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ANALYSIS OF CONTINOUS DISPLACEMENT OF A LANDSLIDE: CASE STUDY OF THE CHARMAIX BRIDGE IN SOUTH EAST FRANCE.

A thesis submitted in partial fulfilment of the requirements for the degree of Master of Engineering (MEng) in Civil Engineering.

Curriculum: Geotechnics

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DEDICATION

This work is dedicated to my entire family.

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LIST OF ABBREVIATIONS AND ACRONYMS

µ:	micrometre
α:	Anchor inclination to the horizontal
a.c:	Alternating current
A:	Cross-sectional area of steel
AASHTO:	American Association of Highway Transportation Officials
BSM:	Bishop's Simplified Method
C:	cohesion
φ**:**	frictional angle
Ca:	Adhesion between grouted anchor and soil/rock.
CSS:	Critical Slip Surface
d.c:	Direct current
DEM:	Digital Elevation Models
EM:	Pressuremeter Modulus
EPTM:	Ménard Pressuremeter Modulus
ERT:	Electrical Resistivity Tomography
FHWA:	Federal Highway Administration
Fig:	Figure
FoS:	Factor of Safety
InSAR:	Interferometry Synthetic Aperture Radar
JSM:	Janbu Simplified Method
k:	Coefficient of permeability
Kh:	Horizontal seismic force coefficient
LEM:	Limit Equilibrium Method
LIDAR:	Light Detection and Ranging
LL:	Liquid limit
M-PM:	Morgenstern-Price Method
Pf:	Creep Pressure
Pl:	Ménard Limit Pressure
PL:	Plastic limit
PMT**:**	Pressuremeter Test
Pu:	Ultimate anchor pull-out resistance

R:	percent allowable stress
RFs:	The shear reduction factor
S.E:	South East
S:	Anchor spacing
SAGE:	Société Alpine de Géotechnique
SD:	Sondage Destructifs
SP:	Sondage Pressiométrique
U. S.:	United States
σu:	Guaranteed ultimate tensile strength (GUTS) of steel in tendon
RFT:	Tensile capacity reduction factor
TC:	Tensile capacity of the tendon
q_u:	Ultimate bond stress

ABSTRACT

The objectif of this work was to interpret the monitoring data gotten during the surveying of a landslide affecting the Charmaix bridge in S.East France, and predict a potential sliding movement and if necessary propose a solution which would stabilize the sliding mass. The monitoring techniques used in this study were inclinometers which permitted the identification of the potential sliding surface and the direction of the landslide, piezometers for water level detection. Numerical interpretations of data were done by creating a slope model, then the model was analysed using the Limit Equilibrium software SLOPE/W. Interpretation of inclinometer displacement curves showed sharp displacements in the downslope direction at the decayed shale layer between the hard shale bedrock and the overlying moraine formation. This indicated that this surface of discontinuity could be a potential sliding surface. A back-analyses was then performed so as to get the residual shear strength parameters just before failure, the residual strength friction, φ'r = 18° was then used in subsequent analyses. It was also noticed from the numerical analysis that the main cause of the landslide was the occurrence of tectonic shifts in the study are which produced seismic vibrations. Installations of six rows of ground anchors to top and bottom faces of the slope was proposed as a possible remediation to the sliding movement. The slope became stable after these anchors were installed since analysis of the slope after installation of ground anchors produced an optimum factor of safety of 1.255 when anchors were placed at an inclination of 25° to the horizontal, contrary to the unreinforced analysis which gave us a lower factor of safety (FoS = 0.613).

Keywords: Landslide, tectonic shift, monitoring, stability, sliding movement, limit equilibrium software, SLOPE/W, ground anchors, back-analyses.

RESUME

L'objectif de ce travail était d'interpréter les données de surveillance obtenues lors de l'étude d'un glissement de terrain affectant le pont de Charmaix dans le sud-est de la France, et de prédire un mouvement potentiel de glissement et si nécessaire de proposer une solution qui stabiliserait la masse de glissement. Les techniques de surveillance utilisées dans cette étude étaient des inclinomètres qui ont permis d'identifier la surface de glissement potentiel et la direction du glissement de terrain, des piézomètres pour la détection du niveau d'eau. Les interprétations numériques des données ont été faites en créant un modèle de pente, puis le modèle a été analysé en utilisant le logiciel SLOPE/W. L'interprétation des courbes de déplacement des inclinomètres a montré de forts déplacements dans la direction de la pente descendante au niveau de la couche de schiste décomposée, entre le socle de schiste dur et la formation de moraine sus-jacente. Cela indique que cette surface de discontinuité pourrait être une surface de glissement potentielle. Une rétro-analyse a ensuite été effectuée afin d'obtenir les paramètres de la résistance au cisaillement résiduelle juste avant la rupture, cette résistance résiduelle, $\varphi'r = 18°$, a ensuite été utilisée dans les analyses suivantes. L'analyse numérique a également montré que la cause principale du glissement de terrain était l'apparition de mouvements tectoniques dans la zone d'étude qui ont produit des vibrations sismiques. L'installation de six rangées d'ancrages au sol sur les faces supérieure et inférieure de la pente a été proposée comme une solution possible au mouvement de glissement. La pente est devenue stable après l'installation de ces ancrages puisque l'analyse de la pente après l'installation des ancrages au sol a produit un facteur de sécurité optimal de 1,255 lorsque les ancrages ont été placés à une inclinaison de 25° par rapport à l'horizontale, contrairement à l'analyse non renforcée qui nous a donné un facteur de sécurité plus faible (FoS = 0,613).

Mots clés: Glissement de terrain, déplacement tectonique, surveillance, stabilité, mouvement de glissement, logiciel d'équilibre limite, SLOPE/W, ancrages au sol, rétro-analyses.

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GENERAL INTRODUCTION

Landslides are downslope movements of rock, debris or earth under the influence of gravity, which may cover a wide range of spatial and temporal scales. Most landslides occur at steep slopes, but they can also happen in low relief areas in connection with excavations by rivers or construction work. Landslides can be triggered by natural environmental changes or by human activities. Earthquakes, volcanic activity, heavy rainfalls and changes of ground water level are typical natural triggering mechanisms for landslides, which amplify the inherent weakness in rock or soil. Major earthquakes have caused widespread landslides (e.g., Oldham, 1899; Wilson and Keefer, 1979; Harp and Jibson, 1996). Topographic slopes fail during earthquakes because addition of gravitational and seismic accelerations causes short lived stresses in excess of the combined cohesive and frictional strength of underlying rock and soils (Newmark, 1965). Landslides may result in severe human casualties, property losses and environmental degradation. Therefore, it is well justified that maintaining the stability of slopes is a critical aspect of any geotechnical project.

The Charmaix Bridge found in South East France was constructed in the years 1976 to 1977 and links two mountains together. The bridge has been permanently subjected to landsliding movements linked to tectonic shift. Observations of this instability movements were first made in the years 1980, when two beams situated around pile no.7 which were normally spaced of 5cm were now in contact. The engineers then noticed that the layer of slope surface displaced itself at a slow rate of about 1cm to 2cm each year. In 1986, several solutions were proposed among which; reconstruction of the structure, replacement of the structure with an embankment, reconstruction of the structure's foundations. But the cost of these were estimated to be more than 50million euros. Finally, engineer Jean Tonnello proposed a solution which was to re-position the different piles at the level of their foundations. But this solution guaranteed just 40years of lifespan to the structure and the operations on the piles caused damage to the structure (L. Brassac, 2003). A decision was then taken to reconstruct the structure for a cost of about 30million euros, but before reconstruction the stability of the mountains had to be verified and remediated, this justifies the importance of this work.

The principal objectives of this work is to interpret the data obtained during the monitoring of the slope by the company SAGE, then from this data predict a potential sliding movement of the slope. After the prediction, a possible solution is to be proposed to render the slope stable. To attain these objectives, this work has been separated into three chapters.

In chapter one, landslides will be presented as a form of soil failure, so the chapter begins with a discussion on soil definitions, composition, different factors influencing soils formation and the different processes in soil formation. The problem in this thesis being a geotechnical one, some geotechnical properties of soils are discussed. After that, soils will be classified and then soil failure mechanisms seen. Here, more emphasis will be placed on landslides. Landslides are defined, classified, different causes presented and landslide monitoring techniques also presented. Proceeding from there, landslide modelling techniques are discussed. The chapter is then concluded with an essay on different soil stabilization or remediation methods. The methods utilised in collecting and analysing the different data are discussed in chapter two. The different hypothesis which will be considered in coming out with different slope and material models will be presented here. Then the chapter will be concluded with the presentation of the different models to be analysed. The results and interpretations of the field investigations and monitoring survey data are presented in chapter three. The study site will be presented, its geological and climatic features, also, here results of the different numerical simulations will be presented, with the proposed remediation solution also designed in this chapter.

The third chapter is then followed by a general conclusion and some recommendations made for an amelioration of future works.

CHAPTER 1: LITERATURE REVIEW

Introduction

Unlike metals, concrete, wood, and other common engineering materials, soils do not respond to the usual stress, strain, and strength relationships of the more elastic materials. Lacking uniformity because of the varied origins and heterogeneous compositions, soils must be sampled and tested and the data analysed by special means and techniques. Although soil is the oldest and most common material used by man for his works, only within recent decades has the science of soil mechanics been developed to its present state of capability. Despite the progress soils science has made, increased engineering requirements over the years to come demand even more soils research.

With the evolution of time, different scientists have given many different definitions to soil to show the evolution of the modern concept of soil. According to Carlos M. et al. (2018), Soil is an unconsolidated natural set of solid mineral particles that result from physical disintegration and chemical decomposition of rocks, which may contain organic matter and voids between the particles, isolated or linked, which may contain water or air. Under the perspective of civil engineering, soil is any uncemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks, the void spaces between the particles containing water and/or air. Weak cementation can be due to carbonates or oxides precipitated between the particles or due to organic matter. If the products of weathering remain at their original location they constitute a residual soil. If the products are transported and deposited in a different location they constitute a transported soil, the agents of transportation being gravity, wind, water and glaciers. During transportation the size and shape of particles can undergo change and the particles can be sorted into size ranges.

Landsliding, been one of the primary soil failure modes, this chapter begins with the different constituents of soils and its different formation processes. This is followed by the different processes intervening in the soil formation, then some properties of the soil are presented and the different soil types presented too. The second part of this chapter deals with the different modes of soil failure with a greater emphasis placed on landslides. Landslides are then defined, classified and the causative agents discussed. After that, some landslide monitoring methods are presented and the chapter concluded by the presentation of some landslide remediation or stabilization techniques.

1.1. Constitution of soils

The basic components of soil are minerals, organic matter, water and air. The typical soil, as seen on figure 1.1 consists of approximately 45% mineral, 5% organic matter, 20-30% water, and 20-30% air. These percentages are only generalizations at best. In reality, the soil is very complex and dynamic. The composition of the soil can fluctuate on a daily basis, depending on numerous factors such as water supply, cultivation practices, and/or soil type.

The water may be bound in variable degrees, and the characteristics of the solid particles are affected by physical and chemical makeup, which may change with time and environment. Depending upon the geological processes, soil deposits can vary from loose to dense, from uncemented to highly cemented, and the particle distribution can vary from poorly graded (highly sorted) to well graded (little or no sorting). As transported soils are moved and reworked, abraded, further weathered, mixed with organic materials and soluble minerals, and leached, all in varying degrees, they eventually form the material that concerns the soils engineer.

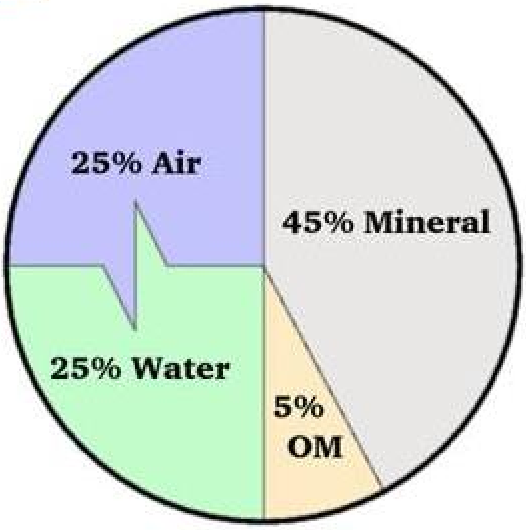

Figure 1.1. Approximate composition of soil.

1.2. Formation of soils

A soil is said to have formed when it attains, through natural processes, an appearance (morphology) that is significantly different from the underlying parent material (K.T. Osman, 2013).
Soil formation is a long term process. It takes several million years to form a thin layer of soil. As soil is a complex mixture of various components, its formation is also more complex. The formation of a particular type of soil depends upon the physicochemical properties of the parent rock and some other factors.

1.2.1. Factors affecting soil formation

Jenny (1941) explained that any soil property is the function of five soil-forming factors such as climate, organism, relief, parent material, and time .Soil is the result of the combined activity and reciprocal influence of these factors.

1.2.1.1. Climate

The precipitation and temperature are two most important climatic factors that affect physical, chemical and biological weathering. The rise in temperature also increases rate of biochemical reactions. The rainfall in substantial quantities create favourable environment for plant growth. The vegetation varies from areas of high rainfall to low rainfall and hence varying organic matters in soils.

1.2.1.2. Organisms

Activity of soil organisms plays a significant role in soil formation by accumulating organic matter and cycling of nutrients. Micro-organisms encourage acidic conditions which eventually determine the kind of soil formation process that occur.

1.2.1.3. Parent material

Soil texture is greatly influenced by parent material which in turn affects movement of water and nutrients. Similarly, mineralogical and chemical composition of parent material have direct bearing on weathering. For example, soil acidity in humid condition can be delayed in case of soils made up of limestone.

1.2.1.4. Topography

Topography is generally described in terms of slope, elevation. The steep slope is vulnerable to erosion whereas flat land slows the process of erosion. In most regions, soils formed from similar parent material under the same climatic conditions present differences due to their positions on the landscape.

1.2.1.5. Time

The time determines the effect of weathering. The alluvial material generally does not get sufficient time to develop in comparison to upland soils.

1.2.2. Soil formation processes

Soil formation being a very long and complex mechanism, several processes take place for a soil to completely change from its parent material to its new nature.

i. **Accumulation of materials**

Materials are added to the soil such as organic matter and decomposing materials or new mineral materials deposited by the forces of ice, water or wind and they accumulate over time. This happens in the top layer of the soil.

ii. **Leaching and losses**

Leaching is the removal of soluble components of the soil column. As water washes down through the soil it can carry away bases such as calcium, held as exchangeable ions in clay humus complexes, as well as acidification through the substitution of hydrogen ions. Through the movement of water, wind, ice or the uptake of the accumulated materials by plants, the new particles including clay, organic matter, silt or other chemical compounds are leached and eroded away or taken up from the soil by plants. As a result, the physical and chemical compositions of the new accumulated materials together with the soil parent material are altered.

iii. **Transformation and illuviation**

Here the soil particles held in the suspension after the leaching are transformed after which they accumulate. Transformation is the chemical weathering of silt, sand, and the formation of clay minerals as well as the change of organic materials into decay resistant organic matter. Illuviation is the accumulation of particles held in suspension such as clay.

iv. **Podsolization and translocations**

Podsolization occurs when strongly acid soil solutions cause the breakdown of clay minerals. As a result, aluminium, silica and iron form complex materials together with organic compounds in the soil. These materials and the other accumulations are translocated within the profile and/or between the horizons.

v. **Laterization**

It is a pedogenic process common to soils found in tropical and subtropical environments. High temperatures and heavy precipitation result in the rapid weathering of rocks and minerals.

vi. **Calcification**

It occurs when evapotranspiration exceeds precipitation causing the upward movement of dissolved alkaline salts from groundwater. At the same time, the movement of rain water causes a downward movement of the salts. The net result is the deposition of the translocated cations in the B horizon. In some cases, these deposits can form a hard layer called caliche.

vii. **Gleying**

Gleying occurs in waterlogged, anaerobic conditions when iron compounds are reduced and either removed from the soil, or segregated out as mottles or concretions in the soil.

1.3. Properties of soils

Three groups of soil properties can be distinguished in engineering, physical properties, mechanical properties and geotechnical properties.

1.3.1. Physical properties

Physical properties concerns notions of weight, density, permeability to liquids, gases, heat, and notions of resistance.

1.3.1.1. Porosity

The total porosity (f) is the volume occupied by pores (Vf) per unit volume of soil (Vt). It is an index of relative pore volume in soil and is generally expressed as percentage as in equation 1.1.

$$f = \frac{Vf}{Vt} * 100 \qquad (1.1)$$

Its value varies between 30 to 60%. Porosity is lower in the coarse textured soils than in the fine textured soils but the size of individual pores is larger in the coarse textured soils than in the fine textured ones. In clayey soils, the total porosity is highly variable as the soil alternatively swells, shrinks, aggregates, disperses, compacts and cracks during wetting and drying.

Two types of pores (macro and micro) occur in soils without any clear demarcation. Usually, pores larger than 0.06 mm in diameter are considered as macropores (water conducting) and those smaller are called as micropores (water retaining) or capillary pores. Macropores allow easy movement of water and air, whereas these movements are restricted to some extent in the micropores. Pore space directly controls the amount of water and air in the soil.

1.3.1.2. Bulk density

Bulk density (ρb) of a soil is the oven- dried mass (Ms) per unit volume (Vt) of soil as a whole including pore space (see equation 1.2).

$$\rho b = \frac{Ma}{Vt} \qquad (1.2)$$

The bulk density of soil is influenced by texture, structure, moisture content, organic matter and management practices of soil. In coarse textured soils, bulk density varies from 1.40 to 1.75 Mg m-3 while in fine textured soils, it normally ranges from 1.10 to 1.40 Mg m-3. The bulk density decreases with increase in organic matter content and fineness of soil texture. Higher values of bulk density indicate more compactness of the soil.

1.3.1.3. Soil colour

Soil colour provides valuable information regarding soil conditions and some properties of soils. For example, dark coloured soils absorb more solar radiation and warm up faster than the light coloured soils. Soil colour is also used for soil classification and interpretation and description of soil profiles. The presence of excessive salts, soil erosion etc., can also be easily identified from the soil colour.

1.3.1.4. Soil structure

The primary soil particles do not exist as such in natural conditions but are bonded together into larger units or aggregates usually termed as secondary particles. These aggregates formed under natural conditions are called peds whereas an irregular shaped coherent mass of soil formed during tillage operations is called a clod. Soil structure is defined as the arrangement of primary and secondary soil particles in a certain structural pattern. This arrangement results in formation of different sized soil pores, therefore, soil structure may also be defined as the arrangement of various sized soil pores in a certain structural pattern.

1.3.2. Geotechnical soil properties

1.3.2.1. Permeability

The amount, distribution, and movement of water in soil have an important role on the properties and behaviour of soil. The engineer should know the principles of fluid flow, as groundwater conditions are frequently encountered on construction projects. Water pressure is always measured relative to atmospheric pressure, and water table is the level at which the pressure is atmospheric. Soil mass is divided into two zones with respect to the water table: (i) below the water table (a saturated zone with 100% degree of saturation) and (ii) just above the water table (called the capillary zone with degree of saturation ≤ 100%) (P.P. Raj, 2012).

Prakash and Jain (2002) explained that water flowing through soil exerts considerable seepage forces, which have direct effect on the safety of hydraulic structures. The rate of settlement of compressible clay layer under load depends on its permeability. The quantity of stored water escaping through and beneath an earthen dam depends on the permeability of the embankment and the foundation respectively. The rate of drainage of water through wells and excavated foundation pits depends on the coefficient of permeability of the soils. Shear strength of soils also depends indirectly on its permeability, because dissipation of pore pressure is controlled by its permeability. According to U. S. Bureau of Reclamation, soils are classified as (i) Impervious: k (coefficient of permeability) less than 10-6 cm/sec, (ii) Semi pervious: k between 10-6 to 10-4 cm/sec (iii) Pervious: k greater than 10-4 cm/sec. Table 1.1. shows the permeability range in some soil types.

Table 1.1. Coefficient of permeability values in some soils (Reasearchgate.net, 2021).

Soil Type	K (cm/sec)
Clean gravel	1.0-100.0
Coarse sand	1-0.01
Fine sand	0.01-0.001
Silt	0.001-0.00001
Clay	Less than 0.000001

1.3.2.2 Specific gravity

Specific gravity is the ratio of the mass of soil solids to the mass of an equal volume of water. It is an important index property of soils that is closely linked with mineralogy or chemical composition (P.P. Raj, 2012) and also reflects the history of weathering (Tuncer, E.R. and Lohnes, R.A., 1977). Higher value of specific gravity gives more strength for roads and foundations. It is also used in calculation of void ratio, porosity, degree of saturation and other soil parameters (Prakash and Jain, 2002). Typical values of specific gravity are given in Table 1.2.

Table 1.2. Typical values of specific gravity (Bowles, 2012).

Type of soil	Specific gravity
Sand	2.65-2.67
Silty sand	2.67-2.70
Inorganic clay	2.70-2.80
Organic soil	1.00-2.60

1.3.2.3. Consistency limits

The consistency of a fine-grained soil is largely influenced by the water content of the soil. A gradual decrease in water content of a fine-grained soil slurry causes the soil to pass from the liquid state to a plastic state, from the plastic state to a semi-solid state, and finally to the solid state. The water contents at these changes of state are different for different soils. The water contents that correspond to these changes of state are called the Atterberg limits. The water contents corresponding to transition from one state to the next are known as the liquid limit, the plastic limit and the shrinkage limit (S.R. Kaniraj, 1988).

The liquid limit of a soil is the water content, expressed as percentage of the weight of the oven dried soil, at the boundary between the liquid and plastic states of consistency of the soil (IS: 2720 – Part 5, 1970), the soil has negligibly small shear strength (S.R. Kaniraj, 1988). The plastic limit of a soil is the water content, expressed as a percentage of the weight of oven dried soil, at the boundary between the plastic and semi-solid states of consistency of the soil (IS: 2720 – Part 5, 1970).

Skempton (1953) observed that the plasticity index of a soil increases linearly with the percentage of the clay-sized fraction. Laskar and Pal (2012) found that plasticity depends on grain size of soil. With the increase of sand content plasticity index of soil decreases, which might be due to decrease of inter molecular attraction force. Due to decrease of attraction force, liquid limit of the soil decreases and accordingly plasticity index decreases. But as the clay content increases inter molecular attraction force increases and liquid limit increases.

The shrinkage limit is the maximum water content expressed as a percentage of oven-dried weight at which any further reduction in water content will not cause a decrease in volume of the soil mass, the soil mass being prepared initially from remoulded soil (IS: 2720 – Part 6, 1972). The finer the particles of the soil, the greater are the amount of shrinkage. Figure 1.2 shows the different Atterberg's limits.

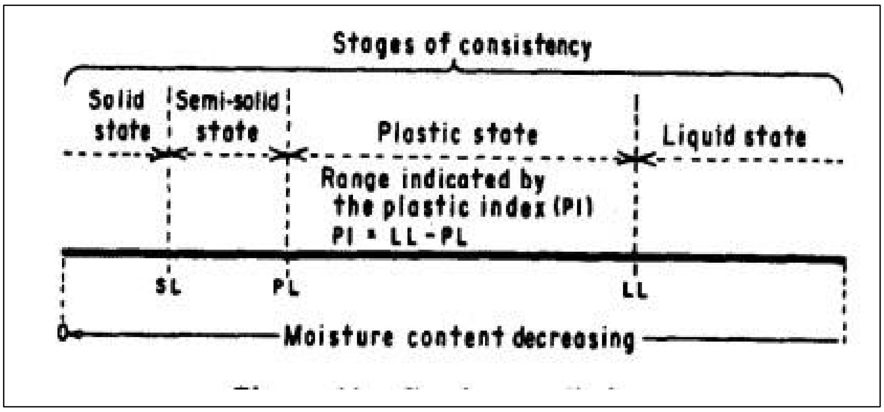

Figure 1.2. Consistency limits (B. Schrefler, 2010).

1.3.2.4. Particle size

The percentage of different sizes of soil particles coarser than 75 μ is determined by sieve analysis whereas less than 75 μ are determined by hydrometer analysis. Based on the particle size analysis, particle size distribution curves are plotted. The particle size distribution curve (gradation curve) represents the distribution of particles of different sizes in the soil mass (Mallo, S.J. and Umbugadu, A.A., 2012). It gives an idea regarding the gradation of the soil i.e. it is possible to identify whether a soil is well graded or poorly graded. In mechanical soil stabilization, the main principle is to mix a few selected soils in such a proportion that a desired grain size distribution is obtained for the design mix. Hence for proportioning the selected soils, the grain size distribution of each soil is required to be known (Prakash and Jain, 2002).

Apparao and Rao (1995) explained that the grain size analysis is widely used in classification of soils. The data obtained from grain size distribution curves is used in the design of filters for earth dams and to determine suitability of soil for road construction, air fields, etc. Raj (2012) stated that the particle size of sands and silts has some practical value in design of filters and in the assessment of permeability, capillarity, and frost susceptibility. Very relevant and useful information may be obtained from grain size curve such as (i) the total percentage of larger or finer particles than a given size and (ii) the uniformity or the range in grain-size distribution.

Bowles (2012) found that particle-size is one of the suitability criteria of soils for roads, airfield, levee, dam, and other embankment construction. Information obtained from particle-size analysis can be used to predict soil-water movement, although permeability tests are more generally used.

The interactions among different geotechnical properties of soils can help the researchers while designing the foundations for different types of civil engineering structures.

1.3.3. Mechanical properties of soils

1.3.3.1. Shear strength resistance

The shear resistance of soil is the result of friction and the interlocking of particles and possibly cementation or bonding at the particle contacts. The shear strength parameters of soils are defined as cohesion and the friction angle. The shear strength of soil depends on the effective stress, drainage conditions, density of the particles, rate of strain, and direction of the strain. Thus, the shearing strength is affected by the consistency of the materials, mineralogy, grain size distribution, shape of the particles, initial void ratio and features such as layers, joints, fissures and cementation (S.J. Poulos, 1989). The shear strength parameters of a granular soil are directly correlated to the maximum particle size, the coefficient of uniformity, the density, the applied normal stress, and the gravel and fines content of the sample. It can be said that the shear strength parameters are a result of the frictional forces of the particles, as they slide and interlock during shearing (Yagiz, S., 2001). Soil containing particles with high angularity tend to resist displacement and hence possess higher shearing strength compared to those with less angular particles (G. Ranjan and A.S.R. Rao, 1991).

Different researchers (P.P. Raj, 2012, Prakash and Jain 2002, S.R. Kaniraj, 1983) explained that the capability of a soil to support a loading from a structure, or to support its overburden, or to sustain a slope in equilibrium is governed by its shear strength. The shear strength of a soil is of prime importance for foundation design, earth and rock fill dam design, highway and airfield design, stability of slopes and cuts, and lateral earth pressure problems. It is highly complex because of various factors involved in it such as the heterogeneous nature of the soil, the water table location, the drainage facility, the type and nature of construction, the stress history, time, chemical action, or environmental conditions.

As per Prakash and Jain (2002), confining pressures play the significant role in changing the behaviour of soils in deep foundations. Similarly in high rise earth dams, the confining pressures are of very high magnitude. The triaxial test is the only test to simulate these confining pressures. For short term stability of foundations, dams and slopes, shear strength parameters for unconsolidated undrained or consolidated undrained conditions are used, while for long term stability shear parameters corresponding to consolidated drained conditions give more reliable results.

Akayuli et al. (2013) found that the friction angle is high for a sandy soil than its cohesion and vice versa for clayey soil. Shanyoug et al. (2009) in their study concluded that there is a general increase in cohesion with clay content. As more clay is introduced into the sandy materials, the clay particles fill the void spaces in between the sand particles and begin to induce the sand with interlocking behaviour. Hence, clayey sand soils are expected to exhibit low cohesion whereas the cohesion increases with high clay content.

Dafalla (2013) observed that the mineralogy can have a major role in the shearing strength capacity of clays. The cementation between particles can either be due to a chemical bond or physicochemical bond. Swelling and shrinkage in expansive soils are of two extreme opposite effects on the shearing strength. The shear strength is generally low for fully expanded clay while dry shrinking clay is capable of developing higher cohesion and angle of internal friction. The study indicated that choosing the appropriate mix or using appropriate quantity of clay, can help to achieve required shear strength. Very moist clay-sand mixture showed steep drop in both cohesion and angle of internal friction when the clay content is high. Figure 1.3 shows a triaxial test apparatus.

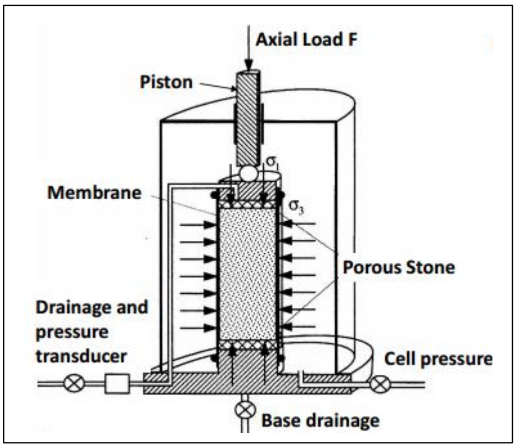

Figure 1.3. Triaxial test apparatus (B. Schrefler and P. Delage, 2010).

1.3.3.2. Resistance to compression

Information on how much a soil mass will ultimately compress under load and what time will be required for a given portion of the ultimate compression to take place is required in the study of foundation settlement and the volume change within earthworks. The phenomenon of compressibility is associated with changes in volume of voids and only to be a minor extent with the volume changes of the solid particles. If the voids of a soil mass are largely filled with air, the application of a load will result in a relatively rapid compression, conversely if the soil voids are filled with water, time is required for the pore water to drain from the soil mass. This phenomenon is called consolidation.

Abeele (1985) explained that lowering of water table or dewatering is probably the best known cause of massive settlement. When submerged, soil particles are subjected to buoyancy. Upon dewatering, the buoyancy is removed and the apparent increase in pressure results in consolidation, even though there is no increase in external load. Vibrations can also have a densification effect on soils and lead to subsequent settlement. Soils often fail and settle disastrously as a result of earthquakes. Devastating landslides are often one of the results of such occurrences.

1.4. Types of soils

A number of systems of classification have been evolved for categorizing various types of soil. Some of these have been developed specifically in connection with ascertaining the suitability of soil for use in particular soil engineering projects. Some are rather preliminary in character while a few are relatively more exhaustive, although some degree of arbitrariness is necessarily inherent in each of the systems (Shanmukha, 2016). The more common classification systems are: (i) Geological classification (ii) Classification by structure and grain-size (iii) Unified soil classification system (iv) Preliminary classification by soil type.

1.4.1. Geological classification

Soil types may be classified on the basis of their geological origin. The origin of a soil may refer either to its constituents or to the agencies responsible for its present status. Based on constituents, soils may be classified under two main classes.

(i) organic

(ii) Inorganic.

Based on the agencies responsible for their present state, soils may be classified seven classes, namely; residual soils, transported soils, sedimentary soils, Aeolian soils, glacial soils, lascutrine and marine soils.

Over the geological cycle, soils are formed by disintegration and weathering of rocks. These are again formed by compaction and cementation by heat and pressure.

1.4.2. Classification by structure and grain-size

Depending upon the average grain-size and the conditions under which soils are formed and deposited in their natural state, they may be classified on the basis of their structure as; soils of single-grained structure, soils of honey-comb structure and soils of flocculent structure.

In the grain-size classification, soils are designated according to the grain-size or particle-size. Terms such as gravel, sand, silt and clay are used to indicate certain ranges of grain-sizes. Since natural soils are mixtures of all particle-sizes, it is preferable call these fractions as sand size, silt size, etc.

1.4.3. Unified soil classification system

This system was originally developed by A. Cassagrande and adopted by the U.S. Corps of Engineers in 1942 as 'Airfield Classification'. It was later revised for universal use and redesignated as the "Unified Soil Classification" in 1957. In this system soils are classified into three broad categories.

(i) Coarse-grained soils with up to 50% passing No. 200 ASTM Sieve

(ii) Fine-grained soils with more than 50% pass No. passing No. 200 ASTM Sieve

(iii) Organic soils

1.4.4. Preliminary classification by soil types

Familiarity with common soil types is necessary for an understanding of the fundamentals of soil behaviour. In this approach, soils are described by designation such as boulders, cobbles, sand etc. A description of each designation is given in what follows.

(i) Boulders: Boulders are the rock fragments of large size, more than 300mm in size.

