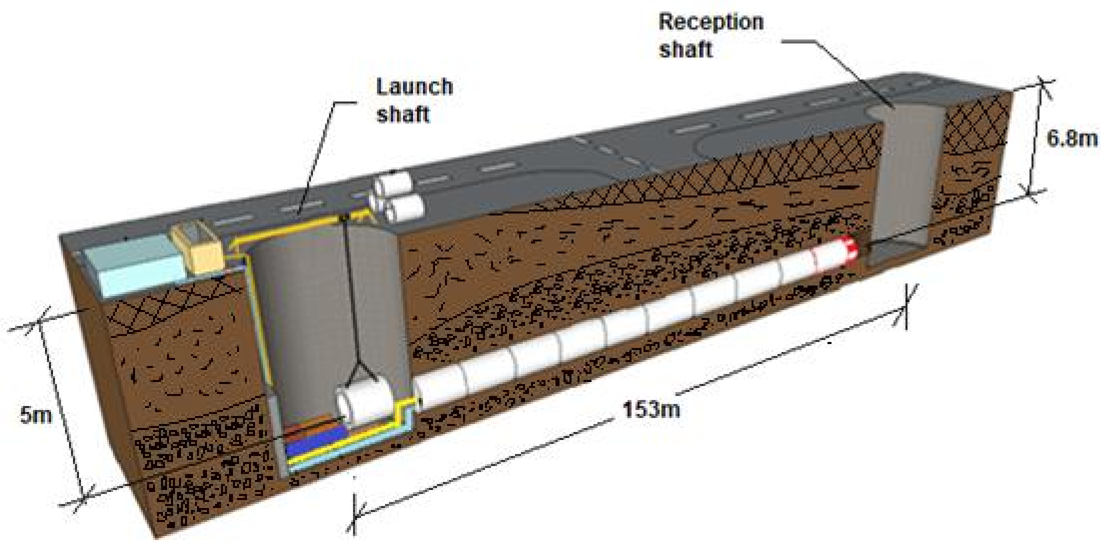


Figure 46. Schematic profile view of the project for example #1

As can be seen in Figures 44 and 45, the pipe alignment is crossing an urban area with high population density. The pipe depth is at least 5.00 m because there are several utilities pipes crossing along the street.

A typical open-trench installation method for this case would have interference with traffic and disruption of business or industry in the zone. Additionally, it would require a large excavation, for this reason, pipe jacking is a practical alternative to study.

2. Geotechnical parameters

Due to the project is in a feasibility stage, there are no geotechnical exploration along the alignment proposed in the hydraulic design. However, it is necessary for the project planner to know the magnitude of the civil works and the equipment required for pipe jacking installation.

According to the coordinates of the shafts, it is possible to obtain preliminary geotechnical parameters from Figures 15 to 18 from chapter 3.

Table 11. Geotechnical parameters for example #1

Friction angle of the soil			**Bulk unit weight of the soil**			**Water table**			**Cohesion**		
Φ' (degrees)			**γ (kN/m³)**			**H₁ (m)**			**c´(kPa)**		
Min.	**Max.**	**Average**	**Min.**	**Max.**	**Average**	**Min.**	**Max.**	**Average**	**Min.**	**Max.**	**Average**
27	31	29	18.2	18.6	1.84	2	2.25	2.13	10	19	14.5

3. Computed jacking forces

With the previous information, the next step is to compute the jacking forces. It will be used the Auld's method for cohesionless soil.

Figure 47 shows the force required for the segment between shafts (total length 153 m).