(ii) Cobbles: Cobbles are large size particles in the range of 80mm to 300mm.

(iii) Gravel: It is a type of coarse-grained soil. The particle size ranges from 4.75mm to 80mm. It is a cohesion-less material.

(iv) Sand: It is a coarse-grained soil, having particle size between 0.075 mm to 4.75 mm. The particles are visible to naked eye. The soil is cohesionless and pervious.

(v) Silt: It is a fine-grained soil, having particle size between 0.002 mm and 0.075 mm. The particles are not visible to naked eyes. Inorganic silt consists of bulky, equidimensional grains of quartz. It has little or no plasticity, and is cohesionless.

(vi) Clay: It consists of microscopic and sub-microscopic particles derived from the chemical decomposition of rocks. It contains a large quantity of clay mineral. It can be made plastic by adjusting the water content. It exhibits considerable strength when dry. Clay is a fine-grained soil. It is a cohesive soil. The particle size is less than 0.002mm.

1.5. Soil behaviour and failures

Knowledge of soil landscapes, soil formation, and the various soil properties and functions has expanded with a classification system oriented to the interpretation of the soil survey. Information about soil properties provides a basis for assessing risks and hazards to buildings and other structures of human populations (G.B. Muckel, 2004). Some of the principal soil failure modes are seen in the sections that follows.

1.5.1. Swelling and shrinking of soils

1.5.1.1. Definition

The soil shrinkage is defined as the specific volume change of soil relative to its water content and is mainly due to clay swelling properties (Haines, 1923). It can be measured in most soils with more than 10% clay content (Boivin et al., 2006) and shows an S-shape curve as in figure 1.4.

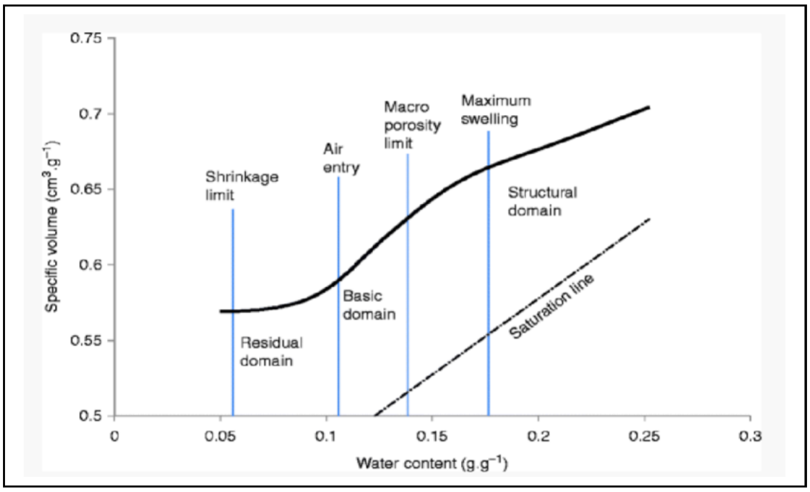

Figure 1.4. Variation of specific volume with water content (Boivin et al., 2006).

Swelling and shrinkage are not wholly reversible procedures (Holtz and Kovacs, 1981). Shrinkage results in cracks on the body of the soil which cannot close-up completely by rewetting the soil. Therefore, it makes the soil to bulk-out somehow. In addition, it can help the water to penetrate into the soil more easily during the swelling process. It is worth mentioning that when substances like sediment enters the existing cracks in the soil, the soil is unable to get rid of them and go back to its previous situation, and hence it will result in the increase in the swelling pressure. Sometimes, the shrinkage cracks may be filled with sediment which leads to the incompatibility of the soil.

1.5.1.2. Causes of shrinking and swelling in soils

Fine-grained, clay-rich soils can absorb large quantities of water after rainfall, becoming sticky and heavy. Conversely, they can also become very hard when dry, resulting in shrinking and cracking of the ground. This hardening and softening is known as 'shrink–swell' behaviour. In some places, subsidence can be caused by the movement of the ground with changing weather conditions, for example dry summers or wet winters. The amount by which the ground can shrink and/or swell is determined by some factors such as; the water content in the near surface, the type of clay in soil and its propensity to change volume, normal seasonal movements associated with changes in rainfall and vegetation growth, changes to surface drainage.

1.5.1.3. Effects of shrinkage and swelling in soils

The main effects of shrinkage in soils are the heaving and subsidence of soils.

i. Heaving

Damage to buildings may occur when the volume change of the soil, due to shrinking or swelling, is unevenly distributed beneath the foundations. For example, if there is a difference in water content in the ground beneath a building, swelling pressures can cause the wall to lift; this is often called 'heave'. This can happen at the corners or towards the centre of a building.

ii. Subsidence

Subsidence is a lowering or collapse of the ground. It can be triggered by an artificial disturbance, a change in drainage pattern, heavy rain, and water abstraction. Subsidence has the potential to cause engineering problems such as damage to foundations, buildings and infrastructure. Dry weather and high temperatures have been found to be a major factor in the emergence of subsidence in clay soils.

1.5.2. Landslide

1.5.2.1. Definitions

The slope instabilities and their movements are posed to different names such as massive movement, slope movement, dip movement, landslide etc. However, the most used term in the engineering field is landslide.

Hutchinson(1988), Cruden (1991), Cruden and Varnes (1996) described landslides as a rapid mass wasting process that causes downslope movement of mass rock, debris, or earth which are induced by a variety of external stimulus.

According to Cosier (1986), landslides is a downward and outward movement of slope forming material without the involvement of surface runoff as transportation medium, under the influence of gravity. Meanwhile Brusden (1985) gave a more restricted definition to landslides as a unique way of mass transport which does not require transportation agent for slope movement.

Landsliding can also be described as a complex geological and geomorphological process that occurs when the resistance of the soil or rock deteriorates.

The principal force acting on the material mass found on a slope is the gravitational force. This gravitational force acting on the slope mass can be resolved into two main components (Fig 1.5), a component acting perpendicular to the slope known as the resisting component and a component acting tangential to the slope, known as the driving component.

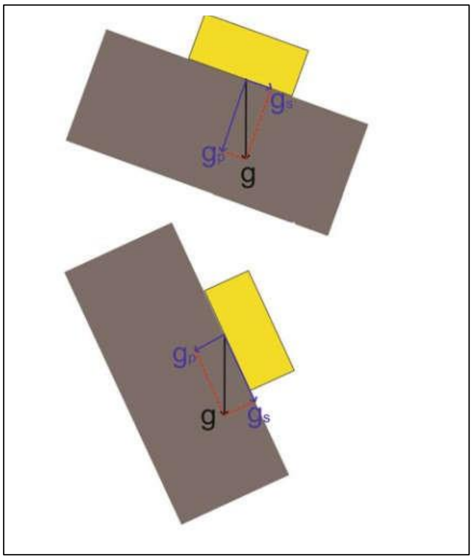

Figure 1.5. Gravitational force and its components (S. Pradhan, landslide practice & modelling, 2019).

Mass movement or land sliding occurs when the driving forces exceed the resisting forces to pull the slope forming material down the slope (Keller EA, 2000). The relation between resisting and driving forces is used in quantifying the stability of a slope, and the degree of stability of a slope is known as its factor of safety.

1.5.2.2. Factor of safety notion

The computed stability of a slope can be expressed as the ratio of resisting forces to driving forces and is defined as the factor of safety (FoS).

When FoS < 1 slope failure will occur, with FoS > 1 indicating stable slope conditions. FoS < 1 can be reached by increasing the driving force, for example, by greater loading on the slope. The FoS has for long been the parameter used in characterizing the stability of slopes, thereby letting to the recognition of susceptible landslides.

i. **Resisting forces**

The resisting force is the available shear strength (stress) along the potential sliding surface multiplied by the area in sliding. They act opposite to the direction of the motion and tend to resist movement. The shear strength of the material is a function of soil properties such as cohesion and friction angle, cohesion being the ability of the particles to hold together and the angle of friction being the measure of the frictional forces acting between the grains (S. P. Pradhan, 2019).

ii. **Driving forces**

Contrary to resistive forces, driving forces act in the direction of the motion and promotes the down slope movement .The major driving force is gravity, which plays a significant role in the initiation of the mass wasting phenomena (S. P. Pradhan and T. Siddique, 2019).

When the shear stress (driving force) exceeds the shear strength (resistive force), the slope forming material will fail. Thus, on steeper slopes, the tangential component of gravity is greater than the resistive component and cause down slope movement of the mass. Slope angle, height of the slope, climatic conditions, types of slope materials, runoff and groundwater etc. are some other significant factors that affect the magnitude of the driving force. Water plays an important role in slope instability. When water is added to a slope, it causes slope failure due to the additional loading on the slope, accelerating erosion rates and increasing pore pressure that ultimately leads to the reduction in shear strength of the slope forming material. Considering the above parameters FoS can be calculated from Eq.1.3.

$$\text{FoS} = \frac{C' + h * g\cos(\gamma r - \gamma w) * tan\varphi}{\gamma r * h * g\ sin\theta\ cos\theta} \qquad \textbf{(1.3)}$$

Where,

C': effective cohesion

h: thickness of potential slide

g: acceleration due to gravity

θ: dip angle of potential slide plane

γr: Material unit weight of potential sliding plane

γw: unit weight of water

Examination of shear parameters and the stress-strain behaviour of materials are primarily experimental, because of the technical difficulties to study the processes in nature. Shear parameters are generally determined in the laboratory by undertaking uniaxial or triaxial shear tests (Wu 1996). A relatively undisturbed soil sample is placed into a shear box and stress is applied until the material fails. Applied loads and subsequent strains are recorded.

1.5.2.3. Parts of a landslide

The different parts of a landslide depend upon the type of mass movement. Various parts of typical slump-earth flow type landslide have been depicted in Fig.1.6.

- Crown: Practically undisplaced material adjacent to the highest part of main scarp.
- Main Scarp: Steep surface on undisturbed ground at upper edge of landslide caused by movement of displaced material.
- Head: Upper parts of landslide along contact between displaced material and main scarp.
- Minor Scarp: Steep surface on displaced material of landslide produced by differential settlement within displaced material.
- Main Body: Part of displaced material of landslide that overlies surface of rupture between main scarp and toe of surface of rupture.
- Foot: Portion of landslide that has moved beyond the toe of surface of rupture and overlies original ground surface.
- Toe: Lower, usually curved margin of displaced material of a landslide, most distant from main scarp.
- Surface of Rupture: Surface that forms lower boundary of displaced material below original ground surface, also termed slip surface or shear surface.
- Toe of Surface of Rupture: Intersection between lower part of surface of rupture and original ground surface.
- Surface of Separation: Part of original ground surface now overlain by foot of landslide.

Figure 1.6 show the different parts of a typical landslide movement.

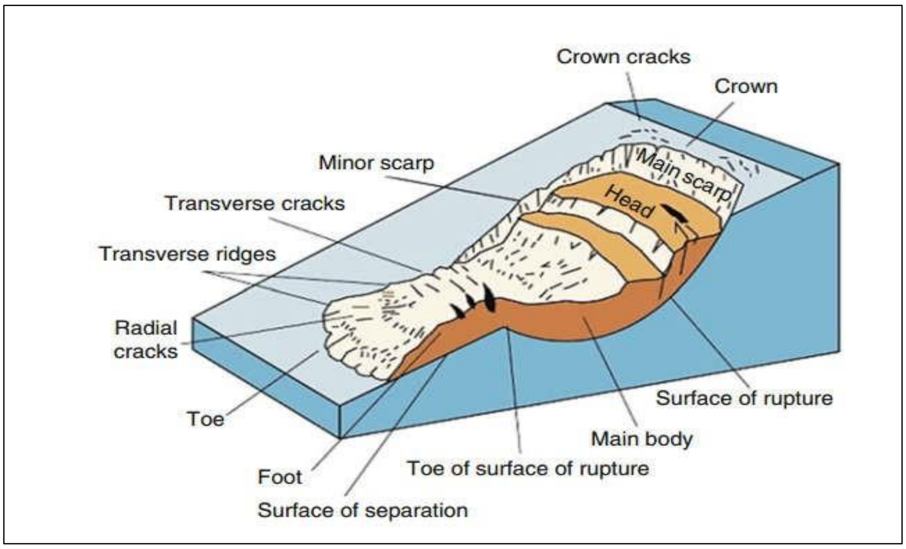

Figure 1.6. Parts of slump-earth type landslide taken form Highland and Bobrowsky (The landslide handbook-A guide to understanding landslides).

1.5.2.4. Landslides classification

Different landslide classifications have been proposed worldwide, most of them consider different parameters like types of movement (falling, sliding, flows), material involved (rock, soil, debris) and rate of movement (rapid or slow) (S. P. Pradhan and T. Siddique, 2019). But despite the wide number of classifications, the system of classification devised by the late D.J Varnes has been the most widely used system of classification.

a) Falls

A fall occurs with the detachment of the mass from a steep slope or cliff without with no shear displacement (S. P. Pradhan and T. Siddique, 2019). Descends are mostly through air by free fall as shown in figure 1.7. After impact with the ground, the mass can roll, slide, or bounce along the slope depending on the energy and constitution of the slope. The movement is very rapid to extremely rapid.

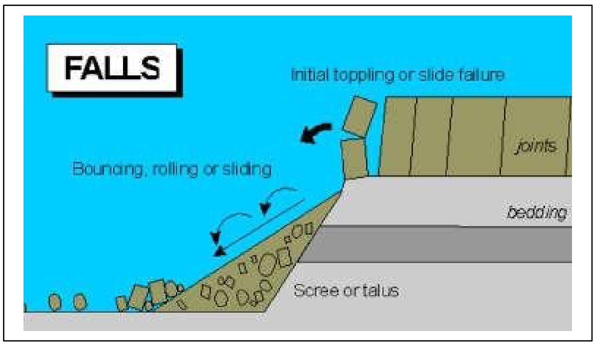

Figure 1.7. Example of fall movement

b) Topples-

It is the forward rotation of a volume of soil or rock about some pivot points under the action of gravity and forces exerted by adjacent units or fluids in cracks (see fig.1.8). In jointed rock mass, closely spaced and steeply dipping discontinuity sets that dip opposite to the slope surface are necessary prerequisites for toppling failure.

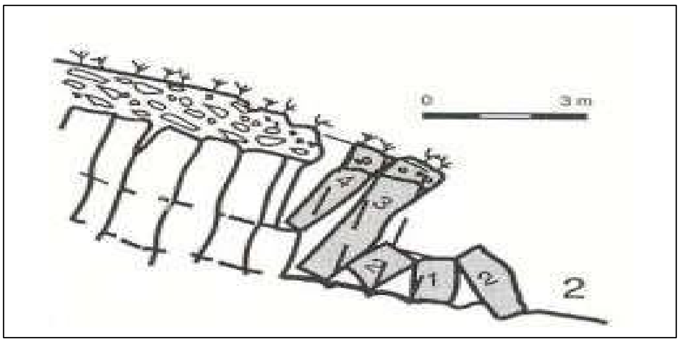

Figure 1.8. Example of topple movement

Figure 1.9. Block topple in limestone, Czech Republic.

c) Slides

Slides are restricted where there is a distinct type of weakness that separates the slide material from more stable material .The mass is isolated by the shear band has a movement almost rigid with small distortional strain .The sliding movement may be translational or rotational, or a combination of both. In rotational slides, the material slides outward and downward on one or more concave failure surfaces that impart a backward tilt to the slipping mass. (S. P. Pradhan and T. Siddique, 2019). Figure 1.10 shows an example of sliding movement.

d) Spreads

Spreads involve the fracturing and lateral movement of rock or soil masses due to the lateral displacement of a lower layer with less strength. Here there is not an evident sliding surface, but the lower layer behaves as a viscous material. The causes are the liquefaction of a saturated granular soil or the loss of cohesion in a cohesive hard soil. The upper more rigid mass breaks itself into several blocks that seem to be floating above the lower consistent material as in figure 1.10.

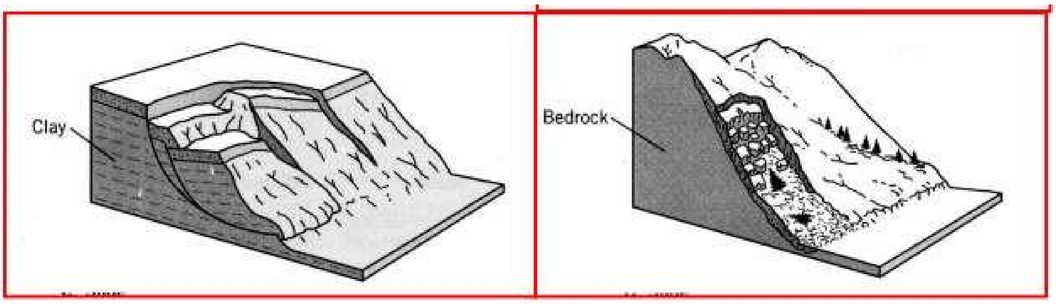

Figure 1.10. Sliding movement in clay and rock (Source: Varnes, D.J., 1978).

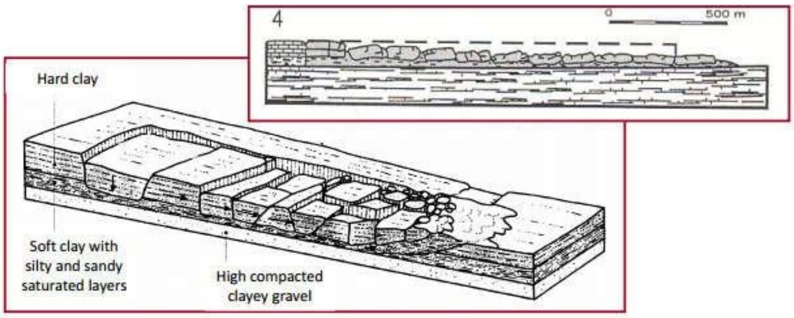

Figure 1.11. Mechanism of spread movement (Source: Varnes, D.J., 1978).

e) Flows

It is the detaching of a mass of soil or rock with a subsequent movement similar to that of a viscous fluid. It is characterized by the presence of several sliding surfaces and the mass does not exhibit a uniform displacement but the displacement varies inside the mass.

Generally, the mass can cover a long path before stopping itself and it is possible to individuate 3 zones: the depletion basin, the intermedium channel and arresting tail. Figure 1.12 shows an example of a flow movement.

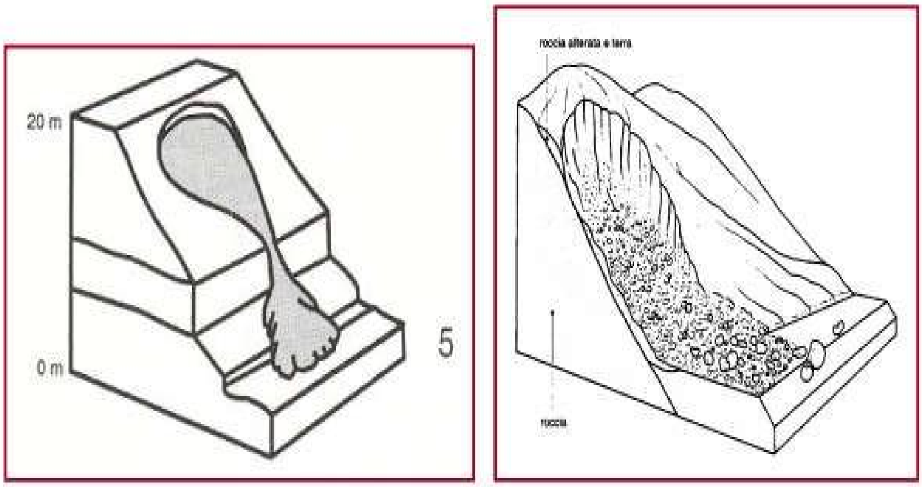

Figure 1.12. Flow movement (Source: Varnes, D.J., 1978).

f) Composite or Complex

Complex mass movements are the combination of two or more principal types of mass movements, whereby a part of the detached mass moves in accord with a type and another part in accord with another type.

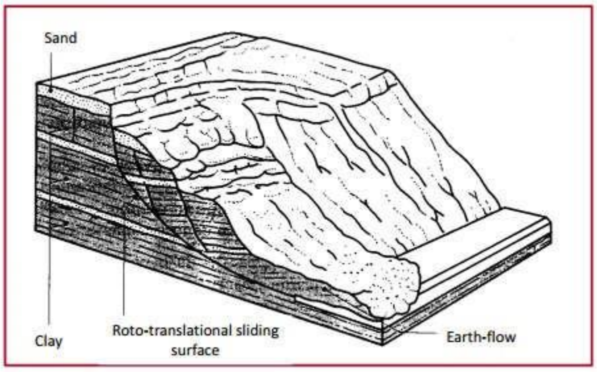

Figure 1.13. Combination of slide and flow movements (Source: Varnes, D.J., 1978).

Generally, a landslide movement is always made up of a combination of two or more movements. The cases of single movements discussed are ideal cases.

1.5.2.5. Causes of landslides

For a landslide to be remediated, the first thing which should be known by the engineer is the cause of the landslide. This avoids a "one solution fits all" mentality which in the long-run can turn to be ineffective or very expensive. Landsliding or mass movement results from both internal and external influences (Varnes, 1978), however, it should be emphasized that there is usually a dominant cause (D.Cornforth, 2005). Different types of mass movements reflect that there exist a diversity of causative factors. Referring to Terzaghi's classification the two types of causative factors can be sorted.

i. External or Exogenic factors

1. Removal of lateral or underlying support as a result of a stream or road cuts from a marginally stable slope can often result in a landslide.
2. Loading or surcharge of the upper edge of the slope following construction, landfill dumping or other factors.
3. Changes in relative relief (local differences in attitude) or slope gradient. The steeper the slope, greater is the chances of its failure. An increase in the steepness or gradient, of a slope leads to an increase in shear stress on the potential rupture plane and to a decrease in normal stress. Such increase in slope gradient may be due to undermining of the foot of the slope by stream erosion or by excavation.
4. Terrain uplift associated with plate tectonics directly implies soil movement. Large earthquakes can induce rock uplift and can also erode mountains and lead to occurrence of landslides. The vibrations resulting from earthquakes decrease resistive forces in slopes thereby favouring driving forces.

ii. Internal or endogenic factors

1. Weathering causes disintegration that weakens soil and decreases its resistance to shearing.
2. The roots of the plants tend to hold soils together, even for a certain period of time after the plant has died until they decay. They can account for up to 90 percent of the stability of certain soils. Hence, deforestation may eventually lead to instability on the denuded slopes.
3. Infiltration of water can lead to soil saturation. This may be caused by poorly organised drainage on a slope that has been modified by deforestation or urbanisation. Saturation increases pore water pressure and increases density and weight of the surface layer, thus exerting a positive force which may cause the slope to fail.

1.5.2.6. Landslide monitoring techniques

Monitoring of deformation of structures and ground surface displacements can be achieved using different types of systems and techniques. Based on Mikkelsen (1996), the landslide monitoring techniques are divided in monitoring of surface movements, monitoring of pore water pressure inside the landslide, monitoring of ground displacements and others. The monitoring equipments could be further divided based on how the measurements are performed, which could be manual or automatic. These techniques could then be classified as remote sensing or satellite technique, geophysical techniques, and geological techniques.

a) Remote sensing or satellite techniques for landslide monitoring

Traditional direct field monitoring techniques have the major drawback of being in physical contact with the monitored object. For this reason, the measurement systems may incur two problems. Firstly, they may influence the system and, therefore, the measured quantities, and secondly, they may be influenced by the system.

Remote sensing refers to the science and technology used in gathering information about an event's occurrence or a phenomenon without establishing direct contact with it. A remote monitoring sensing is composed of a data logger containing three main components.

i. A source of alimentation able to supply a.c or d.c current
ii. A recorder for data storage
iii. A device for transmission.

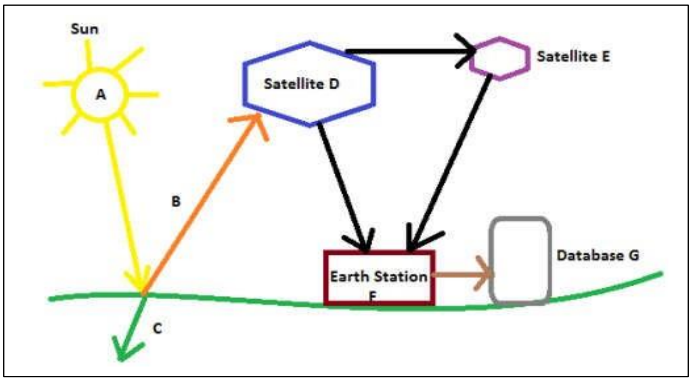

Figure 1.14. Remote sensing architecture.

Figure 1.14 illustrates a generic remote sensing system architecture. Component A in the figure is the sun considered as a source of illumination and energy. Components B and C illustrate the energy being reflected by the Earth's surface and also some being absorbed by the Earth. D and E are satellites used to receive and retransmit this reflected energy in order to detect features on the Earth's surface. F and G correspond to the usage of this data to generate understandable information such as Digital Elevation Models (DEM), raster and vector graphs etc. (M. Scalon, 2014).

Several different sensors can be applied to different typologies of landslides. Broadly speaking, sensors could be categorized as a function of multiple properties, for example, the observed quantity (geometric deformation, geotechnical, hydraulic, or geological property), the location (in-situ, on surface or underground, ground-base, airborne), the rapidity of the output delivery (real time, quasi-real time, offline) and the way outputs are transmitted to the user (manual reading, automatic reading with internal data storage or online communication to a remote control station) (M. Scalon, 2014).

As recently verified by a review work by Tofani et al. (2013a), based on a large survey carried out in Europe, in 83% of the cases remote sensing is being used as a routine tool for landslide detection, mapping and monitoring. The study also reveals that 75% of the users takes advantage of a combination of 2 or more techniques. Despite the advantages, some major problems remain in the adoption of remote sensing methods for landslide monitoring, such as the lack of underground penetration, the lower frequency of acquisition with respect to direct automated methods and the atmospheric disturbance ((Byung-Gon et al., 2017).

b) Geophysical techniques in landslide monitoring

Investigations of subsurface landslide features are necessary to provide the input for modelling and subsequent predictions of potential failure events, for example, estimating the run out length, the mobilized volume, or the velocity of a potential failure event (Malet et al., 2005). In general, geophysical techniques identify spatial variations of a physical parameter of the subsurface, from which inferences on a range of processes and properties can be made (Everett M., 2013; Kearey P. et al., 2001). When applied to landslide investigation, geophysical techniques are able to target characteristics and features of landslide settings that are manifested by physical property contrasts in the subsurface (McCann D. and Forster A., 1990), including what follows.

1. The physical extent of the landslide, comprising critical features such as the subterranean slip surface and water table;
2. Variations in soil layers and their thicknesses ;
3. Variations in distribution and movement of moisture throughout the landslide body;
4. Variations in the geomechanical strength of the landslide body.

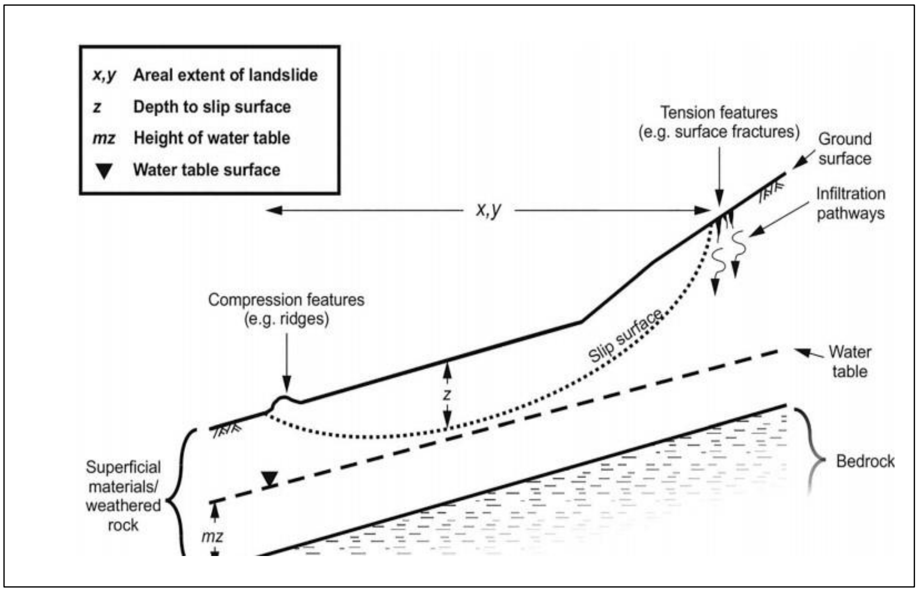

Figure 1.15. Features investigated with geophysical techniques (J.S. Whiteley et al., 2019).

The features of a typical landslide system that can be identified and assessed using geophysical methods are shown in figure 1.15. These features are typified by the existence of a physical discontinuity (e.g., slip surface, lithological contact) or a contrast in material properties (e.g., degree of saturation, clay content) being present in the subsurface.

In a geophysical survey, the result may determine the presence or extents of a landslide property (e.g., the water table). This time-static geophysical data set gives information on the state of one (or more) property in the system at that particular time, but not how that property will change in response to some external influence (e.g., precipitation). With the addition of a second geophysical data set at some point in the future, it may be possible to determine a change in that property (e.g., increase in water table height). The implicit assumption is that in order for a property change to have occurred, a process must have taken place in the time between the two surveys (e.g., infiltration). By looking at the differences between the data at each end of the time period, reasonable inferences can be made about the process that must have occurred to give rise to a change in the system.

Two broad types of geophysical methods can be identified: geoelectrical and seismic methods as seen in figure 1.16. In addition, two modes of acquisition are identified in both types: "active" modes, in which the recorded geophysical signal is artificially generated, and "passive" modes, in which the signal recorded is generated naturally (J.S. Whiteley et al., 2018).

Among various types of geophysical tests the most used are the electrical resistivity tomography (ERT) and geoseismic methods.

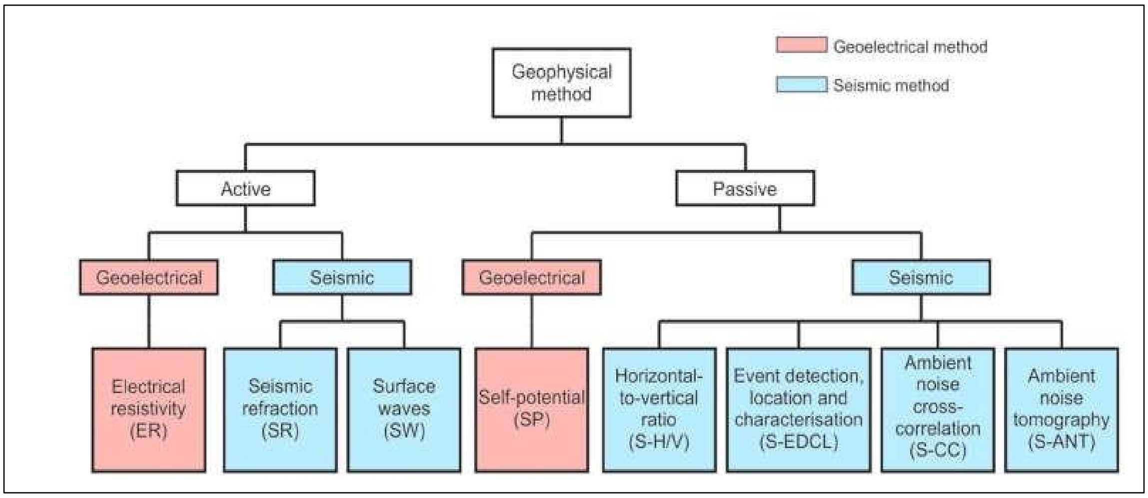

Figure 1.16. Geophysical methods shown by mode of acquisition (J.S. Whiteley et al., 2018).

1.5.3.6.1. Geotechnical techniques for landslide monitoring

Geotechnical techniques are used extensively in the monitoring of structures. These sensors are often placed within the structure and are out of sight (P.D. Savvaidis, 2003). A great number of methods are available for structural monitoring. These methods can however generally be classified into geodetic (surveying) methods and geotechnical methods. Geodetic methods are mainly used to monitor deformations while geotechnical methods can be used to determine some other important parameters beside deformations. The two types of methods complement each other in most of the times in terms of the types of information that they can obtain. It should be noted here that the above classification of the monitoring methods is mainly used in the fields of surveying and geodesy (Chrzanowski, 1994; Ding, et al., 1995). The engineers and geologists often refer all the methods as geotechnical methods (Hanna, 1985). There are a variety of geotechnical instruments that have been developed for deformation measurements. These include extensometers, inclinometers, tilt meters, pendulums and piezometers.

i. Extensometers

Extensometers are used to measure the relative movements between points. They can be applied, e.g., to measure the movements across a crack, inside or on the surface of a slope, as shown in Fig.1.17. Extensometers are made of various types of material, such as steel tapes and wires, tensioned or untensioned steel rods, and fiberglass, for different conditions of application. Extensometers usually use mechanical micrometres, electrical resistance and variable reluctance transducers.

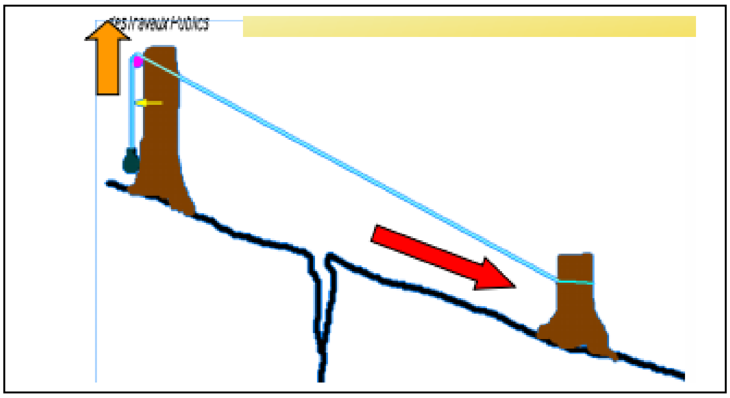

Figure 1.17. Invar wire extensimeter (Sassa, 1984).

Most extensometers currently in use have a digital readout. Readings can be taken by site personnel, or can be stored in an electronic data logger and transferred to a computer afterwards. A small number of mines have established telemetry systems in order to log and check the readings in real or near-real time.

Measurement ranges of the available extensometers vary from a few centimetres (crack meters) up to around 180 m (tensioned rod extensometers). Extensometers are commonly used for slope stability monitoring. They can be used either on the surface or inside a slope, and very easily linked to a data logger and alarm system.

ii. Inclinometer

Inclinometers are used to measure the subsurface lateral displacement of soil or rock. An electrical probe is usually lowered through a guide casing to the base of a near vertical borehole (see Figure 1.18). The probe is then pulled up while the inclination information of the probe in two orthogonal planes is registered at certain intervals. From this information, profiles of the borehole in the two planes can be derived and reviewed graphically. The lateral displacements of the borehole can be determined by comparing the measured profiles of the borehole obtained at different times. Boreholes of up to 200 m in depth can be measured using inclinometers. In practice it is usual to extend a borehole into stable ground in order to have a common reference point to compare borehole profiles for determining displacements. Inclinometers can also be placed permanently at important locations to log data continuously. In this situation the inclinometer is acting as a tilt meter (X.L Ding and H. Qin, 1995).

Inclinometers are ideal to measure the lateral displacements occurring within a slope. However, it is difficult to fully automate the process of measurement with inclinometers.

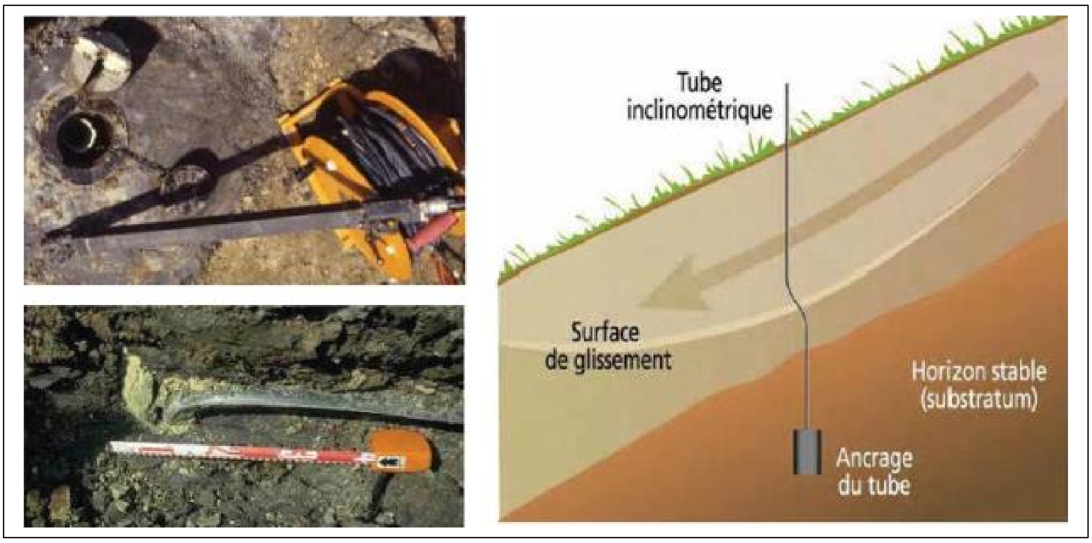

Figure 1.18. (a) And **(b)** Slip surface localisation with inclinometers (D. Cornforth, 2005).

iii. Instruments for measurement of ground water level and pore water pressure

The main aim of the water pore pressure monitoring is the knowledge of the pressure in some points in order to define the seepage characteristics inside an aquifer, to define the number of aquifers, and to define the maximum piezometric head and the position of the piezometric line or, in alternate, the position of the water table.

Ground water level can usually be measured through the use of a standpipe. Various instruments that are based on the use of mechanical, electrical, acoustic and pressure sensing gages can be used for this purpose. The commonly used ones include steel tape, audio reader, and piezometers.

Piezometers are instruments for the measurement of ground water pressures. They are usually used in an open standpipe, sealed in filled embankment, or driven into ground. The central part of a piezometer is a pressure gage, either a mechanical, electrical, hydraulic or pneumatic transducer. Therefore, we have the vibrating wire piezometers, twin-tube hydraulic piezometers, pneumatic transducers, electrical resistance piezometers, etc.

Multipoint piezometers can also be used to measure water pressures in different strata of the ground. Fig.1.19. shows an example of a piezometer.

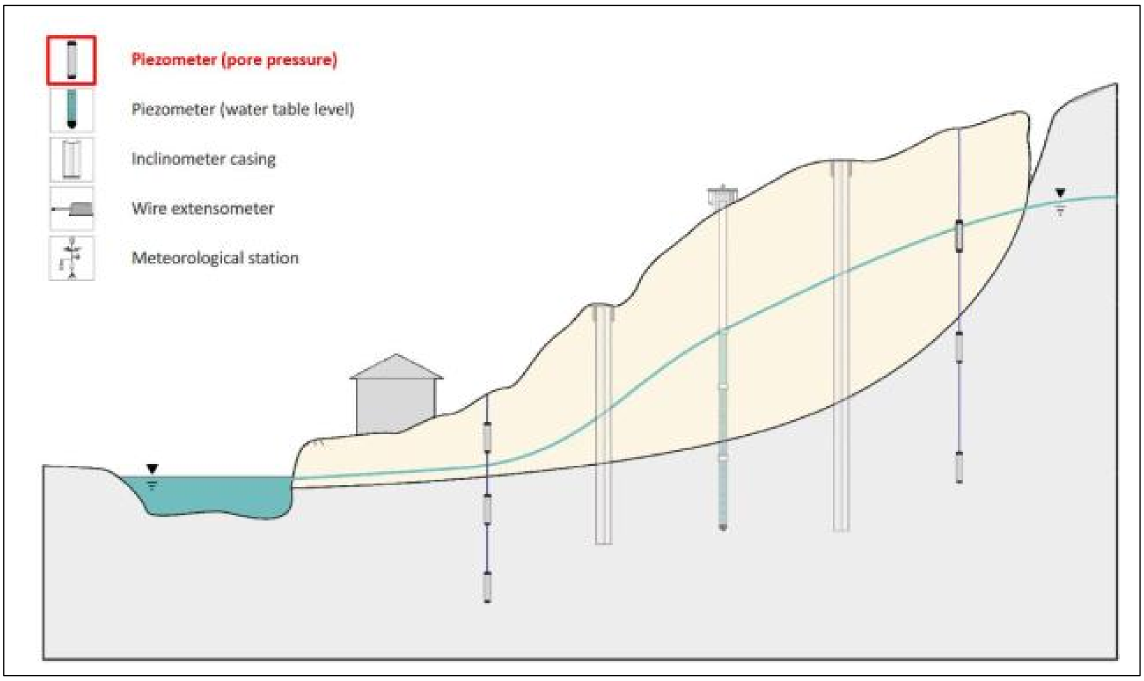

Figure 1.19. Water head determination with a piezometer (B. Schrefler and P. Delage,

1.5.2.7. Landslide modelling Techniques

Landslide modelling is, along with experimental subsoil exploration and experience driven safety assessment, one of the main tasks of slope stability practice (Janbu, 1996). Models are applied to analyse current stability status and to predict slope behaviour under certain conditions such as rainfall events or scenarios for environmental change. Moreover, models are used for the back analysis of already failed slopes and for assessment of effectiveness of geotechnical stabilisation measures (Barla et al. 2004). However, when dealing with modelling it is important to keep in mind that all models are necessarily simplified generalisation and approximations of processes which are occurring in nature (Favis-Mortlock and De Boer 2003).

Models for the analysis of single slope failures have a long tradition in geotechnical slope stability practice. These models have frequently been applied to assess the stability of human made or natural slopes, and the design of slopes, such as embankments, road cuts, open-pit mines etc. Moreover, physically-based models for single slopes allow detailed investigation of failure processes, assessment effects of triggering events, and assessment of the effectiveness of remedial measures and stabilisation works (B. Thiebes, 2011). Today, a wide range of computer calculation programs are available for numerical slope stability assessment. Despite the development of more sophisticated numerical models, limit equilibrium methodology is still widely applied (Abramson, 2002).

Limit-equilibrium methods provide a mathematical procedure to determine the forces within a slope that drive and resist movement. Limit-equilibrium analysis usually calculates stability for discrete two-dimensional slices of a slope and for assumed or known potential shear surfaces (Fig.1.20.), but three-dimensional approaches have also been developed.

Shear strength of materials along shear surface is assumed to be governed by linear or nonlinear relationship between shear strength and normal stress.

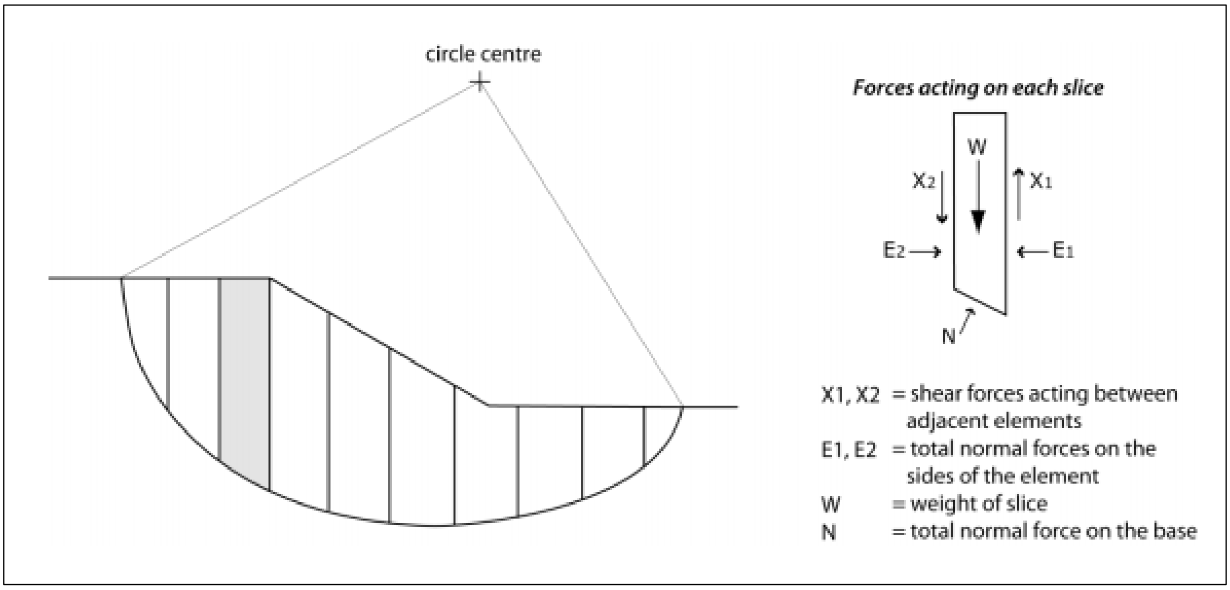

Figure 1.20. Simplified illustration of method of slices (based on Conolly, 1997)

The result of limit-equilibrium analysis is a global FoS for shear surface, which provides a snapshot on stresses and resisting forces relationship. Several numerical methods are available today which assist in locating critical shear surfaces, this is, where the lowest FoS is prevalent. The most widely applied methods are Bishop's simplified approach, which accounts for circular slip surfaces, and Janbu's method for con-circular, i.e. polygonal shear surfaces. Given the wide use of numerical limit-equilibrium methods for slope stability analysis in geotechnical practice, most available models allow assessment of effects of external loads or remedial stabilisation structures. However, limit-equilibrium methods also comprise some drawbacks. The resulting FoS represents a global value for a two-dimensional slope profile, where movement occurs if the FoS<1.0. However, Bonnard (2008) notes that limit-equilibrium methods only provide an approximation of force balance within landslides and that in reality displacements may occur with a FoS between 1.0 and 1.1 - 1.15. Moreover, the FoS is an average value for an assumed critical failure surface and provides no information about the actual distributions of stresses or the progressive development of unstable state (Eberhardt et al. 2004).

A second family of stability models on a local scale is concerned with continuum modelling. The entire slope mass is divided into a finite number of elements and represented as a mesh. Continuum approaches include finite-difference and finite element methods. Finite-difference methods provide numerical approximations of differential equations of equilibrium, strain-displacement relations or the stress-strain equations (Eberhardt 2004). In contrast, finite-element procedure exploits approximations to the connectivity of elements, and continuity of displacements and stresses between elements (Eberhardt 2004). However, in both methods the problem domain is discretised into a set of sub-domains or elements. In contrast to limit-equilibrium analysis, continuum modelling software allows for complex time-dependent landslide analysis by including constitutive models such as elasticity, elasto-plasticity and strain softening.

1.6.Soil stabilization techniques

Soil stabilization can be described as a process which incorporates the various methods used for improving the strength and stability of a given soil mass as well as other engineering and physical properties. Through stabilization, a better engineering performance is obtained, with a reduced chance of bad engineering behaviour.

1.6.1. Principles of soil stabilization

Different methods of soil stabilization are controlled by different factors and variables, as such an all governing principle cannot easily be described which encompasses all the methods of soil stabilization. However, it is generally accepted that before any method of soil stabilization is used irrespective of which, certain factors should be considered.

1. Evaluate the properties of the given soil type.
2. Decide the most suitable, effective and economical method of soil stabilization for supplementing the lacking properties.
3. Design the soil mix with stability and durability values.
4. Considering the construction procedure by adequately compacting the stabilized layers.

1.6.2. Methods of soil stabilization

1.6.2.1. Mechanical stabilization

The process of mechanical stabilization is one which entails the mixture of at least two or more types of natural soil in an attempt to change its gradation and by so doing improve the properties of the soil. This method tries to combine the engineering properties of the constituents of the soil mixture. It is aimed at reducing the void ratio by filling up the spacing between larger granular soil properties with finer soil particles through the combination of soils possessing different granular sizes followed by thorough compaction. The compaction process ensures that the void ratio is reduced improving the soil strength parameters such as cohesion (C) and angle of internal friction (φ).

1.6.2.2. Stabilization using cement

In cement stabilization, pulverized Portland cement is mixed with the soil alongside water and compacted to attain a strong material with an increased strength, durability and minimal moisture variations (Behzad and Huat, 2008). Thus far, cement stabilization has proved to be an effective method of soil stabilization with all soil types except clay, while other organic matter, when present in a soil reduces the strength of the cement stabilization.

1.6.2.3. Grouting

The grouting method of soil stabilization entails the injection of special fluid like materials either in suspension or solution, called grouts, into the ground with the goal of improving the soil. Grouting is usually done under high pressure. It utilizes stabilizers with high viscosity for soils with high permeability and is not primarily suitable for clay soil due to their low permeability. The method is essentially suitable for stabilizing buried zones with limited extent; say a previous stratum below a dam, and for stabilizing soils that cannot be disturbed. Grouting therefore aids in reducing the void spaces and increasing the load carrying capacity of the soil.

1.6.2.4. Stabilization using geotextiles

Geotextiles are those permeable sheets of synthetic fibres like polyester, polypropylene, polyamide nylon, etc (Tingle et al., 2002) while the geomembranes are basically impermeable sheets or films made from a polymer and which may be reinforced with a textile or may be made by spraying asphalt or resin directly on the ground or unto a geotextiles. The geotextiles though permeable can thus be diversified into impermeable geomembranes. The permeability of geotextiles is comparable to that of fine sand to coarse sand. They are quite strong and durable (Yetimoglu and Inanir, 2005). They are not affected by even hostile soil environment (Onyelowe, 2011). Geotextiles provide some important functions such as what follows.

i. Separation function

Geotextile have been used to separate the sub grade soils of roads from the ground base course, thereby preventing the ingress of gravel into the clayey sub-grade, consequently increasing the life of the roads.

ii. Soil reinforcement function

A geotextile layer provides reinforcement through the tensional membrane and tensile member functions acting as reinforcement. It ensures better load distribution especially true for heavy weight and high modulus fabrics.

iii. Filtration function of geotextiles

While designing the upstream and downstream protection works of a weir or a barrage floor, a protection inverted filter is laid on the ground and is covered with concrete blocks having filled open joints. This inverted filter consist of layers of fine sand, coarse sand and some stone aggregate laid from bottom to top. Such an arrangement allows the escape of water seeping from below the weir floor without allowing the foundation soil to be lifted upwards thereby reducing the possibility of piping. The filter gradation is such that while it allows free flow of seepage water, the foundation soil does not penetrate to clog the filter, thus reducing the possibility of failure of weir by piping undermining.

iv. Drainage function of geotextiles

Granular drainage filters are traditionally laid behind the masonry, a concrete retaining wall supporting a soil mass which helps in transmitting water.

Apart of these soil stabilization techniques, some other special techniques have been applied of particular soil instabilities such as those occurring on slopes. Decision on selecting an appropriate slope stabilisation method requires thorough evaluation of the existing slope conditions and assessment of the prevailing causes that are responsible for the instability of the slopes. Once the causes of slope instability are identified, the appropriate remedial measure for slope is selected on the basis of feasibility, stability and economy. These remedial measures are helpful in minimizing the chances of approaching slope failure by addressing its cause.

Correction of an existing landslide or the prevention of a pending landslide is a function of a reduction in the driving forces or an increase in the available resisting forces. Any remedial measure used must involve one or both of the above parameters.

The remedial works proposed by Popescu, (2002) carry four groups namely i) slope geometry, ii) drainage, iii) retaining structures and iv) internal slope reinforcement. An overview of each of these remedial measures shall be seen in what follows.

i. Geometrical Methods in landslide remediation

The reduction of a slope angle can be obtained by the addition of extra material or by an excavation. This method is simple and cheaper in cost but require sufficient space. Slope safety can be enhanced easily to convert steeper slopes into gentler ones. This method can be executed by trimming the slope or making the slope free from extra loading. Backfilling of toe also lies under this category. By changing the geometry of a steep slope to a gentler slope either flatten the slope or backfill at the toe of slope, the stability of a slope can be increased. This method is easy and most cost effective. However, it depends very much on the site condition. As there are existing building at the site, this method cannot be adopted (Chen C., 2001).

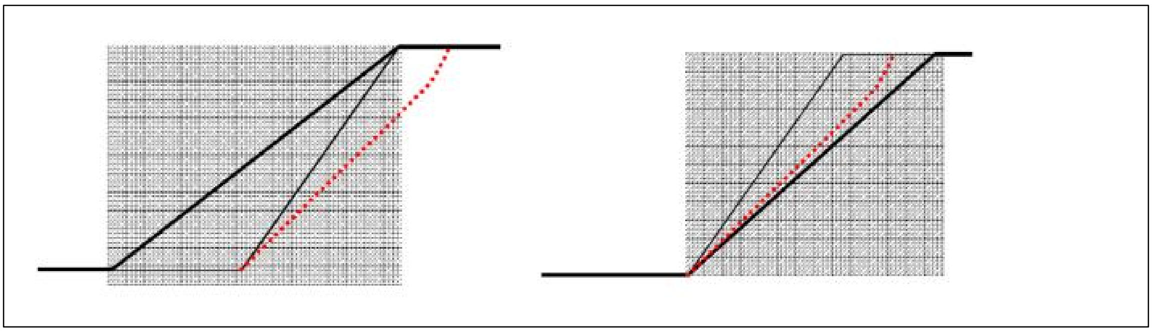

Figure 1.21. slope geometry modification (Van Impe, 1989).

As seen in figure 1.21, the efficacy of a corrective cut or fill is controlled by its location, weight and shape and the characteristics of the actual or potential landslide to be treated. The slope geometry can be modified by removal of removal of (actual or potentially) unstable soil/rock mass, removal of loose or potentially unstable blocks/boulders (trimming and scaling), or removal of material from driving area. Another technique to modify the slope profile consist in the soil benching, shown in figure 1.22. This modification can leave surface movements, but makes it possible to limit and control them. Moreover, the water that flows when it rains decreases its energy, so the erosion will be reduced. This choice is adopted in rock slope with upper layer landslides.

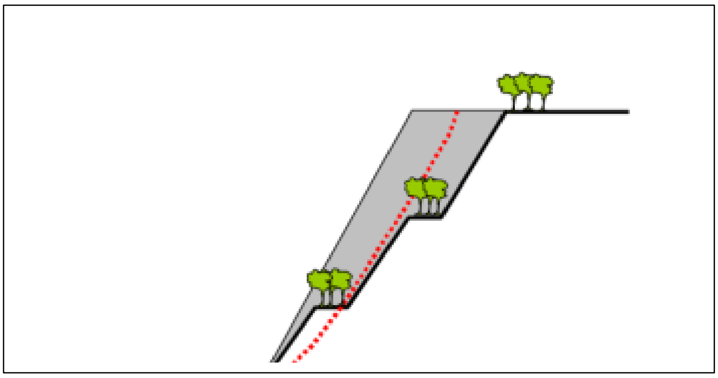

Figure 1.22. Soil benching in landslide remediation (Van Impe, 1989).

ii. Drainage Methods in Landslide Remediation

Drainages include all the operations that aim to reduce the pore pressure, with an increase in the effective stress and the shear strength of the soil. This type of operation is quite efficient, but it needs a good maintenance to keep working. The drainages operations include;

- Protection from erosion, which consist of covering the slope with biomaterial, synthetic material, or using vegetation.
- Control of surface water to avoid the free run-off of water along the slope with subsequent erosion and possible infiltration. This can be accomplished by using a trench draining system.
- Shallow drainage operations by using shallow trenches (Fig.1.23).
- Deep drainage operations such as deep wells.

Saturation of subsoil and pore water pressure building up are major factors causing the instability of slope. With the proper design of surface and subsurface drainage system, the chances of building up pore water pressure and saturation of subsoil can be minimized and therefore the stability of slope can be increased. However, as a long term solution to increase the stability of slope, this method suffers greatly because the drainage systems must be maintained if they are to continue to function It is always easy to maintain the surface drains but very difficult for the subsoil drains. This method is generally used in combination with other methods (Chen C., 2001).

Figure 1.23. Examples of a shallow and deep drainage trenches (Van Impe, 1989)

iii. Retaining Structures in Slope Stability Remediation

The retaining structures include all the operations that aim to stabilize the landslide, increasing somehow the external forces that help the stability. Retaining structures include gravity types of retaining wall, cantilever retaining wall, continuous bored piles, caisson, steel sheet pile, gabions walls and others as shown in figure 1.24. This method is generally more expensive as compared with the other methods. However, it is always the most commonly adopted method in remedial works due to its flexibility in a constraint site (Chen, C., 2001). Independently from the type of adopted structure it is necessary to define or verify the efficiency of the structure, the possibility of any other instability occurring, the geotechnical and structural design of the structure and its permeability.

iv. Internal Slope Reinforcement in Landslide Remediation

This includes operations such as soil nailing, use of micro piles, and use of bolts and anchors. The 'soil nailing' is a reinforcement technique for sub-vertical excavation or slope stabilization. With respect to other techniques it is cheap and rapid with minor environmental impact. Suitable soils for soil nailing are; residual soils and weathered rocks, hard cohesive soils, natural bonding dense sands.

Soil nailing consist in inserting solid or hollow steel or glass fibre bars into the face of an excavation or an existing slope to reinforce it, transferring part of the load from the potentially unstable mass to more competent strata. The nails work predominantly in tension, but are considered to work also in bending/shear, especially where the orientation is perpendicular to the anticipated shear surface.

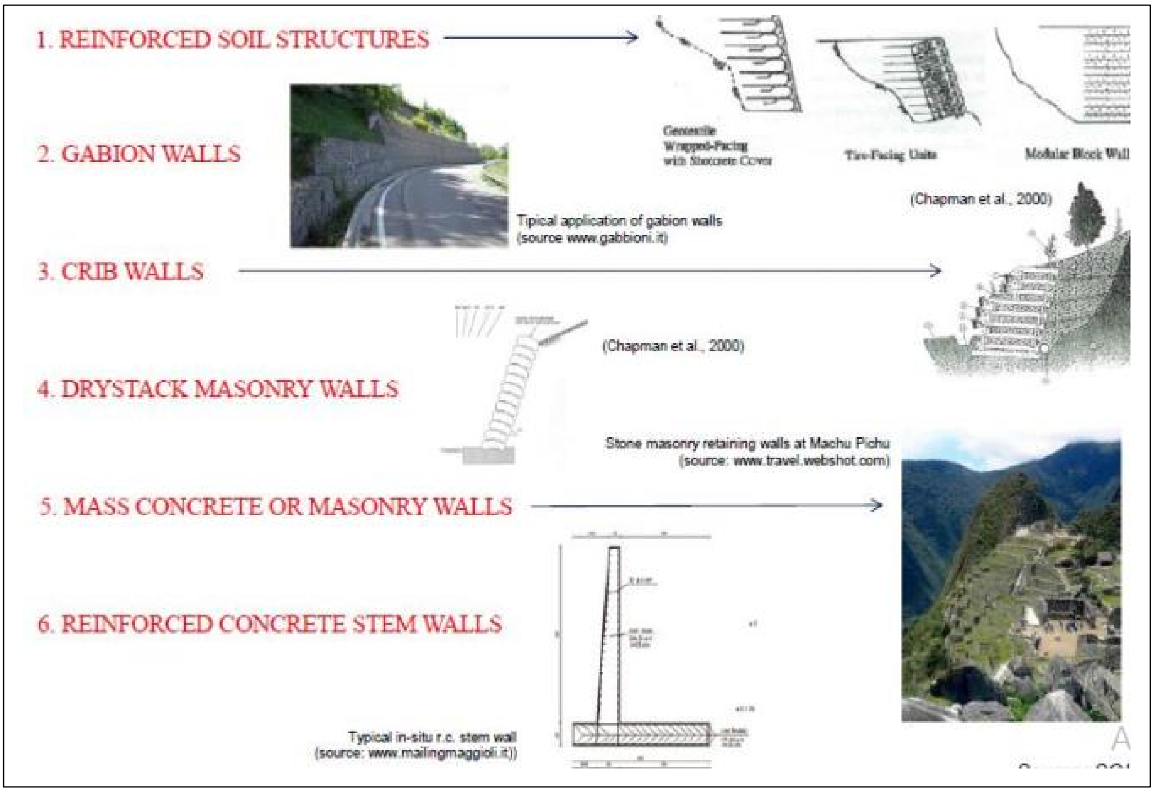

Figure 1.24. Rigid retaining walls on landslide (B. Schrefler and P. Delage, 2010).

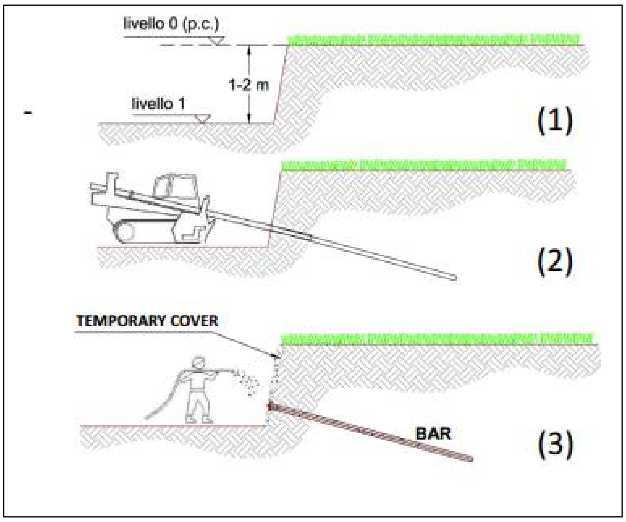

Figure 1.25. Soil nailing sequence

1) Cut of a first step 1 - 2 m high;
2) Set up of the nail (bar) in a borehole self-drilled or driven;
3) Application of temporary facing (if needs) with spritz-beton and welded steel mesh;
4) Cut of other *n* level repeating the operation 1, 2 and 3;
5) Installing of final facing and other devices.

Conclusion

Landslides as an environmental hazard is one of the principal modes of soil instability. Being a mode of soil instability, landslides could not be presented without a presentation of soils. Soils were defined as an unconsolidated natural set of solid mineral particles that result from weathering of rocks, the transportation and accumulation of the weathered material under specific conditions. Soils were seen to be constituted of a solid skeleton, a liquid part and air. The different soil formation processes were presented and some factors affecting these formation processes such as climate, parent material, time were also discussed. The variation in these factors from one region to another leads to variation in soil properties, and as such soils were be classified based on the variations in these properties. Four systems of classifying presented here were the Unified classissification system, classification based on grain-size of soil particles, classification from geological origin and the preliminary classification system based on soil type. Then, different soil failure modes were presented with a greater emphasis placed on landsliding mass movements. Landslides were discussed in details from its definitions, causes and types, the different monitoring techniques present in literature and some numerical methods used in landslide modelling were also discussed.

The chapter was concluded with a discussion on the different techniques used in stabilizing soils in general and landslides specifically.

Moving to our case study, a presentation of the methodology used in resolving the problematic of work is done in the next chapter. The different methods used in obtaining data during field investigations are discussed and the type of data obtained also presented. The chapter is concluding with the presentation of modelling techniques utilized in our analysis together with the different hypothesis used, and also, some criteria of choosing an appropriate reinforcement are also presented.

CHAPTER 2: METHODOLOGY

Introduction

Predicting the potential sliding movement of a landslide consists in determining the direction of landsliding on a slope and identifying the surface at which shear failure occurs or the slip surface. Many methods of determining direction of mass movement across a slope or the slip surface of a landslide exist in literature. Some of these methods are used in identifying just the direction of sliding movement (most remote sensing methods), while some others give more than just information on direction of movement, but also give information about the position of the surface of failure(ex. inclinometers). Describing the methodology of this work consists in describing all the different methods undertaken in identifying the direction of mass movement affecting the Charmaix Bridge, and the techniques that permitted the localisation of the potential slip surface.
This chapter sets out the various methods used in collecting various data during this study. This is then followed by discussion on the software used in this study for the modelling of the landslide, and the different modelling hypothesis used in the modelling are also discussed. The chapter is concluded with the presentation of the design procedures of different reinforcements used in landslide stabilizations.

2.1. Site recognition

The general recognition of the site involves documentary research with the aim of knowing the general characteristics of the site (geographical situation, relief, climate, geology, hydrology). This recognition is important for a better analysis of the site by observing how the different formations present on the site evolved over time.

2.2. Data collection

The aim is to obtain the data necessary to understand and solve our problem. This includes geotechnical data, geometrical, topographical, and data related to the characteristics of the materials.

2.2.1. Geotechnical data

These data are derived from the results of the different in-situ tests performed on site and the results of the monitoring campaign carried-out on the site.

The principal data here are the results of the Ménard pressuremeter test and the destructive test results which give the nature of the different soil formations present at the site, their thicknesses, and their characteristics.