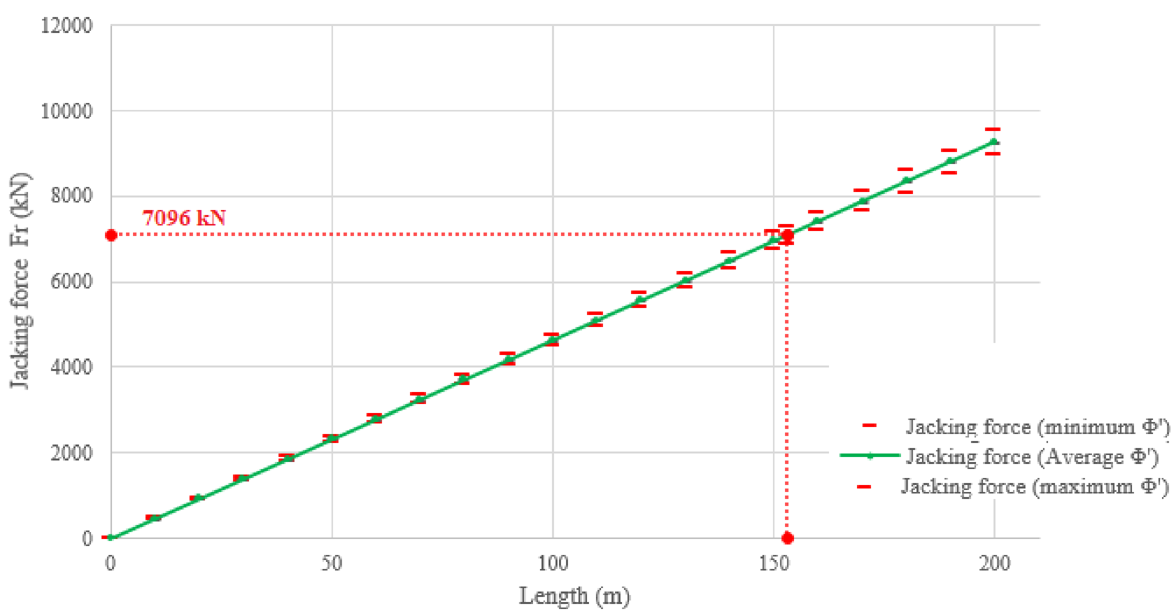


Figure 47. Computed jacking force in example #1

From Figure 47, it can be seen that the jacking force required for the total length between shafts is 7096 kN. Additional points (red lines) corresponds to the computed jacking force for minimum and maximum variation of the angle of friction. The variation is ± 3% respect to maximum and minimum value of (Φ′).

4. Variation of jacking forces

Step 3 showed the variation of the jacking force for the maximum and minimum values of the angle of friction. Now, it will be show how the computed force can vary if (γ) is not constant in the entire length between shafts.

Figure 48 shows how the variability of (γ) affects the magnitude of jacking force.

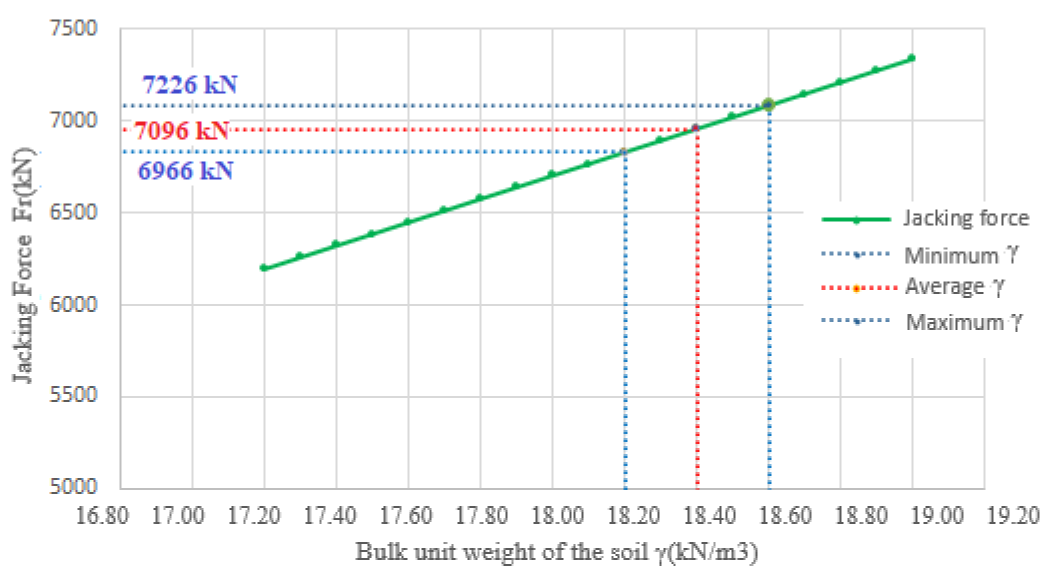


Figure 48. Variation for jacking force with γ in example #1

Figure 48 presents the jacking forces for the minimum, maximum and average values of (γ):

- Minimum jacking force: 6966 kN (for γ_{min} 18.2 kN/m^3)
- Maximum jacking force: 7226 kN (for γ_{max} 18.6 kN/m^3)
- Average jacking force: 7096 kN (for $\gamma_{average}$ 18.4 kN/m^3)

The relative difference between maximum and minimum values respect to the average is 2%.

5. Recommendations for future steps of the project (detailed design).

According to the analysis carried out in steps 1 to 4, designer and planers of this project should consider an average value of 7096 kN (jacking force).

Civil works and mechanical equipment as the reaction wall, the internal space for the shafts, the hydraulic jacks, and other elements need to be dimensioned considering this jacking force magnitude.