Included here also is data gotten from the monitoring campaign, such as inclinometers' cumulative displacement curves for ground movement, and piezometric data for ground water level detection.

2.2.2. Topographic data

The aim here is to obtain the different profiles of the site from topographic maps which show the elevations of different points on the site in form of contour lines. These topographic maps give provide data on the positions and elevations of inclinometers and piezometers on the site.

2.3. Landslide modelling

In this study, the landslide will be analysed and modelled using the numerical analysis software SLOPE/W, which models and performs analysis using limit equilibrium methods (LEM). As mentioned in chapter one, the SLOPE/W model includes several methods like the Morgenstern-Price, Spencer, Bishop, Ordinary, Janbu9 and more methods.

The limit equilibrium method splits a potential sliding mass, defined by a trial slip surface, into vertical slices. An iterative solution is used to determine the factor by which the shear strength of all slices must be reduced such that the sliding mass is just at the point of static equilibrium (before failure occurs). This reduction factor is referred to as the factor of safety. Equilibrium can be evaluated with respect to moment or force equilibrium. Thus, SLOPE/W computes two factors of safety; one with respect to overall moment equilibrium and one with respect to horizontal force equilibrium.

Modern limit equilibrium software is making it possible to handle ever-increasing complexity within an analysis. It is now possible to deal with complex stratigraphy, highly irregular pore-water pressure conditions, and various linear and nonlinear shear strength models, almost any kind of slip surface shape, concentrated loads, and structural reinforcement. Limit equilibrium formulations based on the method of slices are also being applied more and more to the stability analysis of structures such as tie-back walls, nail or fabric reinforced slopes, and even the sliding stability of structures subjected to high horizontal loading arising, for example, from ice flows. Computer assisted graphical viewing of data used in the calculations makes it possible to look beyond the factor of safety. For example, graphically viewing all the detailed forces on each slice in the potential sliding mass (Fig.2.1), or viewing the distribution of a variety of parameters along the slip surface, helps greatly to understand the details of the technique.

SLOPE/W has many tools for inspecting the input data and evaluating the results –tools like allowing you to graph a list of different variables along the slip surface or to display the detail forces on each slice, for example. These types of tools are vitally important to judging and being confident in the results.

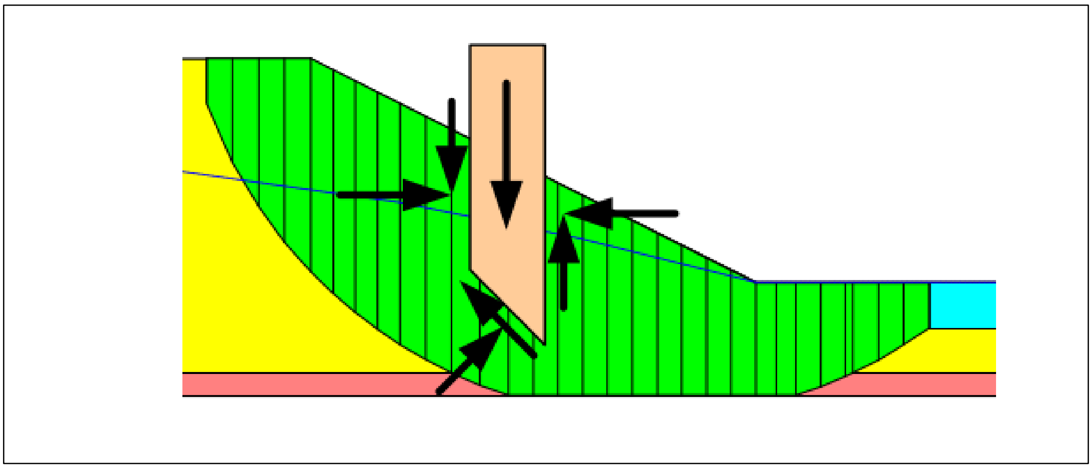

Figure 2.1. Slice discretization and slice forces in a sliding mass.

SLOPE/W is one component in a complete suite of geotechnical products called GeoStudio. One of the powerful features of this integrated approach is that it opens the door to types of analysis of a much wider and more complex spectrum of problems, including the use of finite element computed pore-water pressures and stresses in a stability analysis. Not only does an integrated approach widen the analysis possibilities, it can help overcome some limitations of the purely limit equilibrium formulations. Although, it is not necessary to use this advanced feature as SLOPE/W can be used as an individual product, there is certainly an increase in the capability of the program by using it as one component of a complete suite of geotechnical software programs.

The methodological approach in modelling a problem in SLOPE/W includes five components.

i. Geometry – description of the stratigraphy and shapes of potential slip surfaces
ii. Soil strength – Parameters used to describe the soil
iii. Pore-water pressure – means of defining the pore water pressure conditions
iv. Reinforcement or soil-structure interaction – fabric, nails, anchors, walls
v. Imposed loading – surcharges or dynamic earthquake loads

Many different solution techniques for the method of slices have been developed over the years. Basically, all are very similar. The differences between the methods are depending on what equations of statics are included and satisfied and which interslice forces are included and what is the assumed relationship between the interslice shear and normal forces.

In this work, analysis will be done using the Morgenstern-Price method, since it satisfies both moment and force equilibrium. Tables 2.1. shows the different limit equilibrium methods and the differences between them.

Table 2.1. Interslice force characteristics and relationships

Method	Interslice Normal (E)	Interslice Shear (X)	Inclination X/E Resultant and X-E relationship
Ordinary or Fellenius	no	no	No interslice force
Bishop's Simplified	yes	no	Horizontal
Janbu's Simplified	yes	no	Horizontal
Spencer	yes	Yes	Constant
Morgenstern-Price	yes	Yes	Variable; user function
Corps of Engineers-1	yes	Yes	Inclination of line from crest
Corps of Engineers-2	yes	Yes	Inclination of ground surface at top of slice
Lowe-Karafiath	yes	Yes	Ground surface and slice base inclination
Janbu Generalised	yes	Yes	Applied line of thrust and moment equilibrium of slice
Sarma-vertical slices	yes	Yes	X = C + E tanØ

2.3.1. Geometry modelling

In our model, the determination of different formation thicknesses for the local study area will be done by utilising available drilling data gotten from earlier investigations carried out in the area by GEOTECHNIQUE SAS. However, most of the described drillings were not carried out within the boundaries of the study area. Therefore, some formations or layers thicknesses will be estimated, assuming a uniform cover across the entire slope. As additional source of information, data from seismic hazard maps will be utilised in defining the seismic force coefficient in the region.

The final subsurface model will be simplified for subsequent modelling in Slope/W in which only three material layers are represented (figure 2.2), i.e. a compact shale layer acting as a bedrock, a weathered shale and moraine, which is the overlying material. The soil layers are represented by the coordinates below. The slope model had a gradient of 3-horizontal: 1-vertical (3:1).

a) Compact shale layer

A (350; 1190), B (417; 1177), C (500; 1149), D (632; 1080), E (690; 1090), F (690; 1050).

b) Moraine

A (350; 1190), M (417; 1190), N (473; 1164), O (660; 1100), P (690; 1100).

c) Decayed shale

Q (430; 1160), R (510; 1153), S (580; 1090), T (640; 1085)

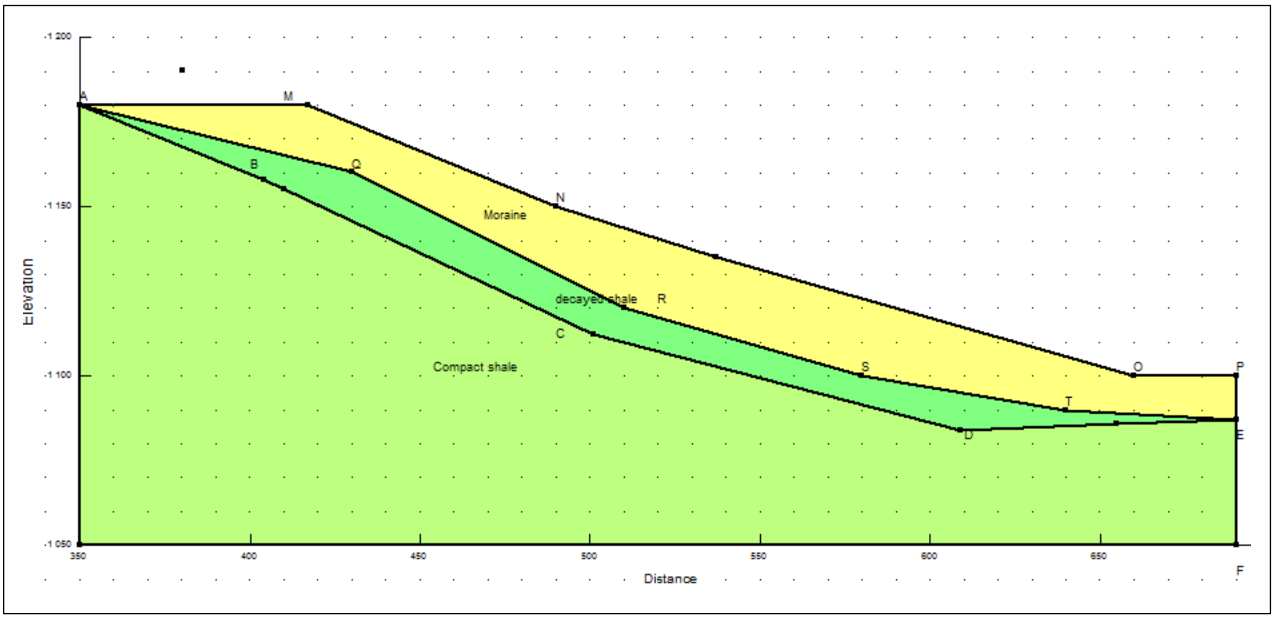

Figure 2.2. Geometry model of slope section.

2.3.2. Soil strength parameters determination

Modelling a slope in Slope/W requires determination of material parameters, including effective angle of internal friction, effective cohesion, saturated and unsaturated unit weights, and bulk densities. Since laboratory tests data were not available for this study, parameter values will be estimated from literature sources and correlative equations.

Angle of friction and cohesion will be gotten from Cassan equations which uses limit pressure gotten with the Ménard test to compute friction angle and cohesion. The other parameters shall be gotten from literature sources.

2.3.3. Groundwater level

Hydrological monitoring data are utilised to create scenarios of ground water table positions for subsequent modelling of slope stability in Slope/W. Available ground water data from each sensor location is analysed and the piezometric line determined. Determination of ground water table will primarily exploit piezometer data, which provide relative water table height.

The maximum piezometric head will be used in this analysis due to the negative effect of the water head on the slope stability. So, the factor of safety gotten from the analysis was the worst possible value. The geometry of the piezometric line used in the model is represented in fig.2.3.

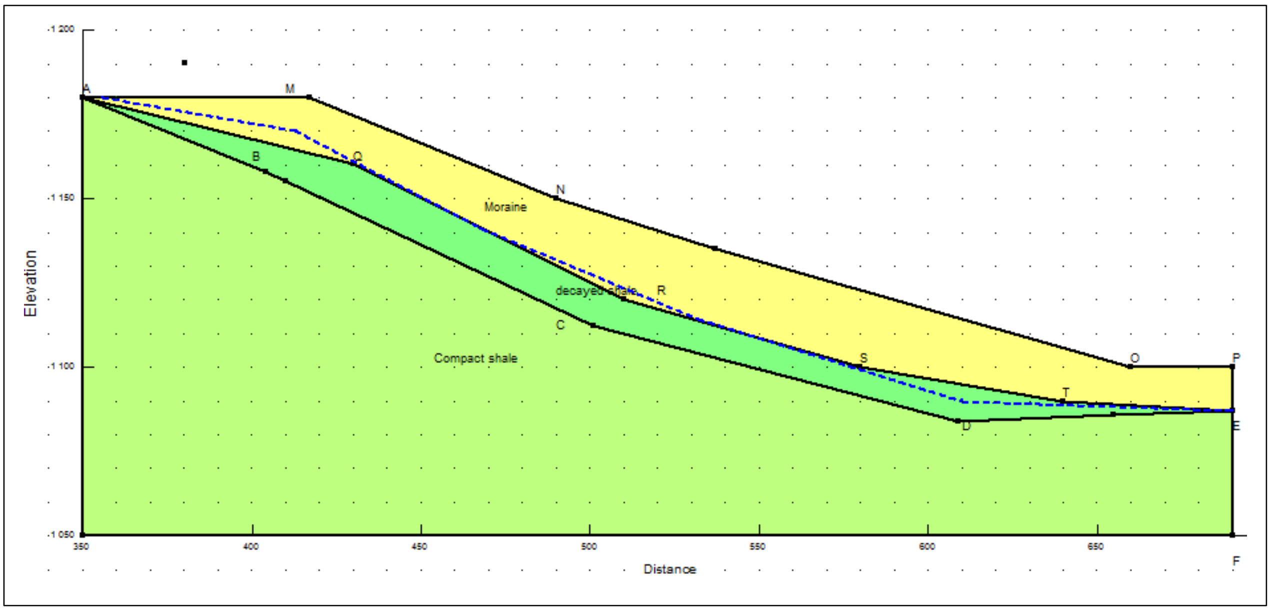

Figure 2.3. Geometry of piezometric line.

2.3.4. Methods of analysis of slope model

In analysing the slope model, a total of three situations will be analysed.

i. A back analysis will be performed in wet conditions so as to get residual values of shear strength parameters of the slip surface, and this will be done under static conditions.

ii. A second analysis will be performed under pseudo-static conditions, taking in consideration the seismic load coefficient.

iii. Reinforcement anchors models will be designed and analysed with the aim of observing the effects of reinforcements on the slope stability. Given that the angle of inclination of anchors to the horizontal has a considerable effect on anchor efficiency, analysis will be done at different inclination angles, i.e. at 25°, 35° and 45°, so that an optimal angle can be chosen.

2.3.5. Back-analysis for residual strength properties

The slope instability in this case is due to a continuous reactivation of a landslide. Hence, the behaviour of the slide is governed by the residual shear strength properties. Residual strength properties can be obtained by either conducting the traditional back analysis or by laboratory testing. The reliability of the back analysis is proportional to the confidence with which the pore water pressure and the location of the location of the slip surface in the slope are known (Bromhead and

Dixon, 1986). Because of the availability of instrumentation data in this case, the defined slip surface and the ground water elevations reported are considered reliable. In this study, a back analysis is performed to determine the residual friction angle (φ'r) along the slip surface. The cohesion value is set to zero and the friction angle value is varied to obtain a factor of safety of unity (Ducan and Wright, 2005).

2.4. Selection and preliminary design of slope reinforcement

2.4.1. Criteria in selecting an appropriate reinforcement

There are many different types of earth retaining structure. The first stage in the design process is therefore to assess the appropriateness of the available types of structure for the given application. Preliminary design may then be carried out on a number of different types of retaining structure, to assess their viability, and finally detailed design calculations will be undertaken.

The available options and constraints on a reinforcing a slope will often make preliminary design a complex proposal. Initially, there will be a need to support soil, or a structural load or any adjacent structure. There will be a desired geometry for the completed structure, but, in addition, constraints due to subsoil and groundwater conditions, available construction methods, and local experience of those methods will also play their part in the choice of the retaining structure. The factors which may influence the choice of structure are; height of slope to be supported, type of retained soil, type of foundation soil, ground water regime, adjacent structure and the available space for construction.

Three criteria will be used in selecting the appropriate reinforcement to be used in this work, these are; (i) functionality (ii) constructionality (iii) cost.

Analysing all these factors affecting the choice of an appropriate reinforcement, given the remote nature of the site of study, the most practical solution which can be used in reinforcing the slope from instability movements would be the use of ground anchors, due to their relatively cheap nature in comparison to retaining wall structures, and the fact that they can be used in any environment, even adjacent to other structures. Soil nailing is a cost-effective alternative to conventional retaining wall structures for most soils. However they are not practical in loose materials or plastic soils (CATRANS, 2015). Also, compared with rock bolts and soil nailing, ground anchors have a relatively large resistance to sliding force and are therefore used to stabilize relatively large-scale slope failures (JICA, 2009).

2.4.2. Preliminary design of ground anchors as a slope reinforcement technique

A grouted ground anchor is a structural element installed in soil or rock that is used to transmit an applied tensile load into the ground, and it may be pre-stressed. The basic components of a grouted ground anchor include the anchorage, free stressing (unbonded) length and bond length as shown in figure 2.4.

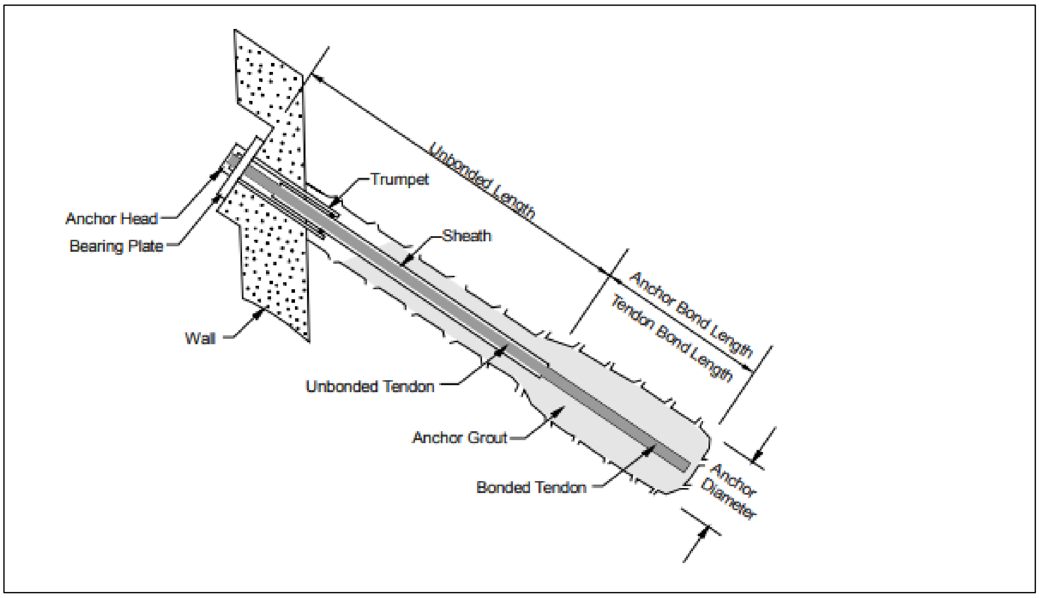

Figure 2.4. Components of a ground anchor (FHWA, 1999).

Figure 2.4 shows the design flowchart for ground anchors. Important considerations for ground anchors are the bearing capacity of the ground under the bearing plate and the bond strength between the anchor grout and rock at the attachment point. In planning ground anchors, a bond strength test at the attachment is to be carried out, but in this study bond strength tests data were not available so data on bond strength will be gotten from literature sources.

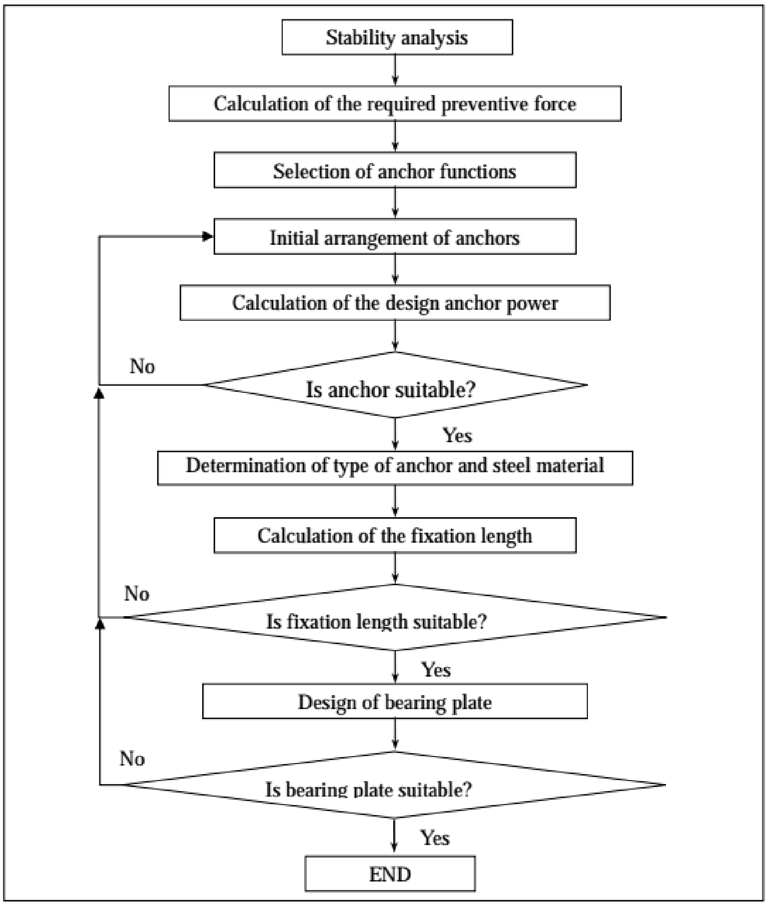

Figure 2.5. Design flowchart for ground anchors (JICA, 2009).

a) Stability analysis and calculation of required preventive force

The required preventive force of the anchors shall be obtained from the following equation 2.1, as schematically shown in Figure 2.6.

$$P = \frac{(PFs - Fs) * \Sigma T}{\cos(\alpha+\theta) + \sin(\alpha+\theta) * tan\varphi} \qquad \textbf{(2.1)}$$

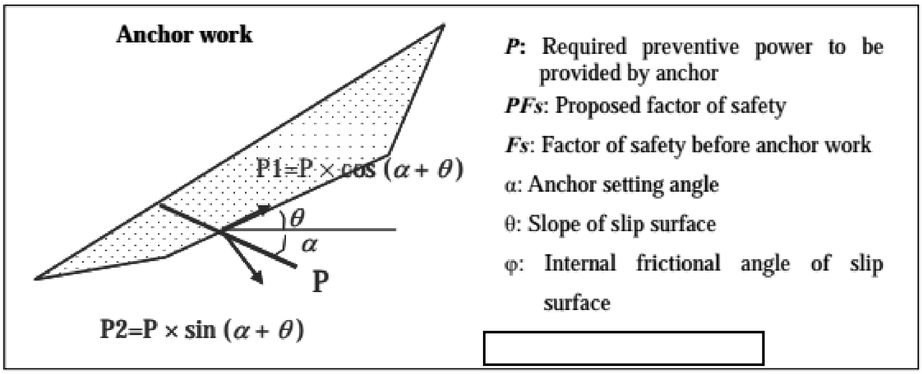

Figure 2.6. Schematic Diagram of Effectiveness of Anchor Works.

b) Selection of anchor functions

Anchors are installed to achieve the following two objectives.

i. Increase the resisting power against shear force by applying stress normal to the sliding surface (clamping effect, shear resistance, P2, shown in Figure 2.6), and

ii. Decrease the sliding force of a landslide by using steel members as anchors (straining effect, tensile resistance, P1, shown in Figure 2.6).

The effect of anchor should generally be selected, in terms of geometry of the shape of sliding surface taking some considerations into account.

- Select the clamping effect when sliding surface is gentle and deep
- Select the straining effect when sliding surface is steep and shallow
- Select both effects when sliding surface is at the middle conditions

c) Calculation of pull-out resistance or anchor power

The design anchor power (Td) is calculated by using equation 2.2.

$$\mathrm{Td} = \frac{P * B}{(\sin(\alpha+\theta)tan\varphi + \cos(\alpha+\theta))N} \quad \textbf{(2.2)}$$

Where,

P = Required preventive power

α = Anchor setting angle (the angle to a perpendicular axis)

β = Angle of slope of the sliding surface

φ = Internal frictional angle of sliding surface

B = Interval between anchors in horizontal direction

N = Number of anchors set in longitudinal direction

d) Determination of type of anchor and steel material

The type of anchor will be determined by comparing the tension strength of steel material with the skin frictional resistance between the ground and the grout as well as the allowable adhesive stress between the tendon and the grout.

e) Determination of fixation length

Fixation length should be 3 to 10 meters. If the fixation length is longer than the upper limit, the allowable anchor force will be largely decreased. The free length of an anchor should be determined so that the coupling portion of anchors is secured firmly to rigid bedrock in an unmovable stratum deeper than sliding surface. The fixed length is determined using equation 2.3.

$$la = \frac{Td}{3.14 * Da * qu} \quad \textbf{(2.3)}$$

Where,

La = Required length of contact between the soil and the grout

Td = Design pull-out resistance of each anchor

DA = Diameter of the anchor

qu = Ultimate bond stress per unit length of steel bar (Figure 2.7)

Rock		Cohesive Soil		Cohesionless Soil	
Rock Type	Average *Ultimate* Bond Stress (psi)	Anchor Type	Average *Ultimate* Bond Stress (psi)	Anchor Type	Average *Ultimate* Bond Stress (psi)
Granite and basalt	250–450	Gravity-grouted anchors (straight shaft)	4.4–10	Gravity-grouted anchors (straight shaft)	10–20
Dolomitic limestone	200–300	Pressure-grouted anchors (straight shaft)		Pressure-grouted anchors (straight shaft)	
Soft limestone	150–200	• Soft silty clay	4.4–10	• Fine-med. sand, med. dense – dense	12–55
Slates and hard shales	120–200	• Silty clay	4.4–10	• Med.–coarse sand (w/gravel), med. dense	15–95
Soft shales	30–120	• Stiff clay, med. to high plasticity	4.4–15	• Med.–coarse sand (w/gravel), dense–very dense	36–140
Sandstones	120–250	• Very stiff clay, med. to high plasticity	10–25		
Weathered sandstones	100–120	• Stiff clay, med. plasticity	15–36	• Silty sands	25–60
Chalk	30–160	• Very stiff clay, med. plasticity	20–50	• Dense glacial till	45–75
Weathered marl	22–36	• Very stiff sandy silt, med. plasticity	40–55	• Sandy gravel, med. dense–dense	30–200
Concrete	200–400			• Sandy gravel, dense–very dense	40–200

Note: Actual values for pressure-grouted anchors depend on the ability to develop pressures in each soil type. *after Sabatini et al., 1999

Figure 2.7. Recommended ultimate bond stress per unit length of steel bar (Sabatini et al., 1997).

f) Design of bearing plates

Plates or cross-shaped blocks set on the surface of the ground are used as pressure bearing plates. The most appropriate pressure bearing plate is selected in consideration of specifications, operational efficiency, cost-effectiveness, maintenance, landscape, etc.

Conclusion

This chapter aimed at defining the methodological approach which was used in this study in obtaining data and the approach utilised in exploiting the data. The data here included all geotechnical and topographic surveys which were conducted, the apparatus used in conducting them, and the type of data obtained from these surveys. The geotechnical survey included tests such as the pressuremeter test and the destructive tests. The limit equilibrium method of analysis was also presented in details with greater attention on Slope/W as a limit equilibrium analysis software. The different design steps in Slope/W were also discussed and the different hypothesis utilised in creating the slope model presented.

Several criteria used in selecting an appropriate slope reinforcement were presented and ground anchors were seen to be the best possible solution in reinforcing the case study, given the nature of the site and neighbouring structure. The preliminary design procedure for ground anchors as a slope reinforcement technique was also seen.

In the chapter that follows, the results of both the field work and software models shall be presented with an interpretation given to the different results. Then a solution shall be proposed if need presents itself, for the slope remediation.

CHAPTER 3: RESULTS AND INTERPRETATION

Introduction

The Charmaix Bridge in S.E France has been permanently subjected to landslide movements affecting the two mountains linked by the bridge, due to tectonics activities taking place in the region. The need to stabilize these mountains called up on geotechnical campaigns in the affected area.
In this chapter, all the results gotten during these campaigns are presented and an interpretation given to them. The chapter begins with a presentation of the site and a presentation of data gotten from field work surveys. These data include topographic data, site subsurface formations data, which were gotten by interpreting the pressuremeter tests curves. The different subsurface are then characterized by their geotechnical parameters. Slope monitoring data such as inclinometer displacement curves and water table measurements are presented, with an interpretation given to each of them.

The chapter ends with the presentation of the different slope models analyses and reinforcement model analyses with their degrees of stability represented by their FoS. The results of these analysis are then interpreted and stabilization strategies proposed.

3.1. Site presentation

3.1.1. Localisation

The case study of the Charmaix Bridge is situated in the Rhone-Alpes region (S.E France), in the municipality of Modane 73500 (Fig.3.1). In the following, an introduction to the study area Modane 73500, at the Rhone-Alpes region, France, is presented. Therein, geological and climatic properties are described.
Modane is a French commune located in the Savoie department, in the Auvergne-Rhône-Alpes region. It is located in the Maurienne valley and belongs to the Vanoise National Park. Modane is located in the Alps in the department of Savoie between the Vanoise massif to the north and the Mont-Cenis massif and the Cerces massif to the south. Crossed by the Arc, it extends to the gateway to the Haute-Maurienne.
Some images of the site are shown in figures 3.2 and 3.3.

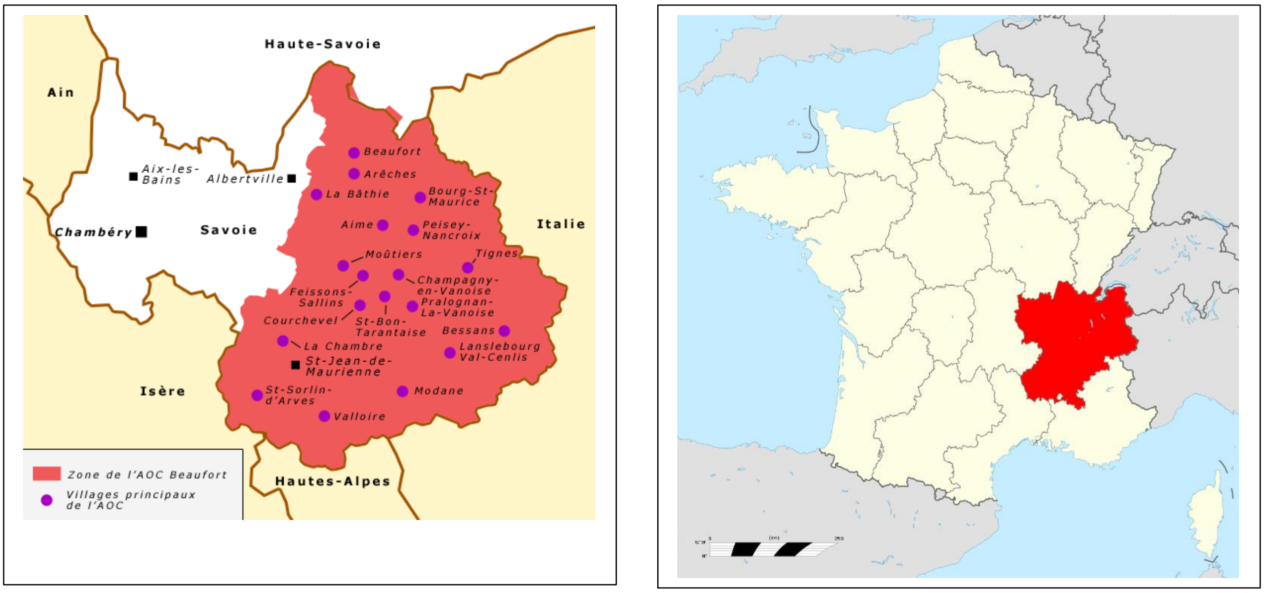

Figure 3.1. Geographical maps of the Rhône-Alpes region, France.

Figure 3.2. Charmaix Bridge, lateral view.

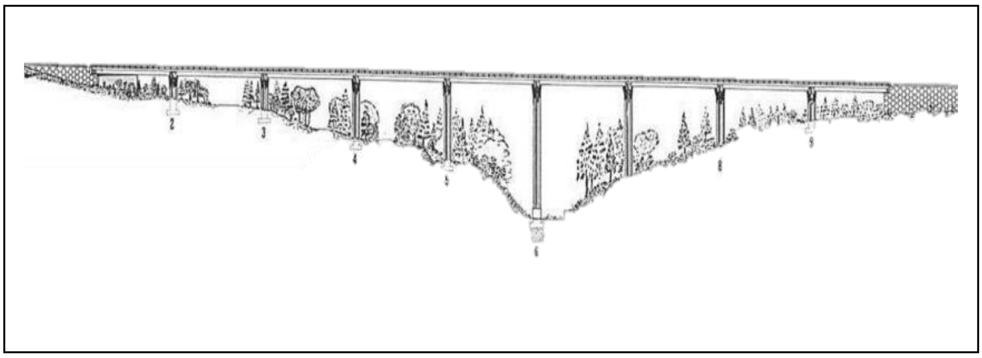

Figure 3.3. Profil of Charmaix bridge (retour d'expérience sur les fondations d'ouvrage d'art).

3.1.2. Climate

Modane has a humid continental climate with no dry season according to Köppen-Geiger classification. The municipality of Modane registers high rainfall even in the driest month there is a lot of rain. Over the year, the average temperature in Modane is 8.9°C and the average rainfall is 934.9 mm.

Figure 3.4 presents the variation of temperature over the year at Modane.

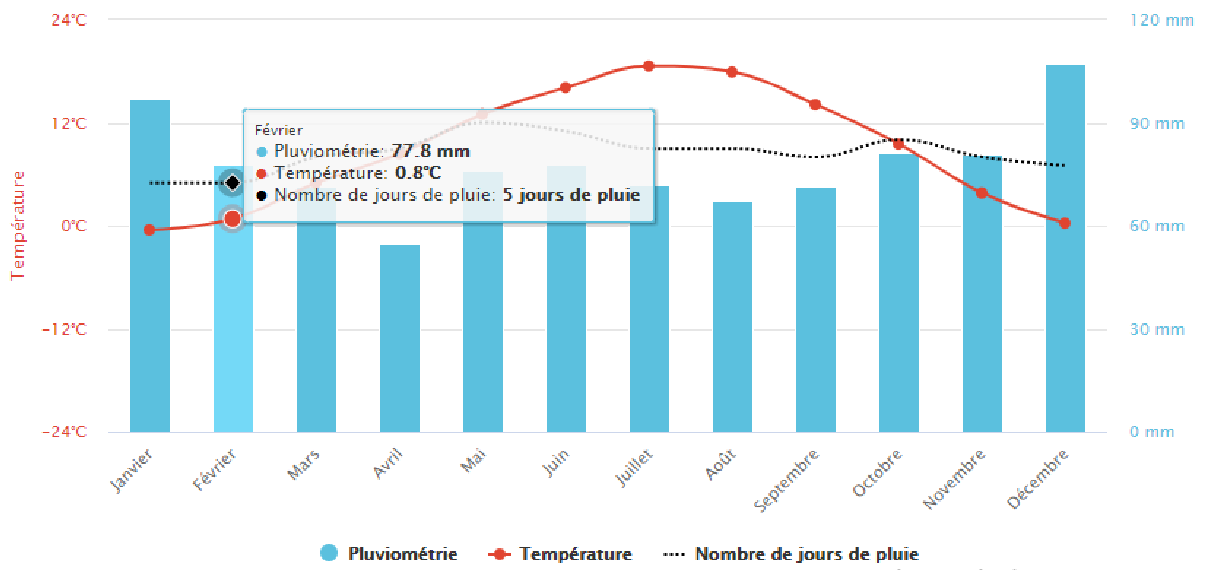

Figure 3.4. Climatic diagram of Modane.

3.1.3. Geology

The Modane municipality has complex geology: the Rhône valley, with its recent sedimentary filling (Tertiary and Quaternary), separates two large formations: the Massif Central and the Northern Alps. The primary rock is found on the eastern edge of the Massif Central, cut by the great collapses of the Forez plain and the Saint-Etienne coal basin. To the east, the Alpine massifs are criss-crossed by deep valleys and bordered by the limestone Pre-Alps. Their fragmentation, well as the numerous faults that run through them, testify to the tectonic upheavals associated with thrusting. This geological complexity gives the region a mineral wealth, with a wide variety of resource such as: coal, metals, uranium, massive rocks, alluvium (Auvergne R., 2016).

3.1.4. Demography

In 2018, the Modane municipality recorded a population of 3077 inhabitants. Figure 3.5 show a histogram of population growth in Modane.

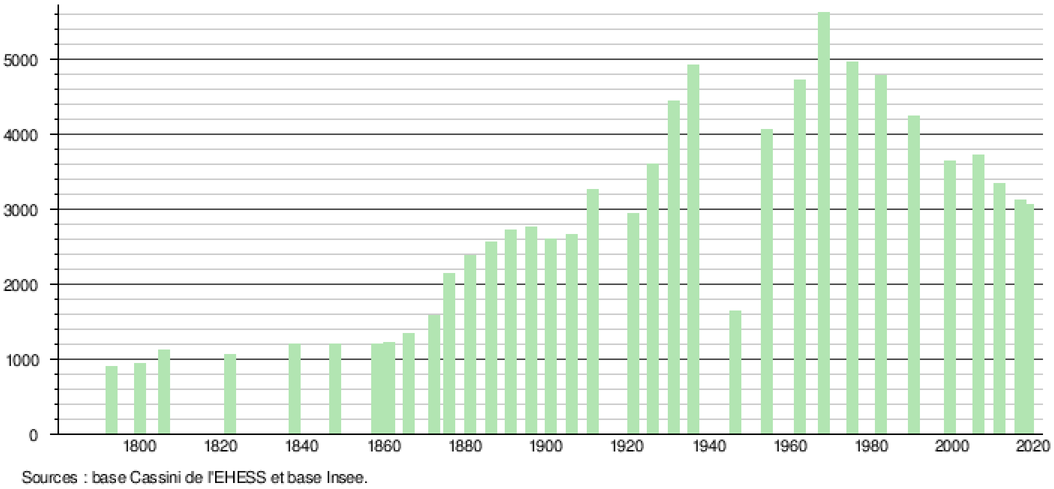

Figure 3.5. Population growth histogram at Modane.

3.2.Data presentation

3.2.1. Topographic data

Information on the topography of the site was gotten from topographic maps of the region (see appendix 2). From these maps the altitudes, longitudinal and latitudinal coordinates of important points on the site could be gotten. Table 3.1 presents the coordinates of the different inclinometers and piezometers which were used during the site campaigns.

Table 3.1. Shows the coordinates of the inclinometers and piezometers (source:SAGE ENS)

Survey	Date of perfora-tion	Date of bore-hole closure	Coordinates			Axes orienta-tions		depth
			X	Y	Z	A	B	m
Inc601	10/08/16	17/08/16	986417.3	6460873.324	1190.0	N74	N157	35
Pz601	29/08/16	-	986417.3	6460873.324	1190.0	-	-	18
Inc602	25/08/16	26/08/16	986473.7	6460849.826	1164.1	N198	N108	24.6
Pz602	31/08/16	-	986473.7	6460849.826	1164.1	-	-	12.1
Inc603	24/08/16	26/08/16	986519.8	6460837.536	1160.4	N185	N95	25.5
Pz603	30/08/16	-	986519.8	6460837.536	1160.4	-	-	11.2
Inc604	13/09/16	16/09/16	986587.8	6460846.347	1142.1	N40	N130	25.8
Pz604	07/09/16	-	986587.8	6460846.347	1142.1	-	-	25.3
Inc605	16/08/16	02/0/16	986632.5	6460853.69	1122.0	N0	N90	40.0
Pz605	27/09/16	-	986632.5	6460853.69	1122.0	-	-	21.3
Inc606	06/10/16	06/10/16	986916.1	6460931.525	1167.8	XX	XX	XX
Pz606	12/10/16	-	986916.1	6460931.525	1167.8	-	-	25.0
Inc607	29/09/16	29/09/16	986833.1	6460874.25	650.00	XX	XX	XX
Pz607	30/09/16	-	986833.1	6460874.25	650.00	-	-	XX
Inc608	23/09/16	26/09/16	986426.9	6460931.200	1141.1	XX	XX	53.5
Pz608	05/09/16	-	986426.9	6460931.200	1141.1	-	-	26.0
Pz609	22/08/16	-	986486.6	6460926.795	1141.1	-	-	30.0

3.2.2. Geotechnical data

3.2.2.1. Subsurface geological constitution

In this study, subsurface reconnaissance was carried out by a detail analysis of the various boreholes drilled for the pressuremetric and destructive surveys. This was done by a detailed analysis of the different geological formations crossed during boreholes drilling.

Initially, a total of seven boreholes were drilled, representing boreholes SPA, SPB, SPC, SPD, SPG, SDE, and SDF.

SPA was drilled until a depth of 15m below. From 0 to 8.7m layers of brown to light brown moraine could be noticed, but a layer of discontinuity was found from 2.6m to 3.3m which contained grey moraine instead of brown. Then a layer of decayed greyish black shale was found from 8.7m to 10m, and drilling was stopped in a layer of shale in the early stages of decay from 10m to 15m. See table 3.2 for geotechnical characteristics.

SPB was found to constitute only of shale, this was for a depth of 25m from natural ground surface level.

The first approximately 6.50m of SPC and SPD constituted of moraine composed of blocks of shale and decayed shale. A layer of gravelly sand was found from 6.50m to 14.60m. At SPD, averagely compact shale was noticed from 14.60m to 20.50m, while at borehole SPC decayed shale was noticed from 14.60m to 22.0m and grey compact shale from 22.0m to 25.06m. See table 3.3.

At SDE and SDF similar results were gotten as those at SPA, with the difference that grey blocks of were noticed from 2.0m to 3.80m depth.

At SPG, grey moraine was found from ground surface to a depth of 2.09m. Depth 2.09m to 21.90m constituted layers of gravelly sand with some cobbles, then clayey sand from 21.09m to 25.0m. See table 3.4.

Table 3.2. Formations at SPA

Parameter	Moraine + shale passages	Gravely sand	Decayed shale	Compact shale
Menard modulus (MPa)	11.9	10.5	20.8	324.5
Limit pressure, Pl (MPa)	1.15	1.13	2.41	5.00

Table 3.3. Formations at SPC and SPD.

Parameter	Greyish moraine	Sand, gravel and grey cobbles	Clayey sand
Menard modulus (MPa)	7.0	44.6	-
Limit pressure, Pl (MPa)	0.60	2.58	-

Table 3.4. Formations at SPG.

Parameter	Moraine + shale passages	Gravely sand	Decayed shale	Compact shale
Menard modulus (MPa)	11.9	10.5	20.8	324.5
Limit pressure, Pl (MPa)	1.15	1.13	2.41	5.00

3.2.2.2 Slope movement data

a) Inclinometric data and interpretation

Slope movement data from inclinometer measurements carried out by SAGE (SOCIETE ALPINE DE GEOTECHNIQUE) included 08 manual inclinometers with an inclinometer chain for continuous monitoring. Displacement data from 05 inclinometers were available during this work.

The monitored data presented in this work is that which was monitored from August 2016. In addition to the already existing seven inclinometers, inclinometer Inc608 was installed for monitoring movements on the lower flank of the mountain. Periodic measurements were carried out from August 2016 with standard inclinometers. To obtain cumulative displacement profiles, data readings were added from the bottom up, assuming fixed casing tip. Measurements were carried out along two axes, axes 1 and 2.

Inclinometer inc601 is located in the western part of the study area and is approximately 246m away from inclinometer inc605 which is the furthest inclinometer eastward among the 05 inclinometers. Therefore displacement monitoring was carried over a distance of about 246m.With a total penetration depth of 35.5m, inc601 could be fixed in a stable rock, i.e. stable shale found at 20m depth. Data were collected on this inclinometer on September 6, 12, 16, 23, and 30, 2016. A final recording was done on October 12 and showed a maximum displacement of approximately 8.0mm downslope of the inclinometer (direction A) at a depth of 5m, while direction B showed a cumulative displacement of 3.0mm.

Inc602 had a depth of 24.64m and was fixed in stable rock. Records also show displacements along both axes. Displacements along direction-A, downslope, show large displacements of about 30.0mm at inclinometer tip.

At inc603, the automatic measurements indicate that the measurement base located between 7.75 and 9.25m depth is effectively anchored. From 06/10/2016 to 12/10/2016 the average global displacement rate was constant and showed an average displacement of 0.25mm/day at the failure surface of inc603.

From data collected during the inclinometers surveys, it was noticed that the direction in which greater movements were observed was downslope, which is A-axis direction. Because of this, the inclinometer displacement data in this direction were analysed in greater details since it was the main direction of slope movement. Below is presented a record of displacements observed at different inclinometers following the downslope direction.

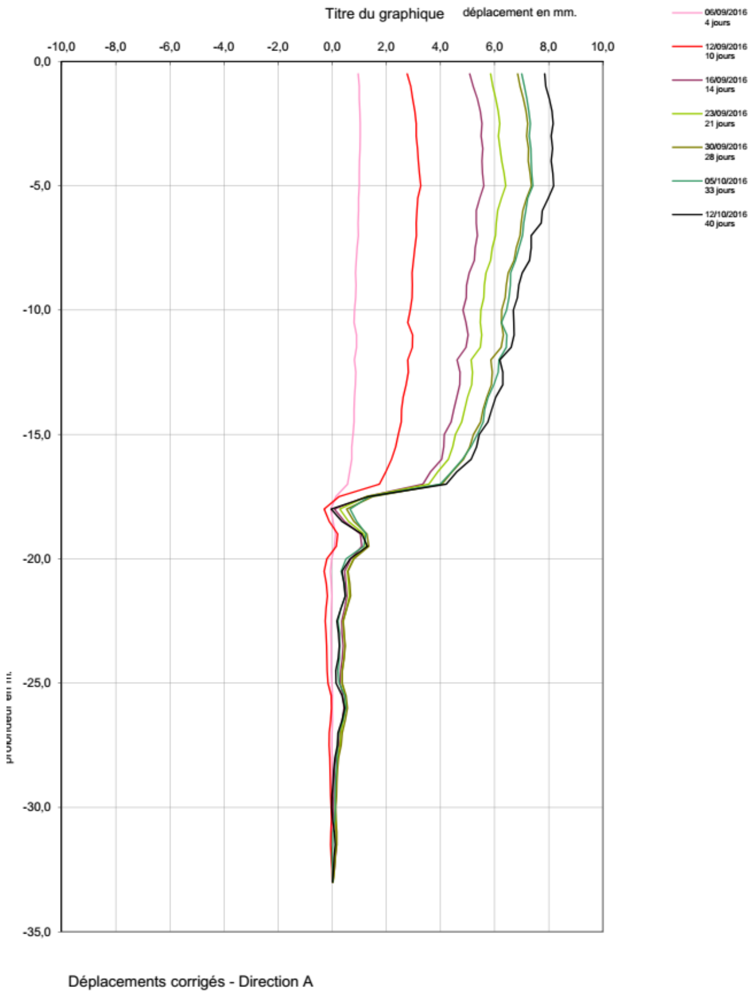

Figure 3.6. Cumulative displacements at Inc601.

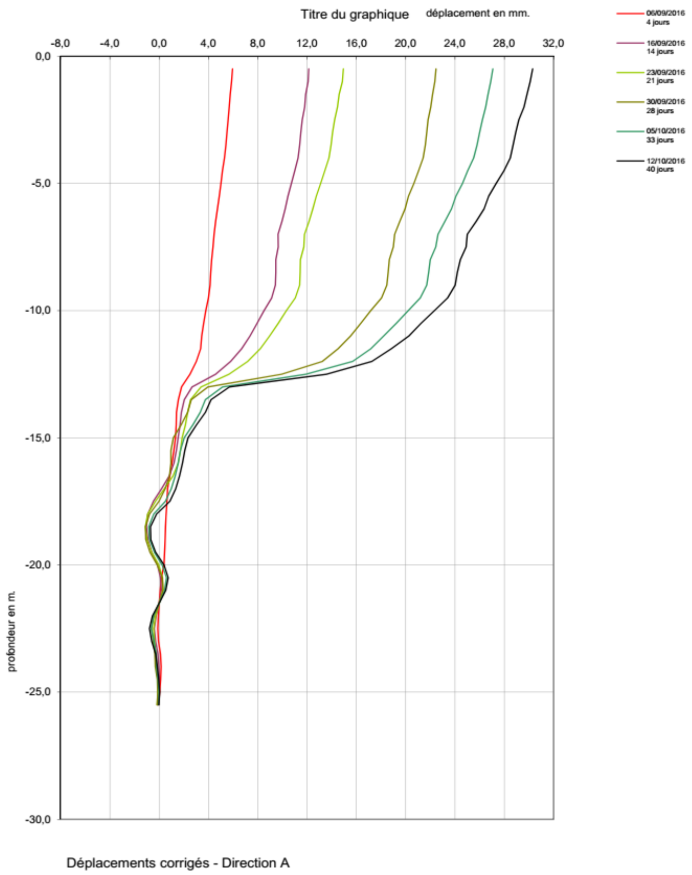

Figure 3.7. Cumulative displacements at Inc602.

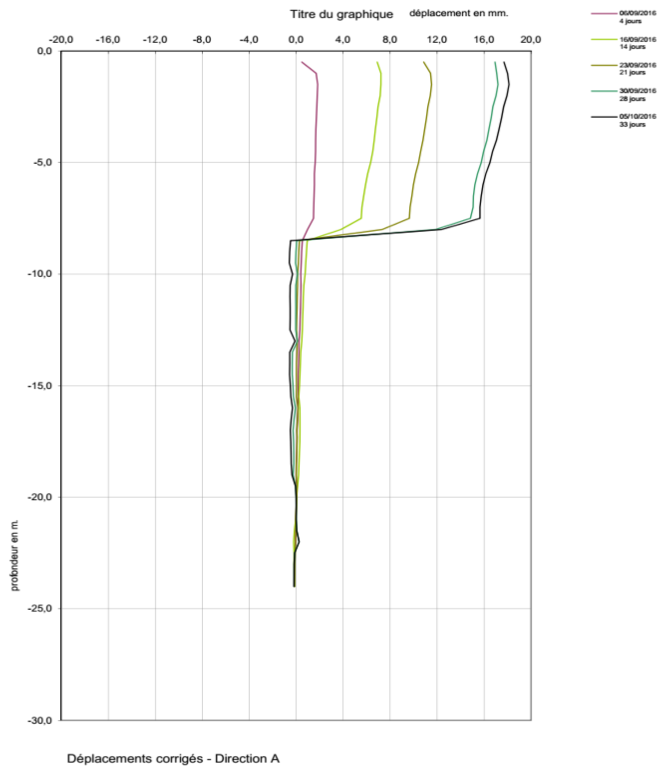

Figure 3.8. Cumulative displacements at Inc603.

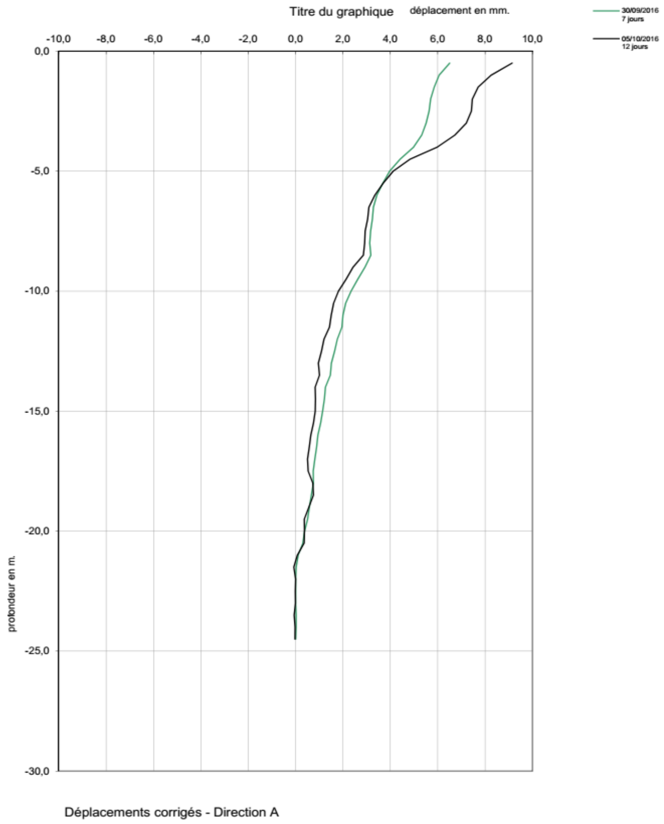

Figure 3.9. Cumulative displacements at Inc604.

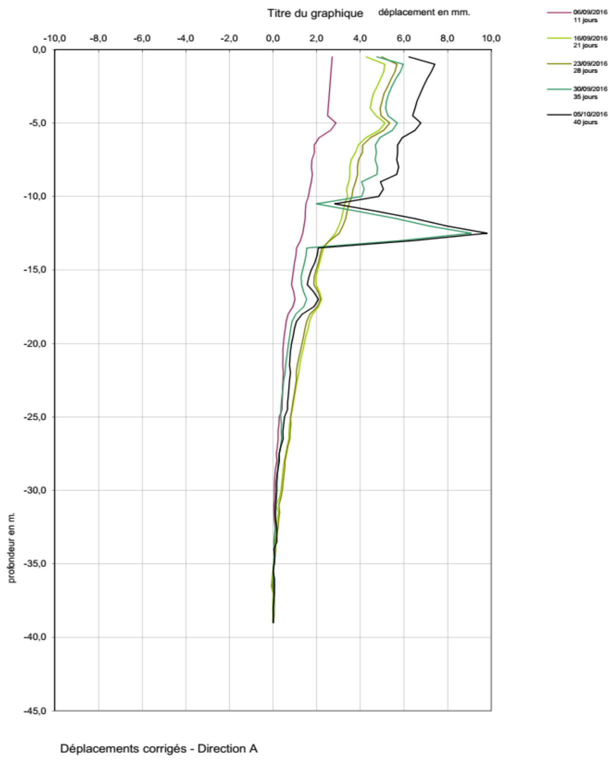

Figure 3.10. Cumulative displacements at Inc605.

From these figures (fig.3.6 to fig.3.10) , it was found that the horizontal displacements increase from bottom to the surface, indicating that the whole monitored area of the landslide was moving during the period of monitoring. Furthermore, there was a stable area with negligible displacement at the bottom of each borehole. The existence of these stable areas indicates that the boreholes were drilled into stable zones within the bedrock.

Inc601 and Inc602 were located at the upper part of the slope, the displacement profiles in these two boreholes are shown in figures 3.6 and 3.7. As time went, the soil mass moved slowly at the borehole bottom, and the displacements began to change significantly at depths of 17 and 13m for Inc601 and Inc602, respectively. These shift points were found near stable bedrock surface, which means that the potential sliding surface may pass through this surface.

The displacement profiles of the Inc603 and Inc604 are presented in figures 3.8 and 3.9. Both inclinometers were located at the middle area of the slope. The shift points of displacements for Inc603 and Inc604 were found at depths of 7.5 and 19m respectively.

Inc605 was located at the bottom of the slope and the corresponding displacement profile is shown in figure.3.10. In general, the displacement of the soil mass at Inc605 were smaller compared to other boreholes, but the shift point of displacement was found at 13m.

In summary, the movements at the crest and middle area of the slope were found to be greater than those at the toe. Since the shift points of displacements were found to be near a stable surface, it can be concluded that this stable surface is the most potential sliding surface within the landslide. Figure 3.11. shows the positions of the most potential slip surface relative to the slope surface and inclinometer depths.

b) Piezometric data and interpretation

Piezometric measurements presented here were taken from the 08th September 2016 to the 05th October 2016. The objective here was to identify the water level on the slope at different periods of time. As presented in table 3.4, the water table height varied at different days, showing an increasing trend in some days and a decreasing trend in others. This could be due to the varying intensity of rainfall at different periods of measurement.

At the time thesis was written, measurements from some piezometers such as pz605, pz606 and pz607 were not yet updated. Table 3.5 presents the different water heads recorded.

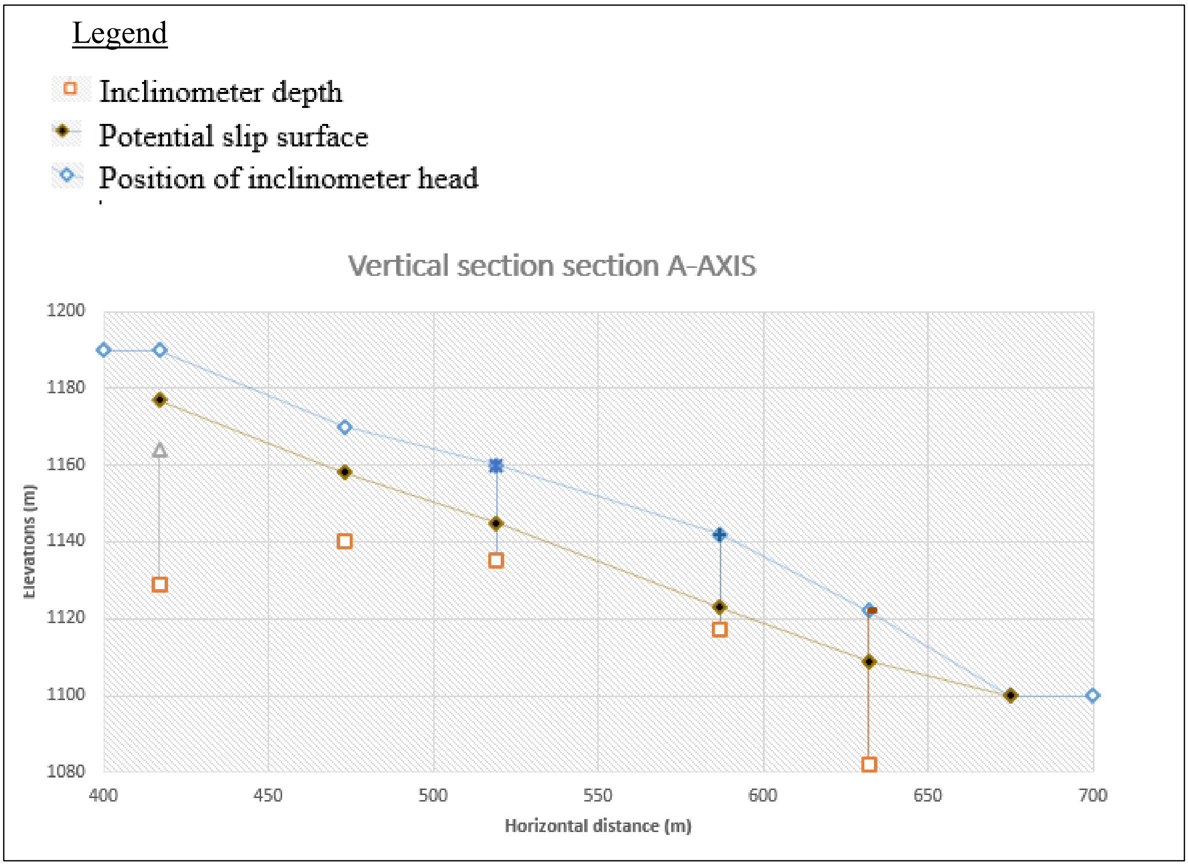

Figure 3.11. Inclinometer positions and position of potential sliding surface.

Table 3.5. Piezometric measurements data.

Piezometers (Depth/meters)									
Inclinome-ter	**601**	**602**	**603**	**604**	**605**	**606**	**607**	**608**	**609**
Depth of hole	18	12	11	17.5	21	xx	25	16.9	23.5
08/09/2016	6.6	-	-	-	-	-	-	16.9	9
12/09/2016	9.88	6.02	10.59	-	-	-	-	16.9	13.08
16/09/2016	10.2	6.17	10.54	-	-	-	-	dry	13.23
23/09/2016	10.14	6.67	10.46	16.3	-	-	-	dry	13.12
30/09/2016	10.39	7.1	10.43	16.22	-	-	-	dry	13.14
05/10/2016	10.58	7.2	10.4	16.15	-	-	-	dry	13.16

3.3.Limit equilibrium analysis

Although inclinometers are able to capture accurate deformations of landslides, the 'factor of safety', which is universally used as a measure of proximity of slope conditions to failure, cannot be obtained from the field monitoring data. In addition, geological materials are very complex materials, whose properties are affected by internal and external factors i.e. the gravitational field and rainfall. Therefore these data are local and easily influenced by selection procedure of monitoring boreholes. To perform more effective evaluation of the instability of the landslide, a combination of both techniques is suggested.

A limit equilibrium analysis was performed in our analysis to find the degree of stability of the slope. As mentioned in chapter two, the software used for analysis was geostudio, slope/w package. One method of analysis was utilised,i.e. the Morgenstern-Price Method, as mentioned in chapter two.

3.3.1. Geometry of slope

The slope under analysis was constituted of principally three materials of varying geometry. A superficial layer of moraine (an accumulation of debris, ranges in size from blocks to sand and clay) and a deep-seated base of high strength compact shale separated by an intermediate layer of decayed shale. The slope had a gradient of about 3:1 and the stratigraphy presented a complex geometry as seen in fig.3.12.

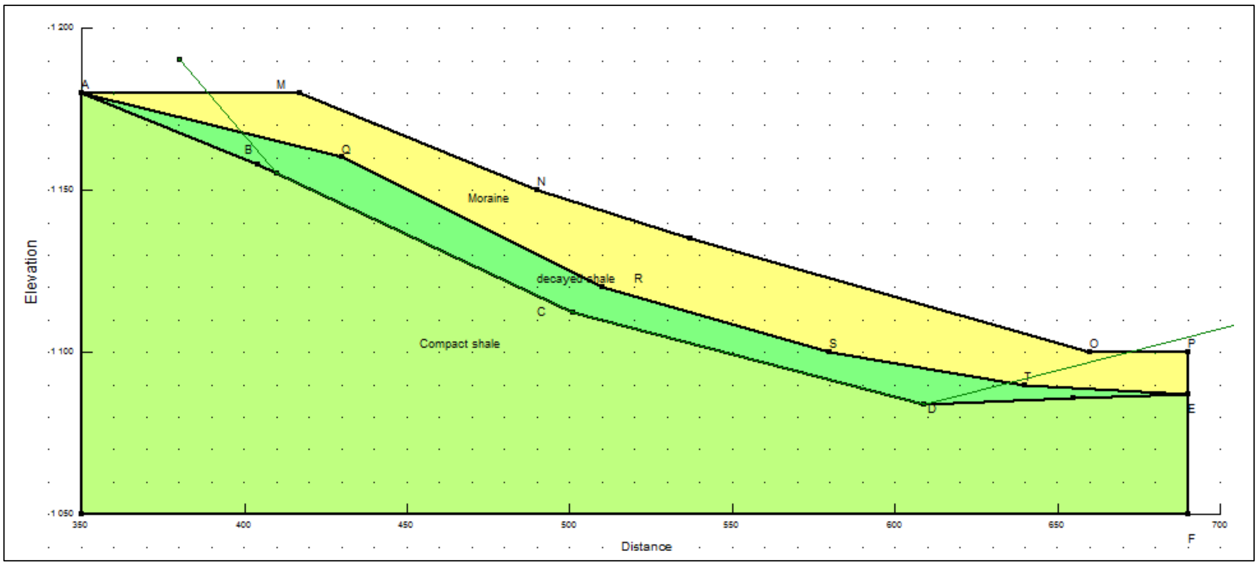

Figure 3.12. Stratigraphy of slope and slip surface definition.

3.3.2. Soil properties

Table 3.6. Geotechnical properties of slope materials.

Parameter	Material		
	Moraine	Compact Shale	Weathered shale
Unit weight(KN/m³)	19	26	22
Effective cohesion (KPa)	0	300	38.4
Effective friction (°)	26	38	15-18
Density (kg/m³)	1800	2500	2000

3.3.3. Piezometric line definition

The piezometric line was defined by taking into account the maximum pore-water head recorded at each borehole. This was to analyse the slope in the most unfavourable conditions. Fig.3.13. presents the piezometric line chosen for the analysis.