Quantification of this infrastructure is a valuable information to approach the cost of the project and move on to a next stage of detailed design.

5.1.2 Practical example #2

Geological formations in the Aburrá Valley corresponds to alluvial and slope deposits, according to geological and geotechnical information described in section 3.3.

Most of those deposits have variable thickness in the valley and usually in the first 10 m – 20 m there are soils composed by a gravel, sand and silt with embedded rock balls (García, 2006).

Taking into account this general information, in this example will be presented the computed jacking forces using Auld´s theory for cohesionless soils in 10 municipalities of the Aburrá Valley.

Typical diameters for pipes in which the pipe jacking technique can present economic advantages will be taken into account (see Figure 3, section 1.4), in order to have information about the jacking forces needed for each project location. Calculations will be presented for a typical length of 120 m and installation depth H =5.0 m.

Installation depth of pipe (H) and total length between shafts (L_t) were assumed as fixed values, because pipe jacking is a trenchless method that offers economic advantages for depths above 5.00 m and lengths above to 50 m (Ueki, M; Hass, C; Seo, J, 1999). However, straight distances greater than 120 m are not common, considering the topography conditions and available spaces in the Aburrá Valley.

Geotechnical parameters for each municipality will be taken according to information from *Armonización de la microzonificación sísmica de los municipios del Valle de Aburrá.* (Universidad de Los Andes, 2016) (see Figures 15 to 18).

Table 12. Computed jacking forces in the Aburrá Valley municipalities

Municipality	Geotechnical parameter (average)				Internal pipe diameter	Jacking force
	Φ'(°)	γ (kN/m^3)	c'(kPa)	H_1 (m)	D_i (m)	F_r (kN)
Barbosa	25	15.6	33	2.5	1.00	2259
					1.20	3085
					1.50	4418
					1.80	5830
					2.00	6805
Girardota	27	17.1	30	2.75	1.00	2834
					1.20	3872
					1.50	5551
					1.80	7333
					2.00	8565
Copacabana	29	18.1	24	3.00	1.00	3306
					1.20	4525
					1.50	6498
					1.80	8597
					2.00	10048
Bello	31	17.4	32	2.00	1.00	2700
					1.20	3711
					1.50	5364
					1.80	7132
					2.00	8360
Medellin	25	17.0	27	1.80	1.00	2343
					1.20	3182
					1.50	4536
					1.80	5974
					2.00	6968
Itagui	29	18.2	15	2.20	1.00	2947
					1.20	4033
					1.50	5798
					1.80	7680
					2.00	8984
Envigado	24	16.5	12	1.75	1.00	2166
					1.20	2937
					1.50	4180
					1.80	5497

Municipality	Geotechnical parameter (average)				Internal pipe diameter	Jacking force
	Φ’(°)	γ (kN/m^3)	c´(kPa)	H_1 (m)	D_i (m)	F_r (kN)
					2.00	6407
Sabaneta	25	16.0	40	3.00	1.00	2605
					1.20	3548
					1.50	5064
					1.80	6666
					2.00	7771
La Estrella	27	18.4	28	2.00	1.00	2854
					1.20	3886
					1.50	5556
					1.80	7333
					2.00	8563
Caldas	28	18.7	40	2.20	1.00	3052
					1.20	4164
					1.50	5966
					1.80	7886
					2.00	9215

Table 12 shows general values of jacking forces that can be used in planning and feasibility stages in future projects where pipe jacking is a good alternative. These values should be taken only as a reference and additional studies should be carried out to detail with greater precision the jacking forces for detailed design and construction.

CONCLUSIONS AND PERSPECTIVES

Trenchless technology like pipe jacking must be evaluated considering technical and economic aspects, in order to have well criteria to select the right construction method for new pipelines. There are some combinations of installation depths and pipe diameters that are not recommended for trenchless technologies as presented in chapter 1, for this reason designers and planners have to study the local conditions of the project, including technical, environmental and social aspects to recommend the construction method in a specific project.

Theoretical models presented in this document can be used to compute the jacking forces in early stages of planning and design, however, designers should estimate the variation of computed values due to variability of geotechnical parameters.
In detailed design, designers can do numerical simulations using a finite element software, in order to optimize the required jacking forces during construction (J. Yen, K. Shou, 2015).

Further research should be done in future projects that will be developed in the Aburrá Valley, in order to study technical aspects on pipes during installation. This could be done using instrumented pipes with sensors to measure radial and tangential stresses of soil on pipes, joint deflections, ground movements, settlements, interface friction between pipe and soil and jacking forces. This investigation would help to validate theoretical and numerical models and allow to have valuable information for the design process of pipe jacking projects in the Aburrá Valley.

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