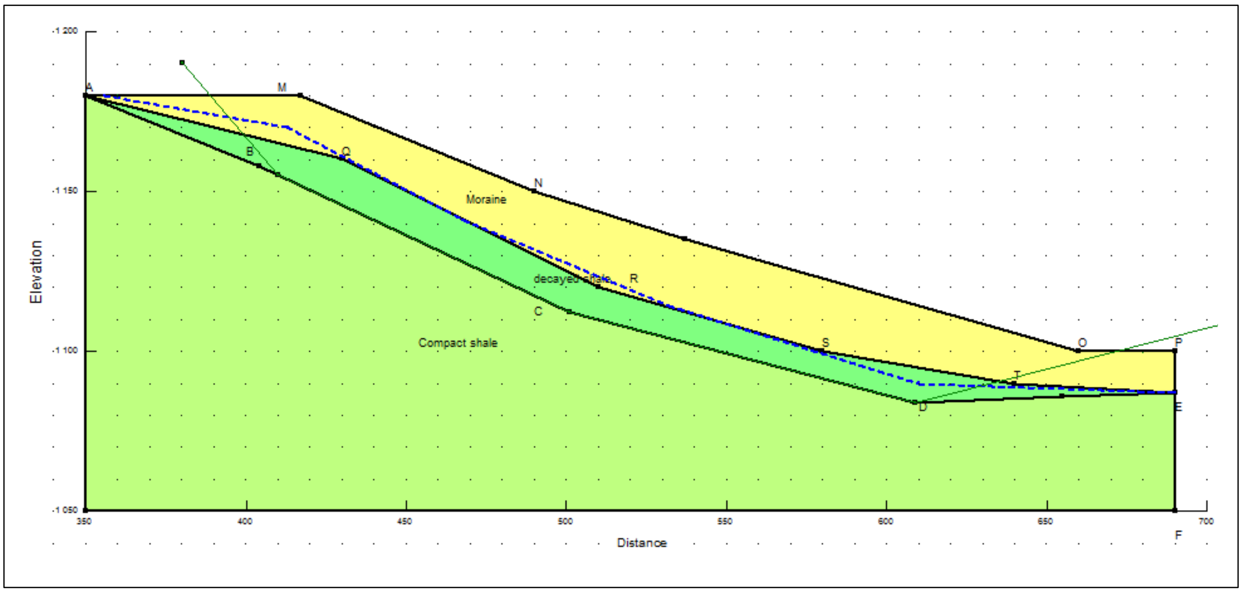

Figure 3.13. Position of piezometric line

3.3.4. Back analyses for residual shear strength properties

A back analyses of the landslide was performed to determine the residual shear strength parameters C’r and φ’r along the critical failure surface at the time of its failure. Here analysis was performed considering highest water level conditions, using LEM software Slope/W, and method of analysis used was the Morgenstern-Price.

Analysis was performed by fixing the value of cohesion at the slip surface to be zero and then varying the friction angle of the material found at this surface until a residual friction angle (φ’) was obtained (friction angle when FoS = 1.00). A factor of safety of FoS = 1.001 was then obtained when friction angle was fixed at 18°, indicating that this was the angle at which the landslide movement began.

This value shall be used in subsequent analyses as the residual friction angle (φ’ = 18°). Figure 3.14 represents the output of the numerical analysis at φ’ = 18° using M-PM.

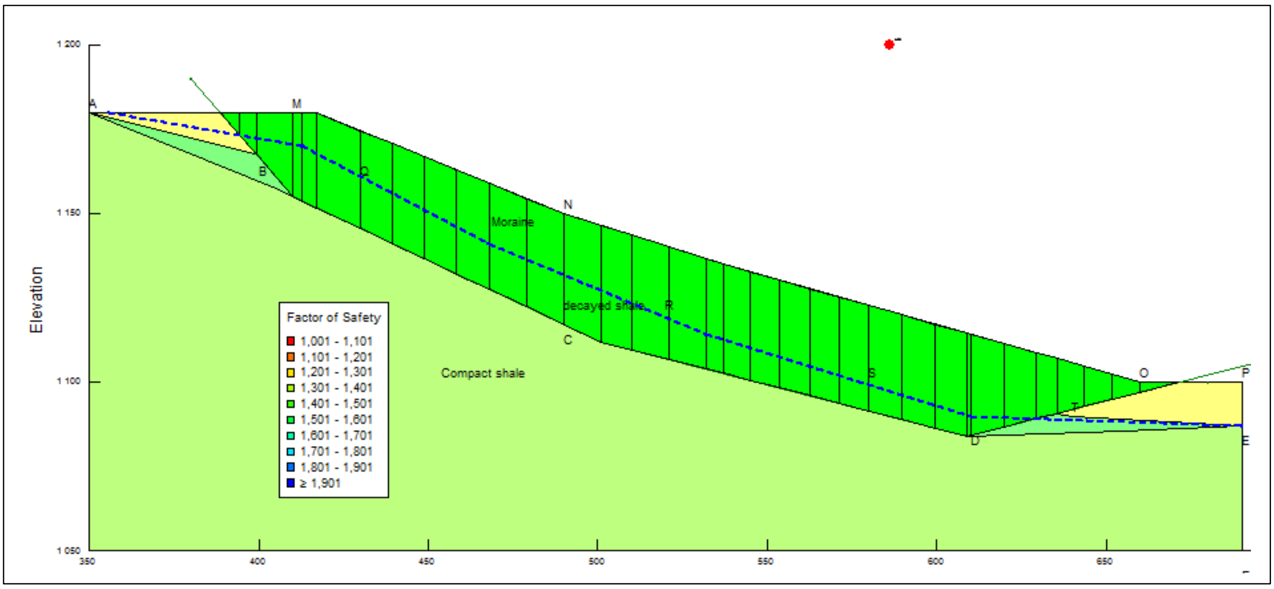

Figure 3.14. Back analysis in wet conditions with M-PM.

3.3.5. Stability analysis of wet unreinforced slope, in pseudo-static conditions.

The slope was analysed here in wet conditions without reinforcement and a coefficient of seismic force was introduced to represent seismic vibrations. Seismic coefficient ($K_h = 0.2$) was gotten from France seismic map (see appendix). Analysis were performed using the Morgenstern-Price Method, and using residual strength parameters.

After running the analysis, a drop in the FoS was noticed, from a value of FoS = 1.001 in static conditions to a value of FoS = 0.613. This is explained by the fact that seismic forces cause a disorientation of gravitational forces acting on the slope, thereby causing an increase in driving forces or causing a decrease in resisting forces. The results of the numerical analysis is seen on figure 3.15. For the stabilization models, the pseudo-static condition shall be applied since it is the most critical condition.

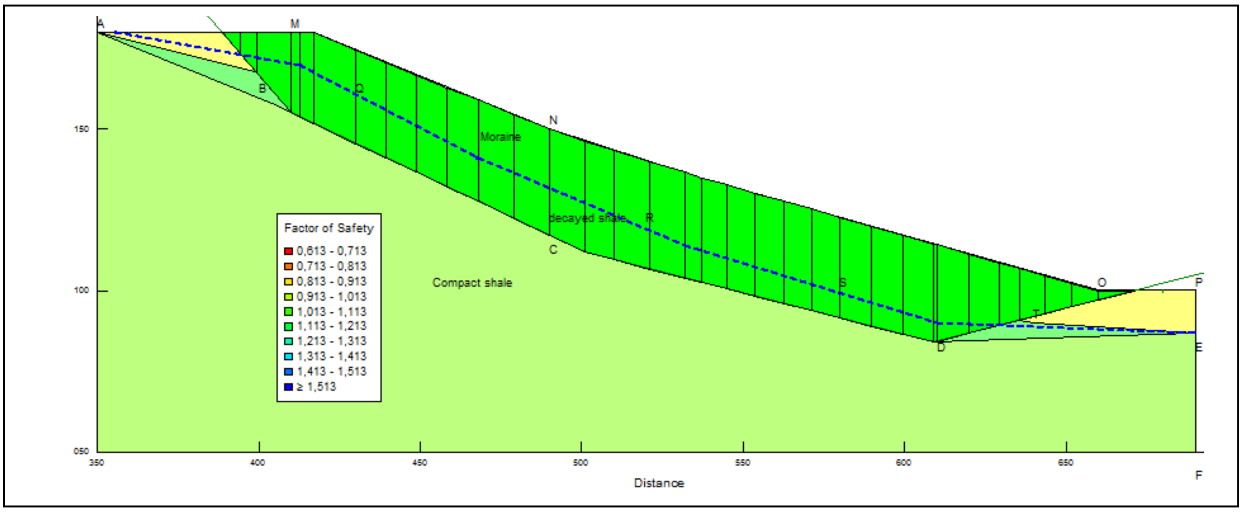

Figure 3.15. Analysis of wet unreinforced slope in pseudo-static conditions with M-PM.

3.4. Slope stabilization

Slope stabilization with reinforcements provides to increase shear strength or reduce the sliding actions along the slip surface with various types of structures such as retaining walls, micropile systems, ground anchors, soil nailing, etc.

Two criteria were used in selecting a stabilizing structure for the slope. Firstly, the selected method must restore the necessary factor of safety for the slope to be considered stable, and secondly, the chosen solution should be cost-effective. According to Cola S. and alt. (2012), retaining walls structures are good in stabilizing very small landslides were the slip surface is shallow. For that reason, they could not be applied in this study were the slip surface was a deep one. The construction of a retaining wall also implied making very large excavations, which proved unfavourable given the site conditions.

Anchors were what was opted to be used in reinforcing the slope, since it has no length limit, and installation of anchors are relatively easier.

3.4.1. Stabilization by the use of ground anchors

Ground anchors are a common stabilization method for slopes. The anchors transfer the load on the face of the slope deeper into the ground behind the slope. Due to the limitations of the retaining wall as an appropriate remediation, anchors were used as a substitute in reinforcing the slope. Anchors were chosen because they present several advantages in slope remediation, as (i) they can be installed on uneven surfaces if needed, thereby reducing cost of site preparations; (ii) Construction is rapid and provides only minor disruption of the surrounding area, (D. Cornforth, 2005). When anchors cross the slip surface at oblique angles, they help in improving the shear strength at the slip surface.

3.4.1.1. Design of Ground anchors

The preliminary design procedures for ground anchors were already seen in chapter two, reinforcement anchors used for this study were designed following those procedures.

Two hypothesis were used in the design.

i. The slope angle of the critical slip surface shall be considered to be identical to the angle formed by the bedrock surface and the horizontal.
ii. The driving forces shall be considered as the resultant of gravitational and seismic forces acting on each slice. An example of forces configuration on a slice is shown in figure 3.16.

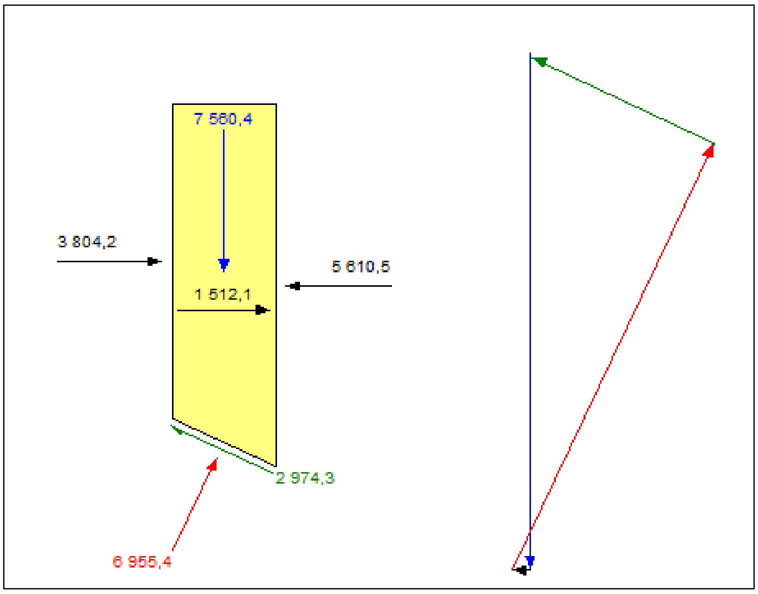

Figure 3.16. Free body diagram and force polygon of slice 6.

The principal parameters needed in designing an anchor are the pull-out resistance, the tensile strength, the shear strength and the anchor fixation length. But these parameters are influenced by certain factors such as the friction angle at the slip surface and the angle of inclination of the anchors to the horizontal.

For this design, residual friction angle gotten during the back-analysis is used and the angle of anchor inclination will be varied from 25° to 35° and then to 45°.

All reduction factors were assigned a value of one, the anchor diameter (Da) taken as 15cm and anchor spacing taken to be 1m. The ultimate bond stress per unit length of anchor bar was taken as 450kpa as according to the recommended value for hard shale (see figure 2.7).

An initial design was done considering the anchor to be inclined at $\alpha = 45°$. Then, the parameters of the anchor gotten for $\alpha = 45°$ were maintained constant for all other analyses and just the angle of inclination was varied so as to observe the effect of varying the inclination angle of anchors.

$$\text{Required preventive power (P)} = \frac{(PFs-Fs)*\Sigma T}{\cos(\alpha+\theta)+\sin(\alpha+\theta)*tan\varphi}$$

$$= \frac{(1.15-0.613)*11013.7\text{kN}}{0.5+0.27}$$

$$= 5914.3 \text{ kN}$$

$$\text{Tensile capacity (P1)} = P*\cos(\alpha+\theta)$$

$$= 5914.3\text{kN} * 0.5$$

$$= 3957.2 \text{ kN}$$

Shear capacity (P2) $= P^* \sin(\alpha + \theta)$

$= 5914.3kN * 0.86$

$= 5086.3$ kN

Pull-out resistance (Td) $= \dfrac{P*B}{(\sin(\alpha+\theta)tan\varphi+\cos(\alpha+\theta))N}$

$= \dfrac{5086.3kN*1m}{(0.27+0.5)*6}$

$= 1100kN$

Anchor fixation length (la) $= \dfrac{Td}{3.14*Da*\mathrm{qu}}$

$= \dfrac{1100kN}{3.14*0.15m*450kpa}$

$= 5.18m$

Table 3.7. Anchor parameters.

Anchor Parameter	Value
Tensile capacity	3957.2kN
Shear capacity	5086.3kN
Pull-out resistance	1100kN
Anchor fixation length	5.18m

3.4.1.2. Stability analysis of slope model with reinforcements

The unstable slope model was reinforced in SLOPE/W using ground anchor reinforcements designed as seen in section 3.4.1.1, have parameters found in table 3.7. These parameters were inputed in the software as presented in figure 3.17. The free length of the anchors were dependent on the angle at which the anchors were inclined to the horizontal. Anchors 1, 2 and 3 in figure 3.17 represent the anchors found at the base of the slope, while anchors 4, 5, and 6 represent the anchors found at slope top. The slope model was then analysed under wet conditions, in the presence of seismic vibrations. Six rows of ground anchors were included in the 2D analysis this time around, having characteristics described in table 3.7. Analysis was performed using the Morgenstern-Price Method, and seismic force coefficient was taken as 0.2.

a) Analysis at α = 45°

A first analysis was performed when the anchors were inclined at an angle of α = 45° to the horizontal. A FoS = 1.142 was obtained with the M-PM as seen in figure 3.18.

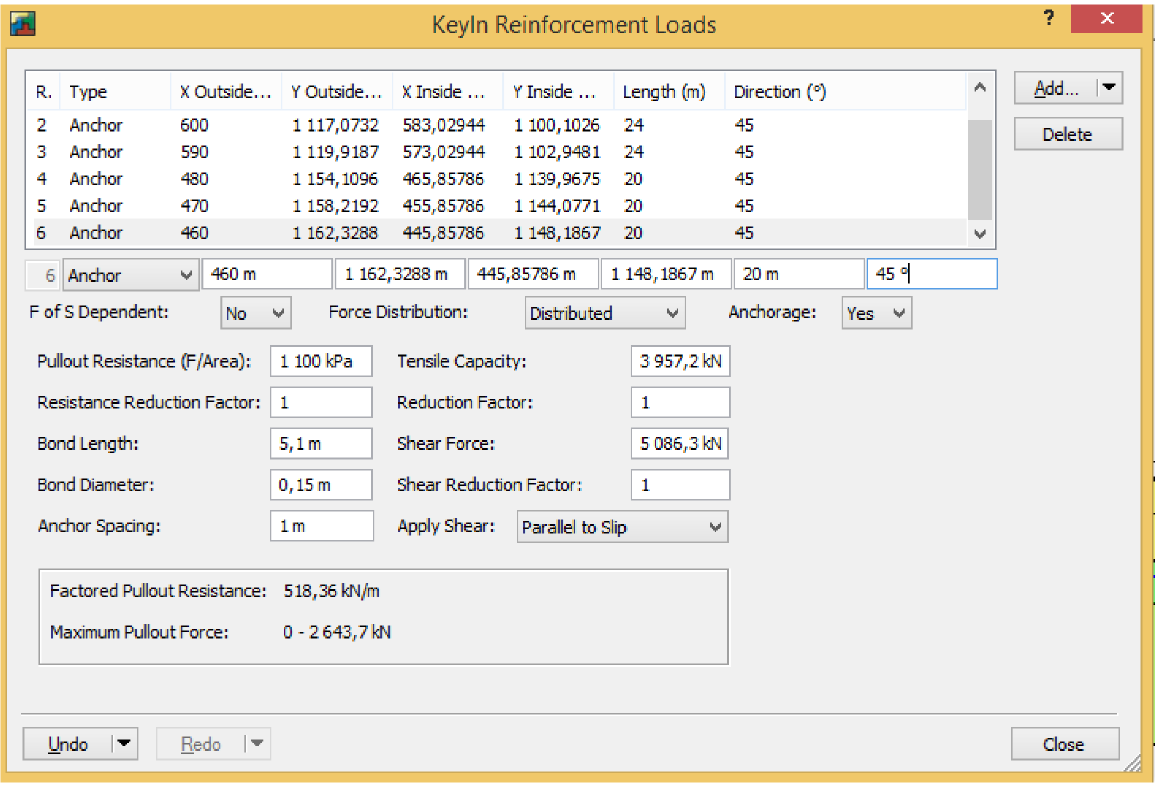

Figure 3.17. Slope/w interface for entering reinforcement parameters.

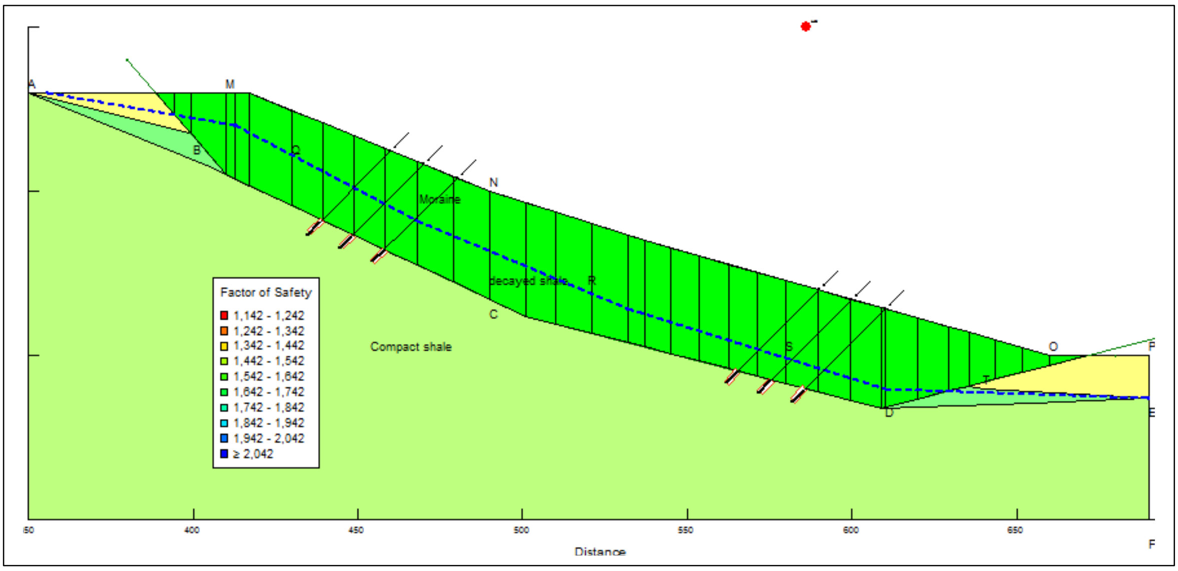

Figure 3.18. Reinforced slope analysis in wet conditions, at α = 45°.

b) Analysis at α = 35°

A second analysis was then performed on the same model, using the same reinforcements but at angle of inclination of α = 35°. Due to the decrease in inclination angle, the length of the lower anchors had to be increased from l = 24m to l = 28m while the upper anchors increased from l = 20m to a length of l = 22m. Running the analysis with this angle gave an increased value of factor of safety as seen in figure 3.19. A FoS = 1.201 was obtained with the M-PM.

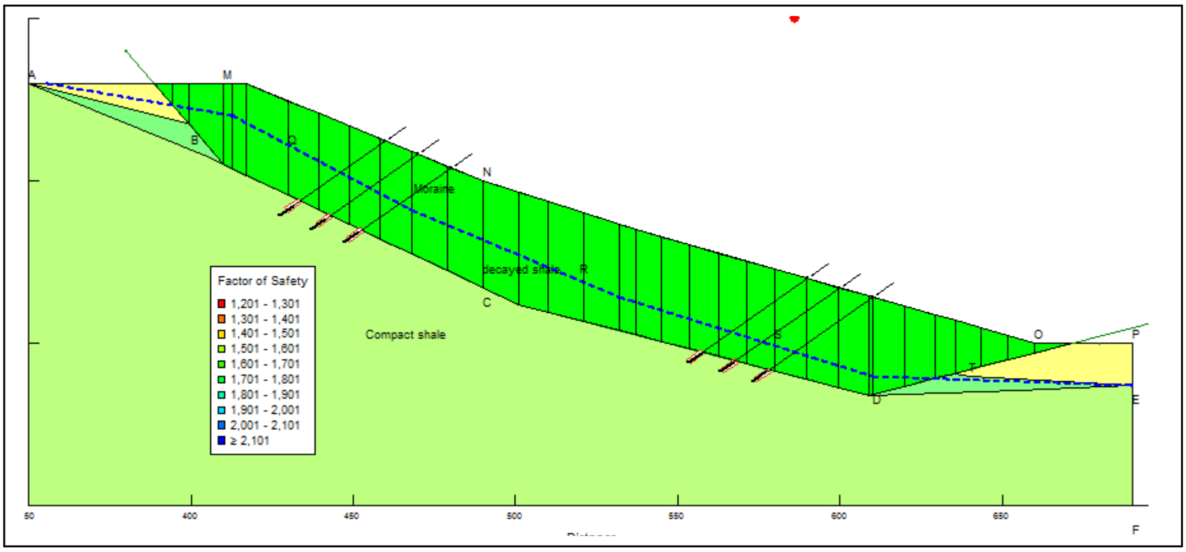

Figure. 3.19. Reinforced slope analysis in wet conditions, at α = 35°.

c) Analysis at α = 25°

The inclination angle was then reduced to 25° with all other parameters remaining unchanged. The length of the lower anchor was increased to 32m, while upper anchors to 24m.Running the analysis with the M-PM, an increased factor of safety of FoS = 1.255 was obtained as seen in figure 3.20.

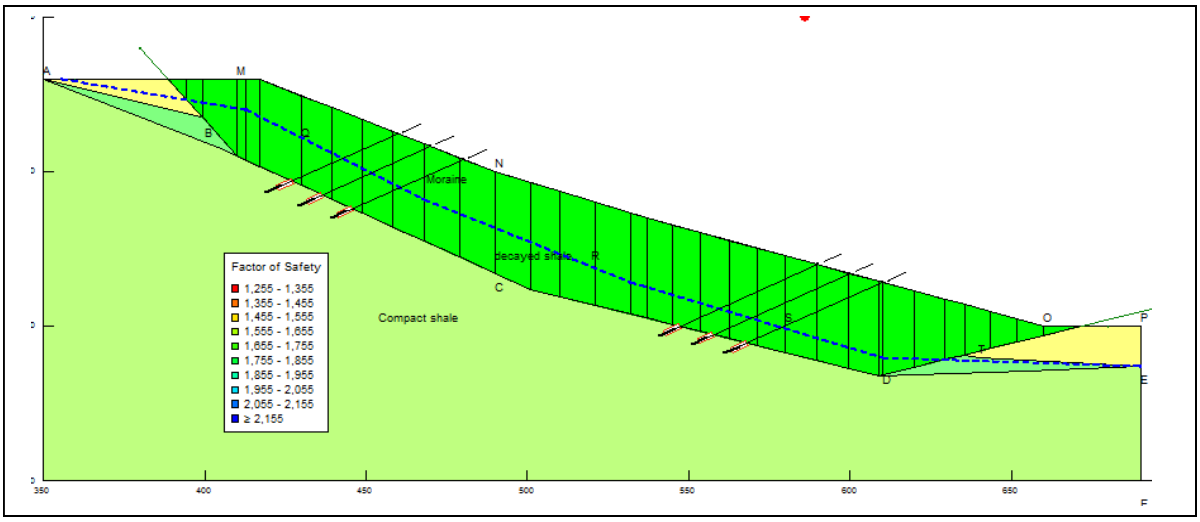

Figure. 3.20. Reinforced slope analysis in wet conditions, at α = 25°.

3.4.1.3. Interpretation of stability analyses

From the results of all the stabilization analyses, it could be observed that the use of ground anchors in remediating the slope proved to be effective as they gave satisfactory factors of safety.

All the analyses which were performed on the slope model in presence of anchor reinforcements all gave values of FoS close to or greater than 1.15 which is the minimum value of safety for seismic zones (Seed, 1979), this implies stability has been reached. It was noticed that the FoS of an anchor reinforced slope is greatly dependent on the angle at which the anchor is inclined to the horizontal, an increasing trend of FoS was observed as the angle of inclination was been decreased, and an optimum value of FoS was obtained when the anchor was inclined at $\alpha = 25°$.

3.4.2. Complementary remarks on slope remediation

It should be noted that the presence of a high water head is usually among the primary causes of landslides. This is due to a decrease in the effective shear strength of the soil, since an increase in pore water pressure implies a decrease in effective shear strength. This implies that introducing a drainage system in the slope remediation program will increase the factor of safety to a higher value. An effective method of draining the slope is by using horizontal drains (figure 1.23.), as they need just gravity for drainage contrary to deep well points where there is a need for a permanent energy source for pumping water out of the well.

Conclusion

Results of all surveys conducted on site and analysis performed were presented in this chapter. Geotechnical characteristics of different formations present on site were presented. Inclinometric data was interpreted and sharp movements were noticed at the moraine-shale interface, this implied this could be the potential sliding surface of the landslide.

The study area model was analysed varying various parameters to see their influence on the slope instability. Back-analysis was conducted in undrained conditions to find the residual strength parameters of the slip surface at moment of failure. Then from these residual parameters the slope could be analysed under pseudo-static conditions and ground anchors reinforcement designed. Then, six rows of ground anchors were used as reinforcement. These anchors were placed at different angles to the horizontal so as to get an optimum inclination. An optimum FoS of 1.255 was obtained when the anchors were placed at an angle of 25° to the horizontal implying the slope had attained stability.

Apart from the anchors, the installation of a drainage system was proposed as a complimentary solution to reduce the water table head, since it has a negative effect on the stability of the slope.

GENERAL CONCLUSION AND PERSPECTIVES

Landslide movement caused by movement of tectonic plates has been presented in this work. The Charmaix Bridge has been subjected to movements at the level of its foundation due the instability of the mountains on which it was constructed. It was discovered that these instability movements were a result of the plate tectonic movements that constantly occurred in the region.

The objectives of this thesis were firstly to predict a potential sliding movement occurring on the slope from the monitoring data available, and then finally proposing a solution to the sliding movement. The monitoring surveys constituted of placing inclinometers at specific locations for monitoring of ground displacements, and piezometers installations too. Data from five inclinometers were used in this work to determine the position of the potential slip surface and also to identify the direction of sliding movement. Other surveys such as the Ménard pressuremetric tests and destructive tests were also performed to gain more information on the site geotechnical conditions.

In the first chapter of this work, landslides were presented as a mode of soil failure mechanism. The chapter began with a definition of soils followed by soil constituents. Then, the processes of soil formation were presented, and the different factors affecting soil formation processes also presented. This was followed by a discussion on some geotechnical soil characteristics. Then soil types were presented and modes of soil failure presented. Here, landslides were defined, causes of landslides given, and classified. Different monitoring techniques were also discussed and then soil stabilization methods presented.

In the second chapter, the methodology used in getting results was discussed. Then the entire monitoring program was discussed, from the different site characterization surveys such as the pressuremeter tests and the destructive tests to the monitoring program which was proposed. The chapter ended with a presentation of the SLOPE/W software which was used for numerical modelling, and different soil reinforcement techniques were also discussed. In modelling the slope, it was assumed that the coordinates of the slope surface coincided with the coordinates of the inclinometers installed on the slope surface. It was seen that ground anchors would be the most suitable of all available techniques to be used in this study, due to some reasons such as its relative cost, the non-cohesive nature of the overlying slope material and the fact that it can be installed adjacent to other structures without difficulty. The preliminary design steps of ground anchors was then presented.

The last chapter served for results presentations and interpretation. Analysing the inclinometric curves revealed the presence of sharp movements at the decayed shale layer, which is a transition layer between the moraine formation and the underground bedrock. This implied that this transition interface could be a potential slip surface.

Also, a back-analysis of the slope model was performed to find the residual shear strength parameters of the decayed shale layer at which the potential slip surface is found. This was done by varying the friction angle and keeping cohesion constant until a factor of safety of unity was obtained. Frome the back-analyses, a residual friction angle of $\varphi'r = 18°$ was gotten. The slope model was then analysed under pseudo-static conditions taking into consideration seismic forces, and using residual shear strength parameters gotten from the back-analyses. A factor of safety of 0.613 was gotten from the pseudo-static analyses, and this represented worse slope conditions. Ground anchors were then designed as a possible remediation to the slope stability. A last analysis was then performed in the presence of six rows of ground anchor reinforcements as a possible solution in stabilizing the slope. Varying the angle of inclination of the anchors to the horizontal from 25° to 45° showed that the optimum FoS is obtained when the anchors are inclined 25° to the horizontal, since at 25° a FoS of 1.255 was obtained with the M-PM. The minimum FoS to consider a slope subjected to a horizontal seismic acceleration coefficient of 0.2 as stable as per Seed (1979) is 1.15, so the slope is considered to be stable under these reinforcements.

As for the challenges encountered during this study, the first challenge was the absence of information on the positions of some inclinometers and piezometers. Since the slope geometry was defined in function of the inclinometers' positions, absence of these information handicapped the definition of the geometry with a greater precision. Secondly, data on the laboratory tests carried out on different soil samples could not be gotten for a better characterization of the slope material. Also, some information of piezometer measurements were not yet updated such as piezometer pz605.

So, for a better understanding of landslide processes occurring on these mountains, an extensive monitoring should be made, particularly inclinometric and piezometric measurements, this will permit not only to better define a potential slip surface, but also it would provide more information on the rates of ground movements at different points within the slope. Then more laboratory tests should be performed on samples taken from site so as to have a better characterization of site materials. This will give more precision in the results, since slope movement is greatly influenced by material properties.

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APPENDIX

Appendix 1: Pressuremeter test and destructive test survey result sheets.

Appendix 2: Inclinometers and piezometers implantation plans.

Appendix 3: Vertical soil profiles at inclinometer and piezometer points.

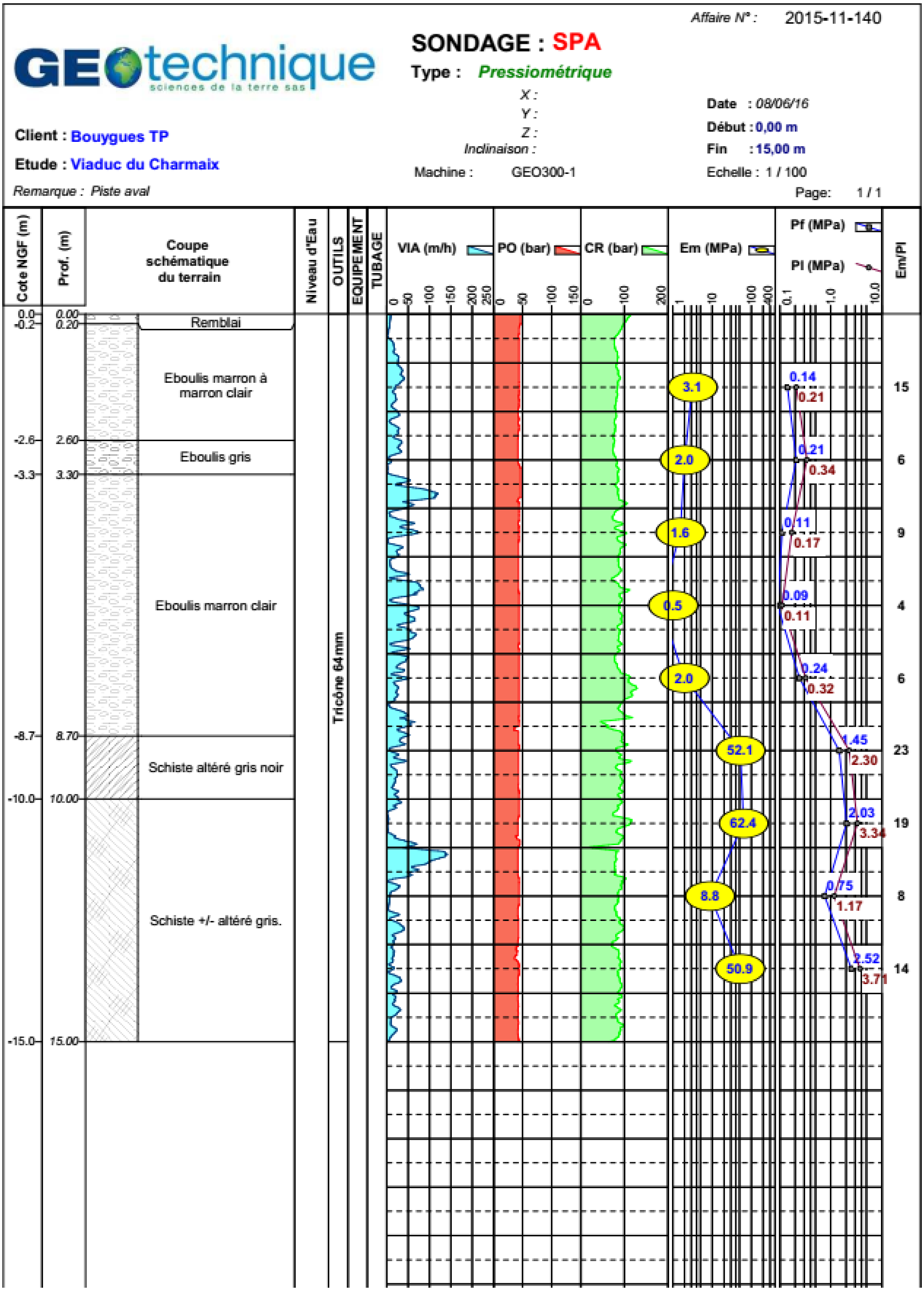
Appendix 4: Inclinometer curves (displacements in B- direction).

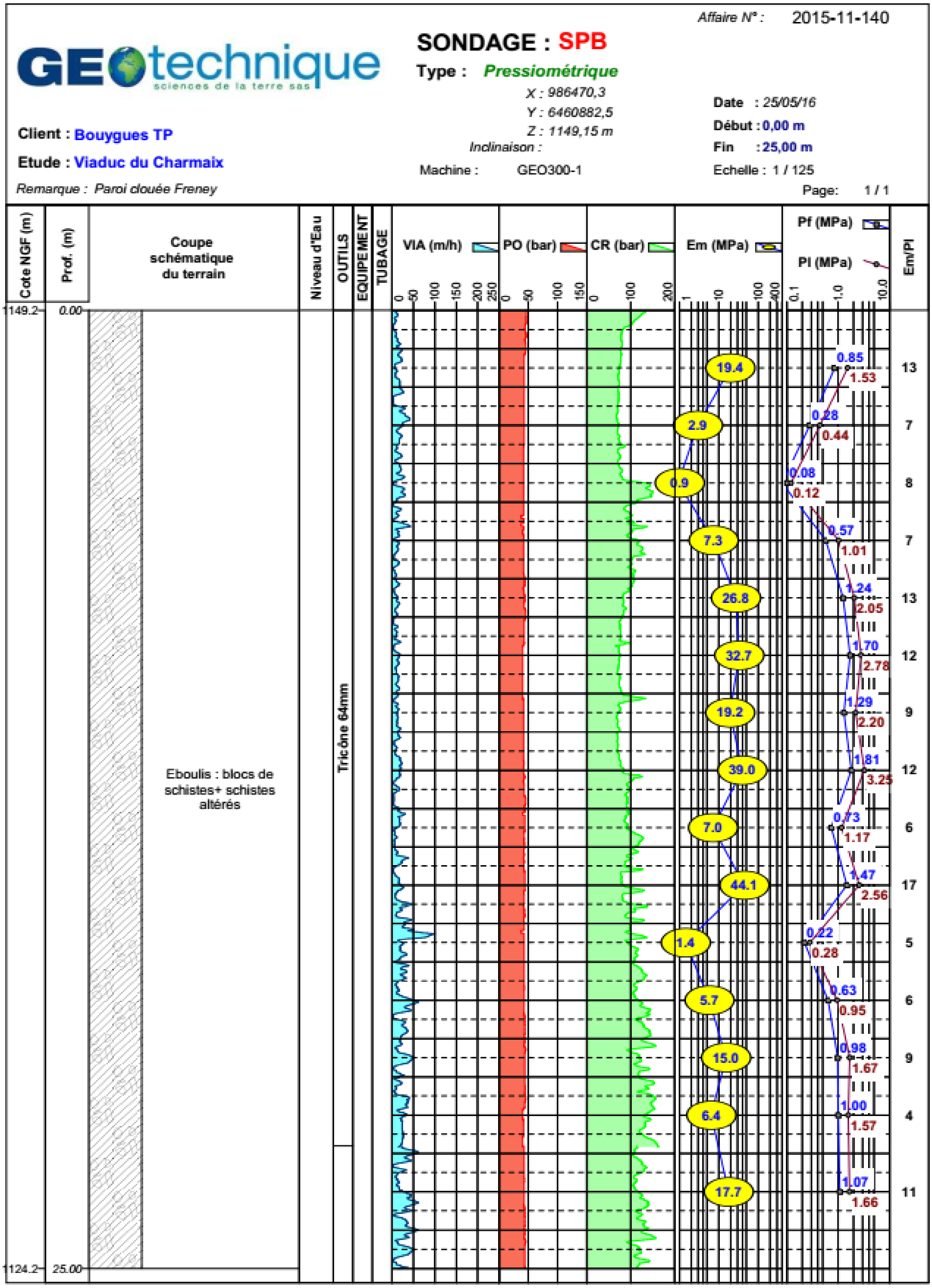
Appendix 5: Map of seismic acceleration isovalues in France.

Appendix 6: Tensile strength catalogue of different materials.

Appendix 7: LIDAR images.

Appendix 1: Pressure meter test and destructive test survey result sheets

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sciences de la terre sas
SONDAGE : SPB
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Y : 6460882,5
Z : 1149,15 m
Inclinaison :
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Date : 25/05/16
Début : 0,00 m
Fin : 25,00 m
Echelle : 1 / 125
Page: 1 / 1
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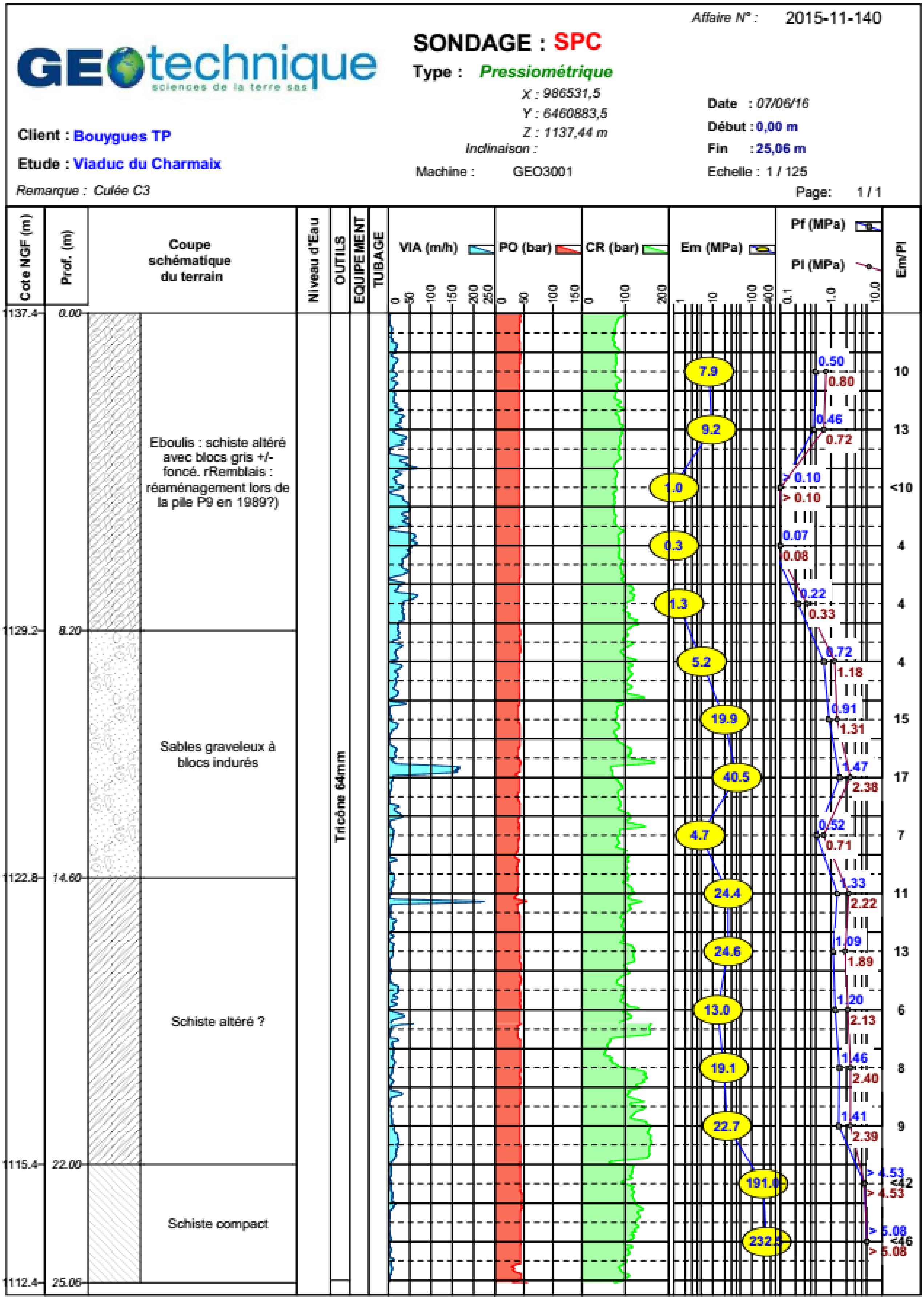
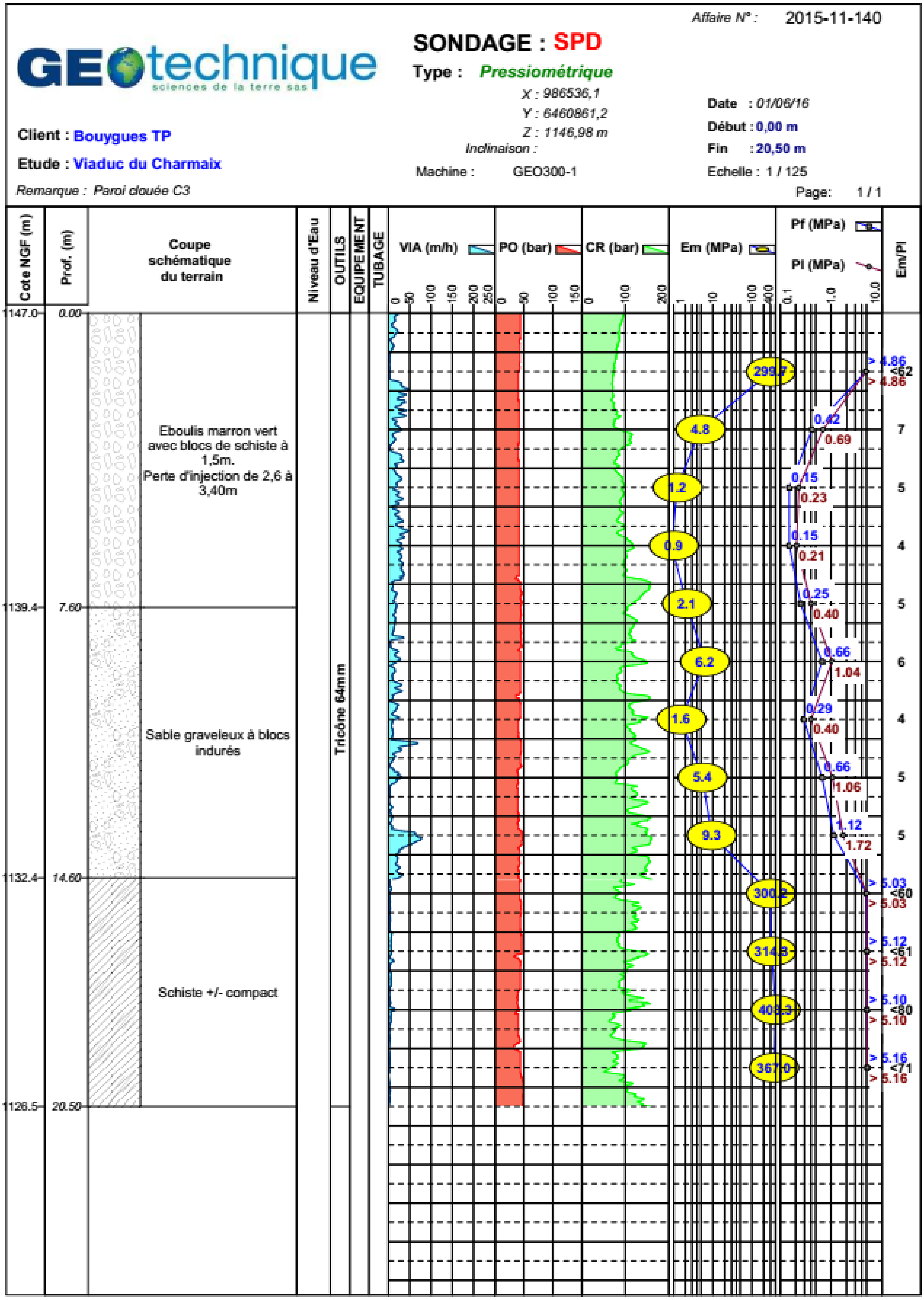
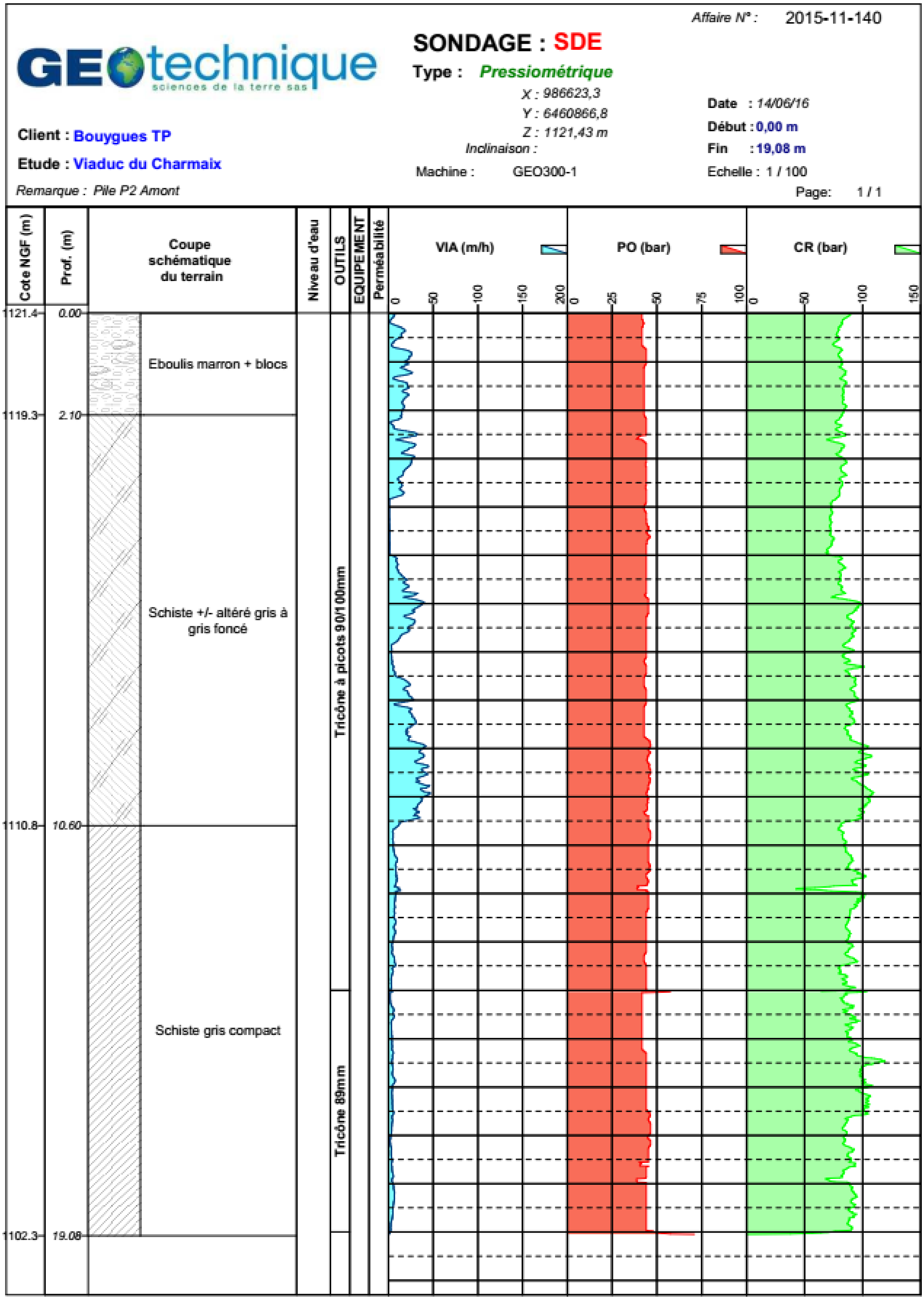
Etude : Viaduc du Charmaix
Remarque : Paroi clouée Freney
Cote NGF (m)
Prof. (m)
Coupe schématique du terrain
Niveau d'Eau
OUTILS
EQUIPEMENT
TUBAGE
VIA (m/h)
PO (bar)
CR (bar)
Em (MPa)
Pf (MPa)
Pl (MPa)
Em/Pl
1149.2
0.00
Eboulis : blocs de schistes+ schistes altérés
Tricône 64mm
1124.2
25.00
19.4
2.9
0.9
7.3
26.8
32.7
19.2
39.0
7.0
44.1
1.4
5.7
15.0
6.4
17.7
0.85
1.53
0.28
0.44
0.08
0.12
0.57
1.01
1.24
2.05
1.70
2.78
1.29
2.20
1.81
3.25
0.73
1.17
1.47
2.56
0.22
0.28
0.63
0.95
0.98
1.67
1.00
1.57
1.07
1.66
13
7
8
7
13
12
9
12
6
17
5
6
9
4
11


Affaire N° : 2015-11-140
GEOtechnique
sciences de la terre sas
SONDAGE : SPC
Type : Pressiométrique
X : 986531,5
Y : 6460883,5
Z : 1137,44 m
Inclinaison :
Date : 07/06/16
Début : 0,00 m
Fin : 25,06 m
Client : Bouygues TP
Etude : Viaduc du Charmaix
Machine : GEO3001
Echelle : 1 / 125
Remarque : Culée C3
Page: 1 / 1
Cote NGF (m)
Prof. (m)
Coupe schématique du terrain
Niveau d'Eau
OUTILS
EQUIPEMENT
TUBAGE
VIA (m/h)
PO (bar)
CR (bar)
Em (MPa)
Pf (MPa)
Pl (MPa)
Em/Pl
1137.4
0.00
1129.2
8.20
1122.8
14.60
1115.4
22.00
1112.4
25.06
Eboulis : schiste altéré avec blocs gris +/- foncé. rRemblais : réaménagement lors de la pile P9 en 1989?)
Sables graveleux à blocs indurés
Schiste altéré ?
Schiste compact
Tricône 64mm
7.9
9.2
1.0
0.3
1.3
5.2
19.9
40.5
4.7
24.4
24.6
13.0
19.1
22.7
191.0
232.5
0.50
0.80
0.46
0.72
> 0.10
> 0.10
0.07
0.08
0.22
0.33
0.72
1.18
0.91
1.31
1.47
2.38
0.52
0.71
1.33
2.22
1.09
1.89
1.20
2.13
1.46
2.40
1.41
2.39
> 4.53
> 4.53
> 5.08
> 5.08
10
13
<10
4
4
4
15
17
7
11
13
6
8
9
<42
<46



Affaire N° : 2015-11-140
GEOtechnique
sciences de la terre sas
SONDAGE : SPD
Type : Pressiométrique
X : 986536,1
Y : 6460861,2
Z : 1146,98 m
Inclinaison :
Machine : GEO300-1
Date : 01/06/16
Début : 0,00 m
Fin : 20,50 m
Echelle : 1 / 125
Page: 1 / 1
Client : Bouygues TP
Etude : Viaduc du Charmaix
Remarque : Paroi clouée C3
Cote NGF (m)
Prof. (m)
Coupe schématique du terrain
Niveau d'Eau
OUTILS
EQUIPEMENT
TUBAGE
VIA (m/h)
PO (bar)
CR (bar)
Em (MPa)
Pf (MPa)
Pl (MPa)
Em/Pl
1147.0 0.00
1139.4 7.60
1132.4 14.60
1126.5 20.50
Eboulis marron vert avec blocs de schiste à 1,5m.
Perte d'injection de 2,6 à 3,40m
Sable graveleux à blocs indurés
Schiste +/- compact
Tricône 64mm
299.7
4.8
1.2
0.9
2.1
6.2
1.6
5.4
9.3
300.2
314.8
408.3
367.0
> 4.86
> 4.86
<62
0.42
0.69
7
0.15
0.23
5
0.15
0.21
4
0.25
0.40
5
0.66
1.04
6
0.29
0.40
4
0.66
1.06
5
1.12
1.72
5
> 5.03
> 5.03
<60
> 5.12
> 5.12
<61
> 5.10
> 5.10
<80
> 5.16
> 5.16
<71


Affaire N° : 2015-11-140
GEOtechnique
sciences de la terre sas
SONDAGE : SDE
Type : Pressiométrique
X : 986623,3
Y : 6460866,8
Z : 1121,43 m
Inclinaison :
Machine : GEO300-1
Date : 14/06/16
Début : 0,00 m
Fin : 19,08 m
Echelle : 1 / 100
Page: 1 / 1
Client : Bouygues TP
Etude : Viaduc du Charmaix
Remarque : Pile P2 Amont
Cote NGF (m)
Prof. (m)
Coupe schématique du terrain
Niveau d'eau
OUTILS
EQUIPEMENT
Perméabilité
VIA (m/h)
PO (bar)
CR (bar)
1121.4
0.00
Eboulis marron + blocs
1119.3
2.10
Schiste +/- altéré gris à gris foncé
1110.8
10.60
Schiste gris compact
1102.3
19.08
Tricône à picots 90/100mm
Tricône 89mm

GEOtechnique
sciences de la terre sas

Affaire N° : 2015-11-140

SONDAGE : SDF

Type : *Pressiométrique*

X : 986627,7
Y : 6460868,3
Z : 1121,45 m
Inclinaison :

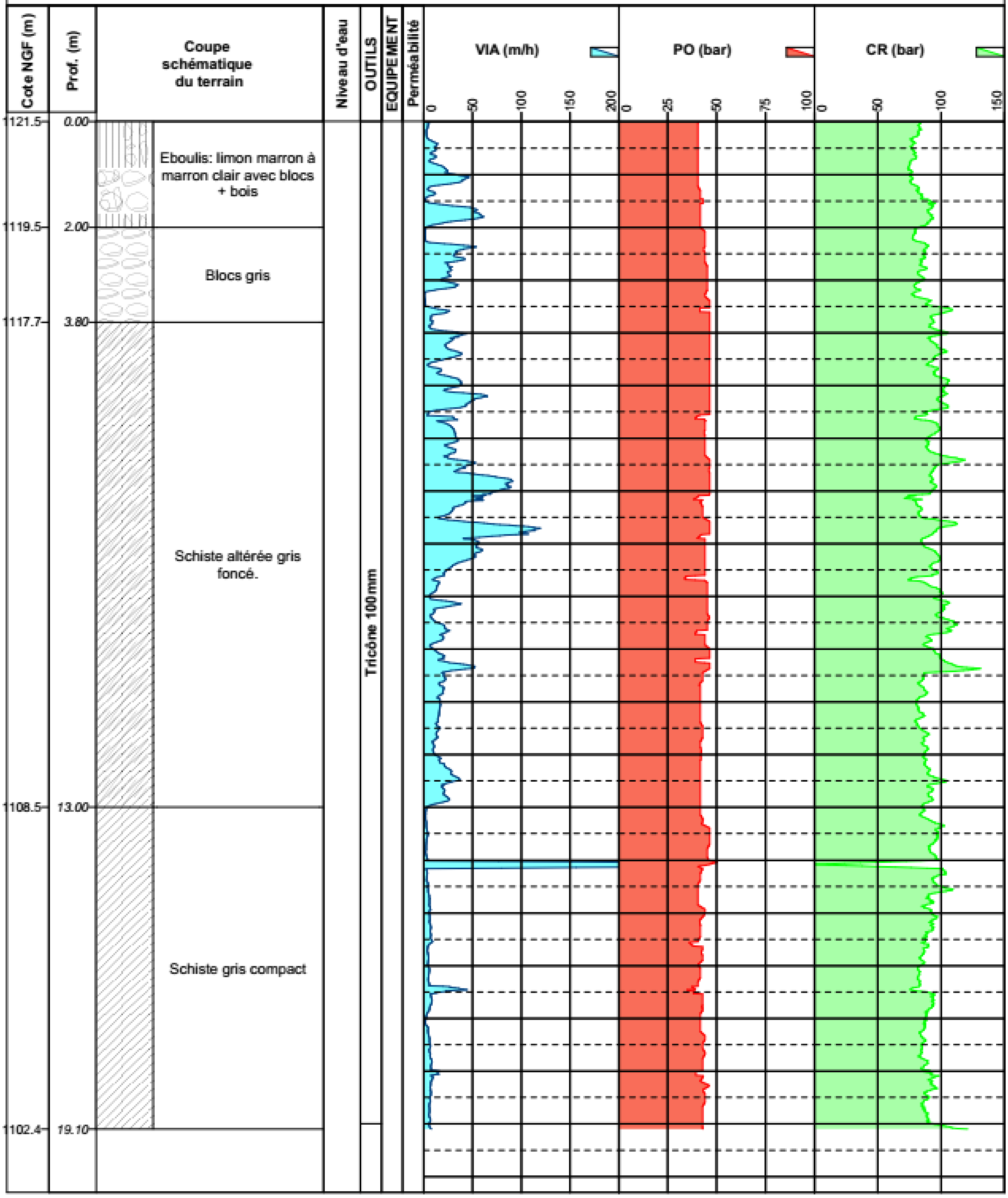
Machine : GEO300-1

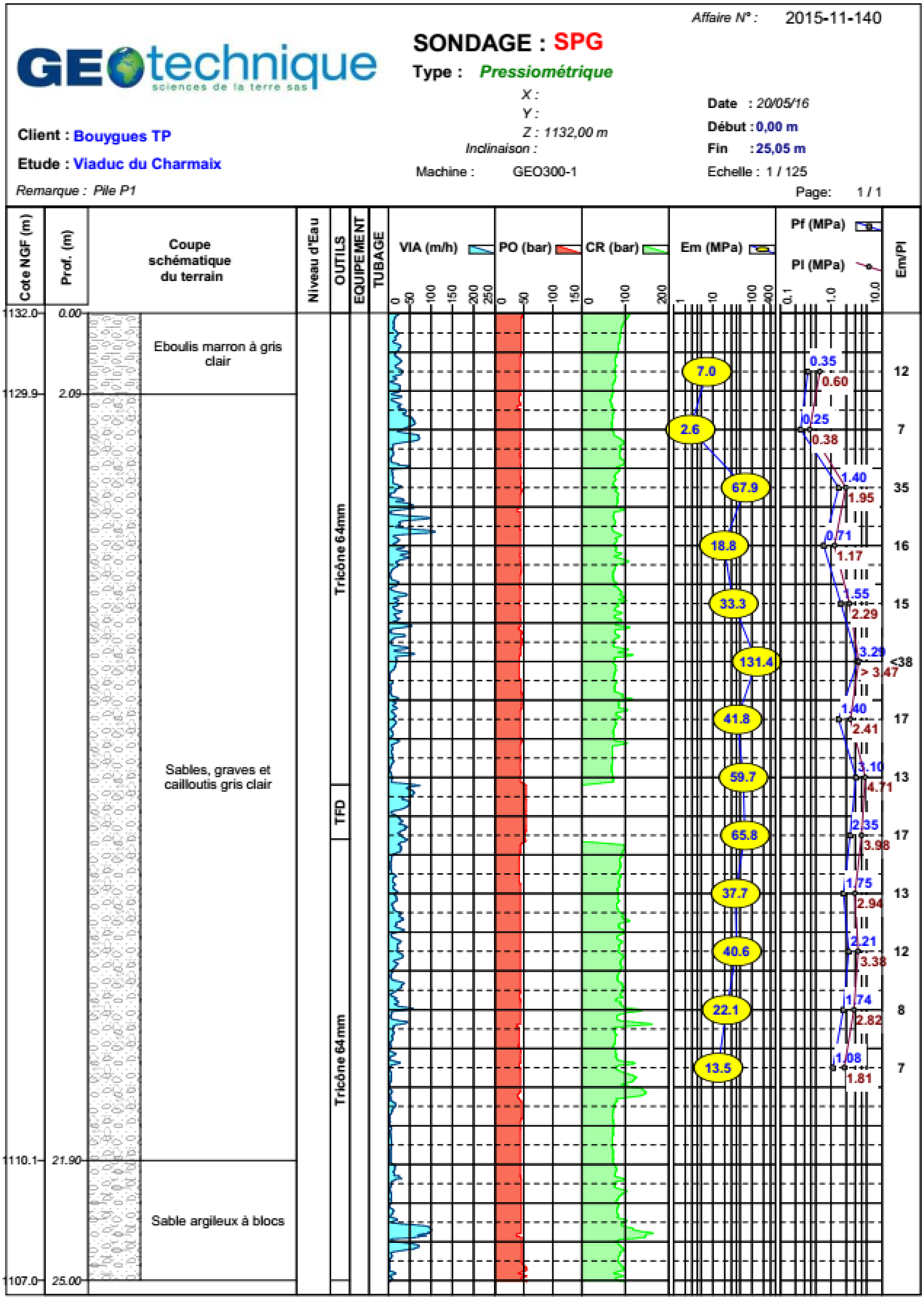
Client : Bouygues TP

Etude : Viaduc du Charmaix

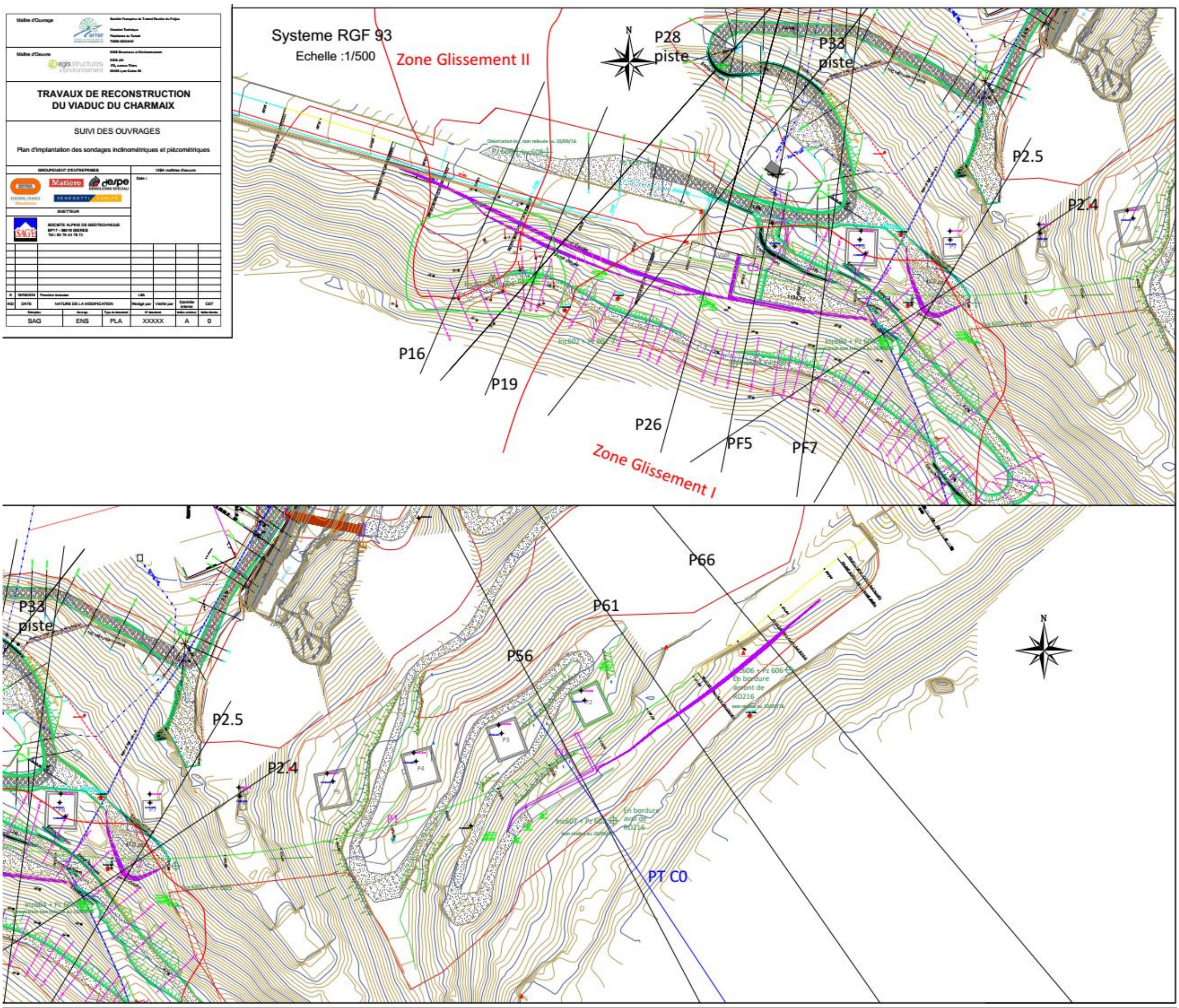
Remarque : Pile P2 Aval

Date : 15/06/16
Début : 0,00 m
Fin : 19,10 m
Echelle : 1 / 100
Page: 1 / 1




Affaire N° : 2015-11-140
GEOtechnique sciences de la terre sas
SONDAGE : SPG
Type : Pressiométrique
X :
Y :
Z : 1132,00 m
Inclinaison :
Machine : GEO300-1
Date : 20/05/16
Début : 0,00 m
Fin : 25,05 m
Echelle : 1 / 125
Page: 1 / 1
Client : Bouygues TP
Etude : Viaduc du Charmaix
Remarque : Pile P1
Cote NGF (m)
Prof. (m)
Coupe schématique du terrain
Niveau d'Eau
OUTILS
EQUIPEMENT
TUBAGE
VIA (m/h)
PO (bar)
CR (bar)
Em (MPa)
Pf (MPa)
Pl (MPa)
Em/Pl
1132.0 0.00
1129.9 2.09
1110.1 21.90
1107.0 25.00
Eboulis marron à gris clair
Sables, graves et cailloutis gris clair
Sable argileux à blocs
Tricône 64mm
TFD
Tricône 64mm
7.0
2.6
67.9
18.8
33.3
131.4
41.8
59.7
65.8
37.7
40.6
22.1
13.5
0.35
0.60
0.25
0.38
1.40
1.95
0.71
1.17
1.55
2.29
3.29
> 3.47
1.40
2.41
3.10
4.71
2.35
3.98
1.75
2.94
2.21
3.38
1.74
2.82
1.08
1.81
12
7
35
16
15
<38
17
13
17
13
12
8
7

Appendix 2: Inclinometers and piezometers implantation plans



Appendix 3: Vertical soil profiles at inclinometer and piezometer points

SUIVI DE SONDAGE

AZURITE Sondages Forages Essais SAS

Client :

Commune : Modane

Sondage : 601 | Dossier n° :

Equipe : | Date début : | Date fin : | Heure début : | Heure fin :

Réalisation du sondage : Sondage destructif [X] Sondage carotté [ ] Avec enregistrement [X] Sans enregistrement [ ]

Profondeur De	Profondeur à	Echantillons / Cuttings / Carottes Nature / texture / couleur / odeur / humidité ... Notes de forage : perte d'injection, ...
0,00	2,70	bloc shisteux
2,70	19,80	Moraine a bloc shisteux
19,80	35,00	Rocher (Shiste).
eau	de	7,50 m a 16,30 / TN

Niveau d'eau en fin de forage :

Fluide de forage

[ ] Eau claire
[X] Injection d'air
[ ] Polycol
[ ] ....

Type d'outils / Réalésage

MFT ø115 De 0,00 à 35,00
.... ø....... De.......... À..........

Type de tubage

Tubage odex ø 115
De 0,00 À 20,00
Tubage .... ø ....
De .... À ....

Carottage

Echantillonneur .... ø ....
De .... à ....
Carottier ø ....
De .... à ....
Couronne diamant [ ]
Type :

Equipement Piézomètre ø ......

Crépiné de .... à ....
Plein de .... à ....
[ ] Massif filtrant [ ] Geotextile
[ ] Capot métal [ ] Bouche à clé

Equipement Inclinomètre

.... ø ....
De .... À ....
[ ] Capot métal [ ] Bouche à clé

Arret du sondage

[X] Volontaire
[ ] Incident
[ ] Refus

Essais	Echantillons
[ ] Préssio	[ ] Cuttings
[ ] Phico	[ ] Remanié
[ ] Lefranc	[ ] en caisse
[ ] Lugeon	[ ] intact

SUIVI DE SONDAGE

AZURITE
Sondages Forages Essais SAS

Client : SAGE
Commune : Modane
Sondage : [illegible]C602
Dossier n° :
Equipe :
Date début :
Date fin :
Heure début :
Heure fin :

Réalisation du sondage : Sondage destructif [x] Sondage carotté [ ] Avec enregistrement [x] Sans enregistrement [ ]

Profondeur De	Profondeur à	Echantillons / Cuttings / Carottes — Nature / texture / couleur / odeur / humidité ... — Notes de forage : perte d'injection, ...
0,00	2,30	bloc de schiste.
2,30	13,78	foraine argileuse grise. avec passage de bloc
13,78	19,84	Rocher schiste gris
19,84	26,00	Rocher schiste avec passage altéré
25,50	26,00	Rocher schiste avec veine de Quartz
eau		24/07 : 5,80 / TN
		25/07 : 5,30 / TN

Niveau d'eau en fin de forage :

Fluide de forage
[ ] Eau claire
[x] Injection d'air
[ ] Polycol
[ ] ....

Type d'outils / Réalésage
TFT ø 115 De ... À ...
... ø ... De ... À ...

Type de tubage
Tubage Odex ø 115
De 0,00 À 13,00
Tubage ... ø ...
De ... À ...

Carottage
Echantillonneur ... ø ...
De ... à ...
Carottier ø ...
De ... à ...
Couronne diamant [ ]
Type :

Equipement Piézomètre ø ......
Crépiné de ... à ...
Plein de ... à ...
[ ] Massif filtrant [ ] Geotextile
[ ] Capot métal [ ] Bouche à clé

Equipement Inclinomètre
... ø 76
De 0,00 À 26,00
[x] Capot métal [ ] Bouche à clé

Arret du sondage
[x] Volontaire
[ ] Incident
[ ] Refus

Essais
[ ] Préssio
[ ] Phico
[ ] Lefranc
[ ] Lugéon

Echantillons
[ ] Cuttings
[ ] Remanié
[ ] en caisse
[ ] intact

SUIVI DE SONDAGE

AZURITE — Sondages Forages Essais — SAS

Client : SAGE
Commune : Modane
Sondage : 603
Dossier n° :
Equipe :
Date début :
Date fin :
Heure début :
Heure fin :

Réalisation du sondage : Sondage destructif [X] Sondage carotté [ ] Avec enregistrement [X] Sans enregistrement [ ]

Profondeur De	à	Echantillons / Cuttings / Carottes Nature / texture / couleur / odeur / humidité ... Notes de forage : perte d'injection, ...
0,00	1,57	bloc schiste
1,57	7,08	Moraine grise argileuse
7,08	11,00	Rocher Altéré (schiste).
eau : 28/08 : 9,60		

Niveau d'eau en fin de forage :

Fluide de forage
[ ] Eau claire
[X] Injection d'air
[ ] Polycol
[ ] ...

Type d'outils / Réalésage
... Ø ... De ... À ...
... Ø ... De ... À ...

Type de tubage
Tubage odex Ø 30
De ... À ...
Tubage ... Ø ...
De ... À ...

Carottage
Echantillonneur ... Ø ...
De ... à ...
Carottier Ø ...
De ... à ...
Couronne diamant [ ]
Type :

Equipement Piézomètre Ø ...
Crépiné de 3 à 11
Plein de 0,00 à 3
[X] Massif filtrant [ ] Geotextile
[ ] Capot métal [ ] Bouche à clé

Equipement Inclinomètre
... Ø ...
De ... À ...
[ ] Capot métal [ ] Bouche à clé

Arret du sondage
[X] Volontaire
[ ] Incident
[ ] Refus

Essais
[ ] Préssio
[ ] Phico
[ ] Lefranc
[ ] Lugeon

Echantillons
[ ] Cuttings
[ ] Remanié
[ ] en caisse
[ ] intact

SUIVI DE SONDAGE

AZURITE — Sondages Forages Essais — SAS

Client : SAGE

Commune : Modane

Sondage : 604 PZ — Dossier n° :

Equipe : — Date début : — Date fin : — Heure début : — Heure fin :

Réalisation du sondage : Sondage destructif [ ] Sondage carotté [ ] Avec enregistrement [ ] Sans enregistrement [ ]

Profondeur De	Profondeur à	Echantillons / Cuttings / Carottes Nature / texture / couleur / odeur / humidité ... Notes de forage : perte d'injection, ...
0,00	5,78	bloc et Moraine Remanié
⚠	3,00	vide sur 30 cm
5,78	19,80	Moraine limoneux graveleuse (Marron) et bloc
19,6	25,0	schiste Altéré limoneux gris
06/03		eau à 9,60 m / TN

Niveau d'eau en fin de forage : —

Fluide de forage

[ ] Eau claire
[X] Injection d'air
[ ] Polycol
[ ] ....

Type d'outils / Réalésage

MFC ø 105 De 0,00 À 25,50
.... ø ...... De .... À ....

Type de tubage

Tubage odex ø 90
De .... À ....
Tubage .... ø ....
De .... À ....

Carottage

Echantillonneur .... ø ....
De .... à ....
Carottier ø ....
De .... à ....
Couronne diamant [ ]
Type :

Equipement Piézomètre ø 60

Crépiné de 3 à 17,50
Plein de 0,00 à 3
[ ] Massif filtrant [ ] Geotextile
[ ] Capot métal [ ] Bouche à clé

Equipement Inclinomètre

.... ø Bx
De 0,00 À 25
[ ] Capot métal [ ] Bouche à clé

Arret du sondage

[X] Volontaire
[ ] Incident
[ ] Refus

Essais	Echantillons
[ ] Préssio	[ ] Cuttings
[ ] Phico	~~[ ] Remanié~~
[ ] Lefranc	[ ] en caisse
[ ] Lugeon	[ ] Intact

SUIVI DE SONDAGE

AZURITE
Sondages Forages Essais SAS

Client : SAGE
Commune : Modane
Sondage : 605 PZ
Dossier n° :
Equipe :
Date début : 26/09
Date fin : 27/09
Heure début :
Heure fin :

Réalisation du sondage : Sondage destructif [X] Sondage carotté [ ]
Avec enregistrement [X]
Sans enregistrement [ ]

Profondeur De	Profondeur à	Echantillons / Cuttings / Carottes Nature / texture / couleur / odeur / humidité ... Notes de forage : perte d'injection, ...
0,00	13,30	Moraine avec passage de bloc
13,30	21,00	Rocher schisteux.
eau	le	27/09 : 16 m /TN

Niveau d'eau en fin de forage :

Fluide de forage
[ ] Eau claire
[X] Injection d'air
[ ] Polycol
[ ] ....

Type d'outils / Réalésage
.... Ø ...... De ...... À ......
TFT Ø 92 De 0 À 21

Type de tubage
Tubage .... Ø ....
De .... À ....
Tubage .... Ø ....
De .... À ....

Carottage
Echantillonneur .... Ø ....
De .... à ....
Carottier Ø ....
De .... à ....
Couronne diamant [ ]
Type :

Equipement Piézomètre Ø 76
Crépiné de 15 à 21
Plein de 0,00 à 15
[ ] Massif filtrant [ ] Geotextile
[ ] Capot métal [ ] Bouche à clé

Equipement Inclinomètre
.... Ø ....
De .... À ....
[ ] Capot métal [ ] Bouche à clé

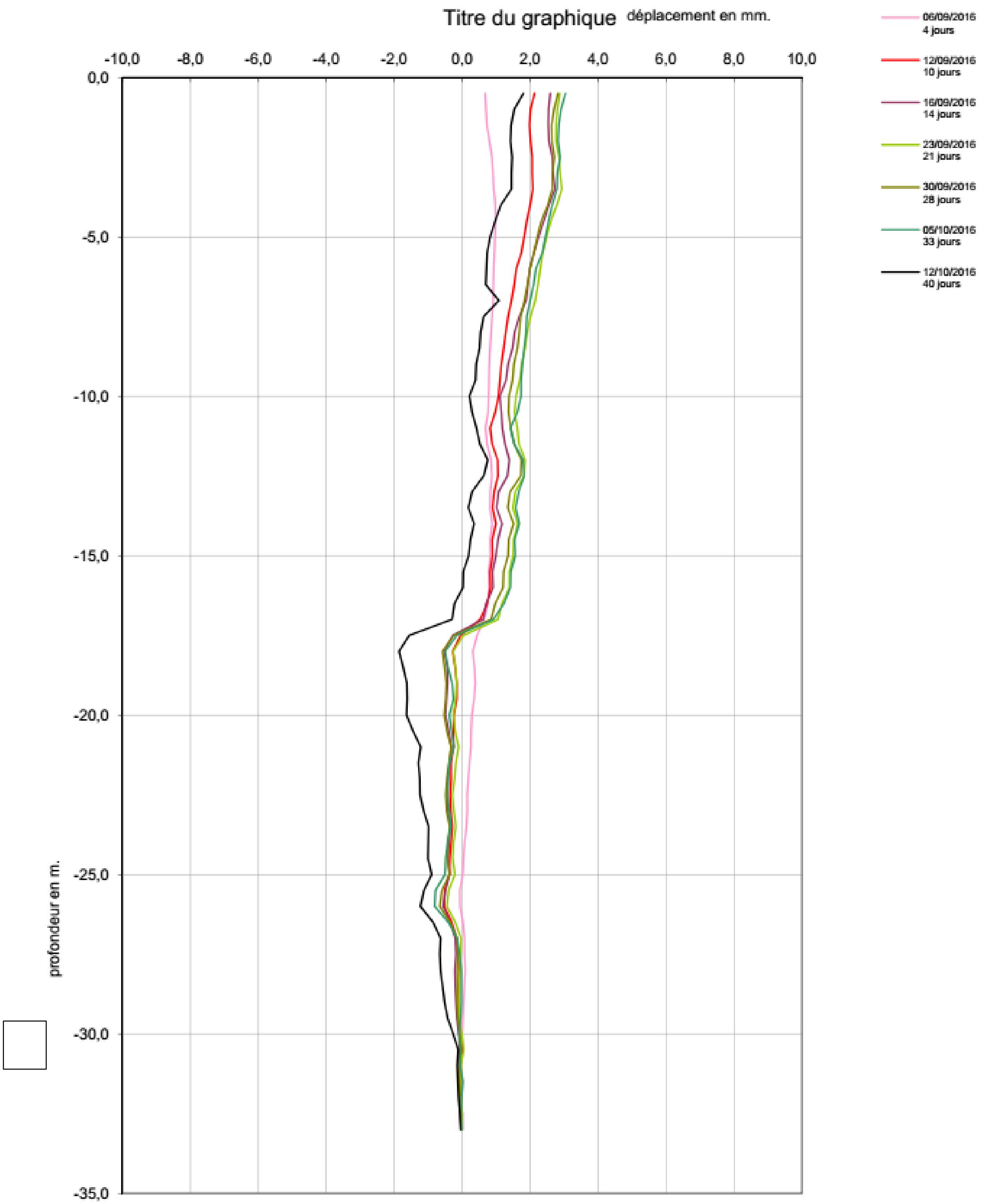
Arret du sondage
[X] Volontaire
[ ] Incident
[ ] Refus

Essais
[ ] Préssio
[ ] Phico
[ ] Lefranc
[ ] Lugeon

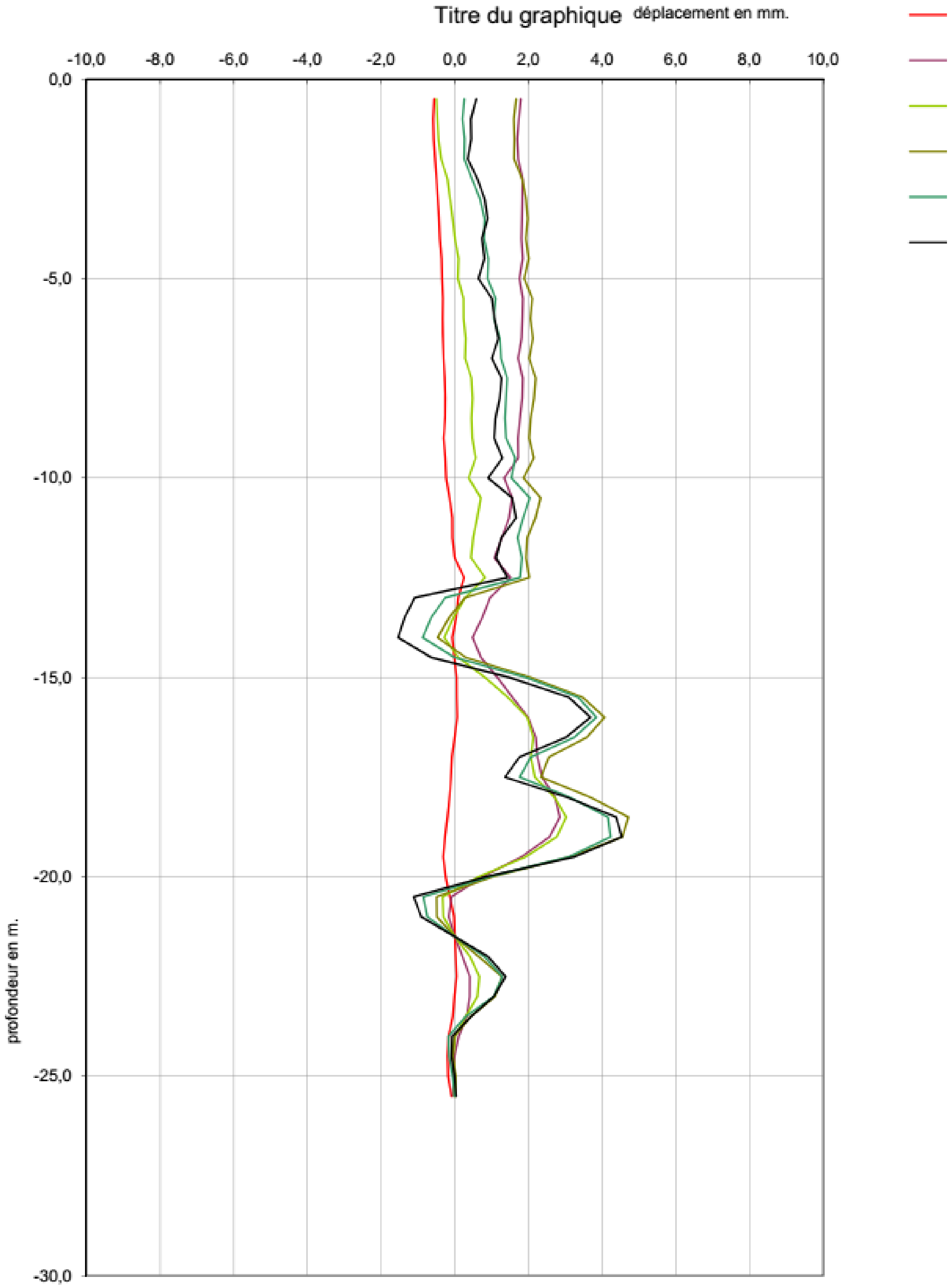
Echantillons
[ ] Cuttings
[ ] Remanié
[ ] en caisse
[ ] intact

Appendix 4: Inclinometer curves (displacements in B- direction)

SITE : Fourneaux (73) Projet : Remplacement viaduc du Charmaix
INCLINOMETRE : 601-1 Code : 601-1
Coordonnées :
Mesure origine 02/09/2016

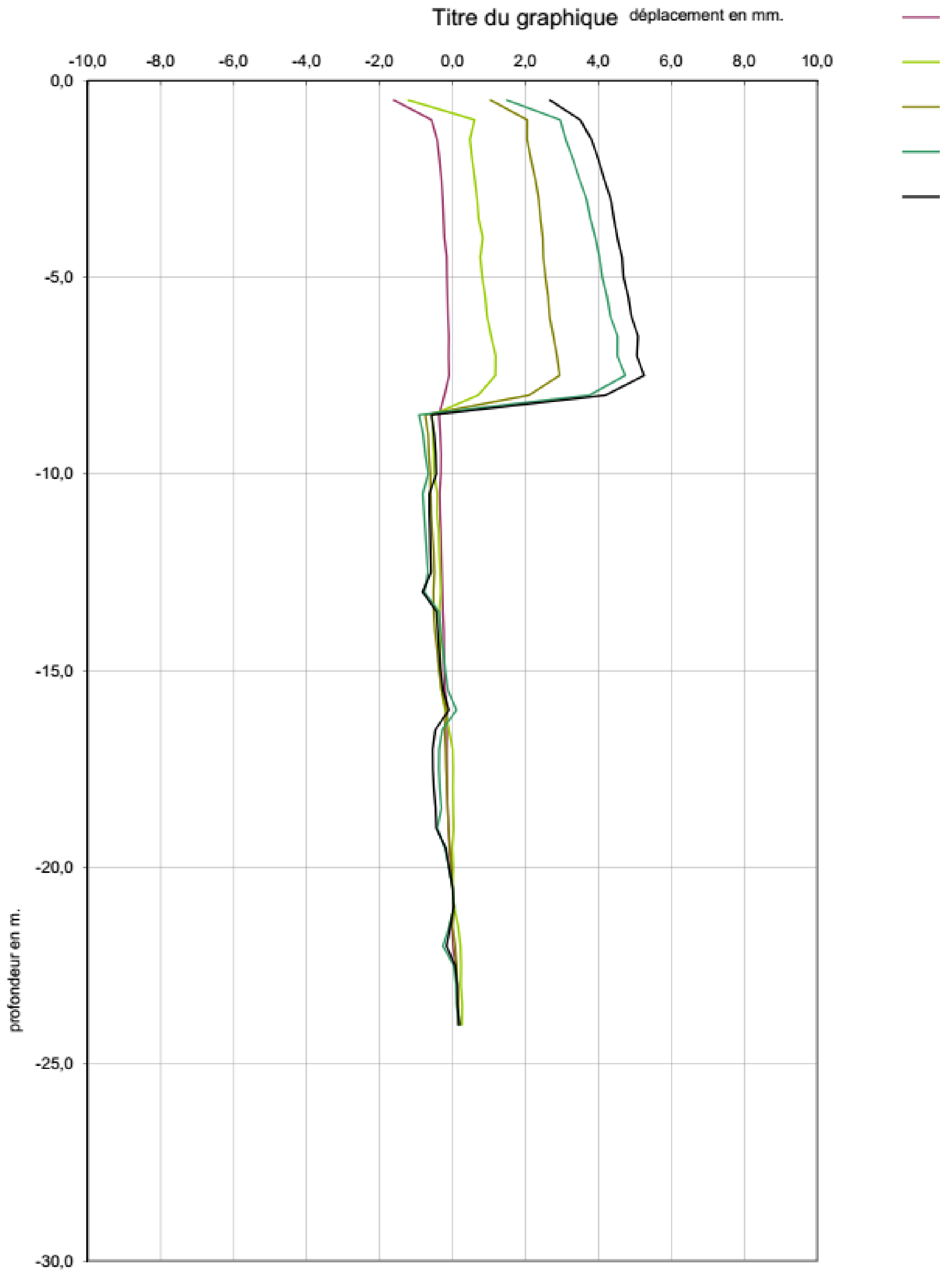


SITE : Fourneaux (73)   Projet : Remplacement viaduc du Charmaix
INCLINOMETRE : 602-1   Code : 602-1
Coordonnées :
Mesure origine 02/09/2016



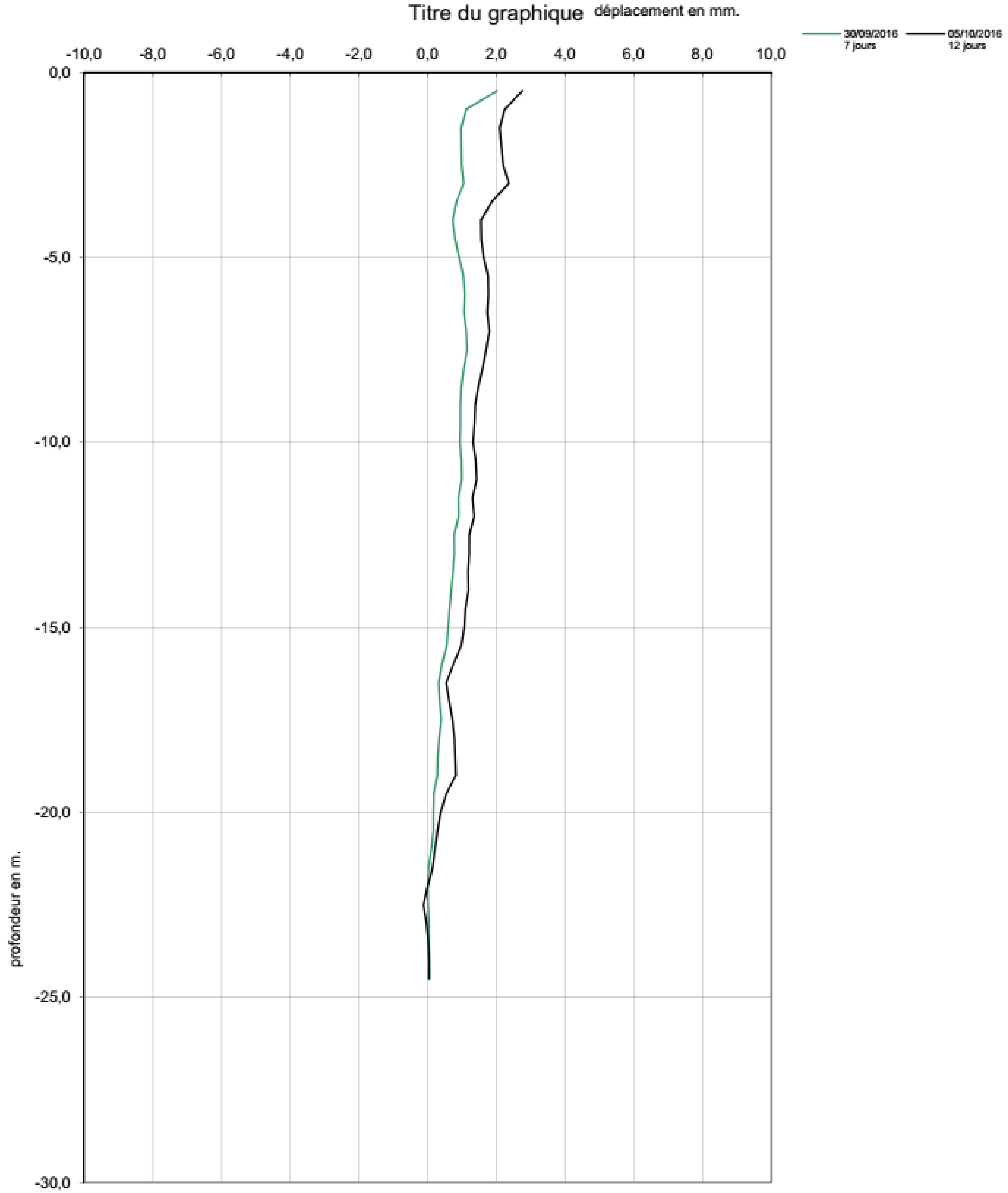
Déplacements corrigés - Direction B

SITE : Fourneaux (73)   Projet : Remplacement viaduc du Charmaix
INCLINOMETRE : 603-1   Code : 603-1
Coordonnées :
Mesure origine 02/09/2016



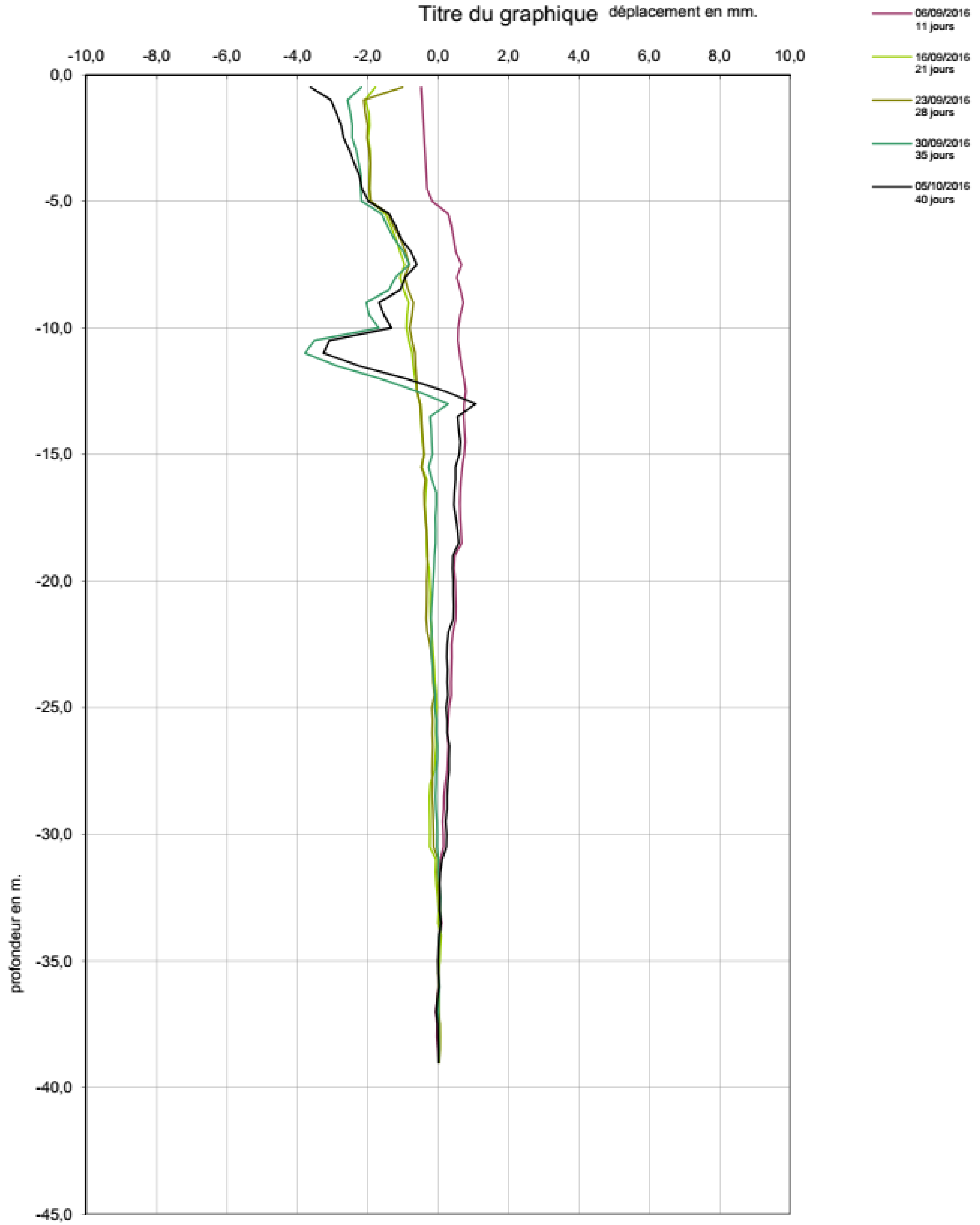
Déplacements corrigés - Direction B

SITE : Fourneaux (73)   Projet : Remplacement viaduc du Charmaix
INCLINOMETRE : 604-1   Code : 604-1
Coordonnées :
Mesure origine 23/09/2016



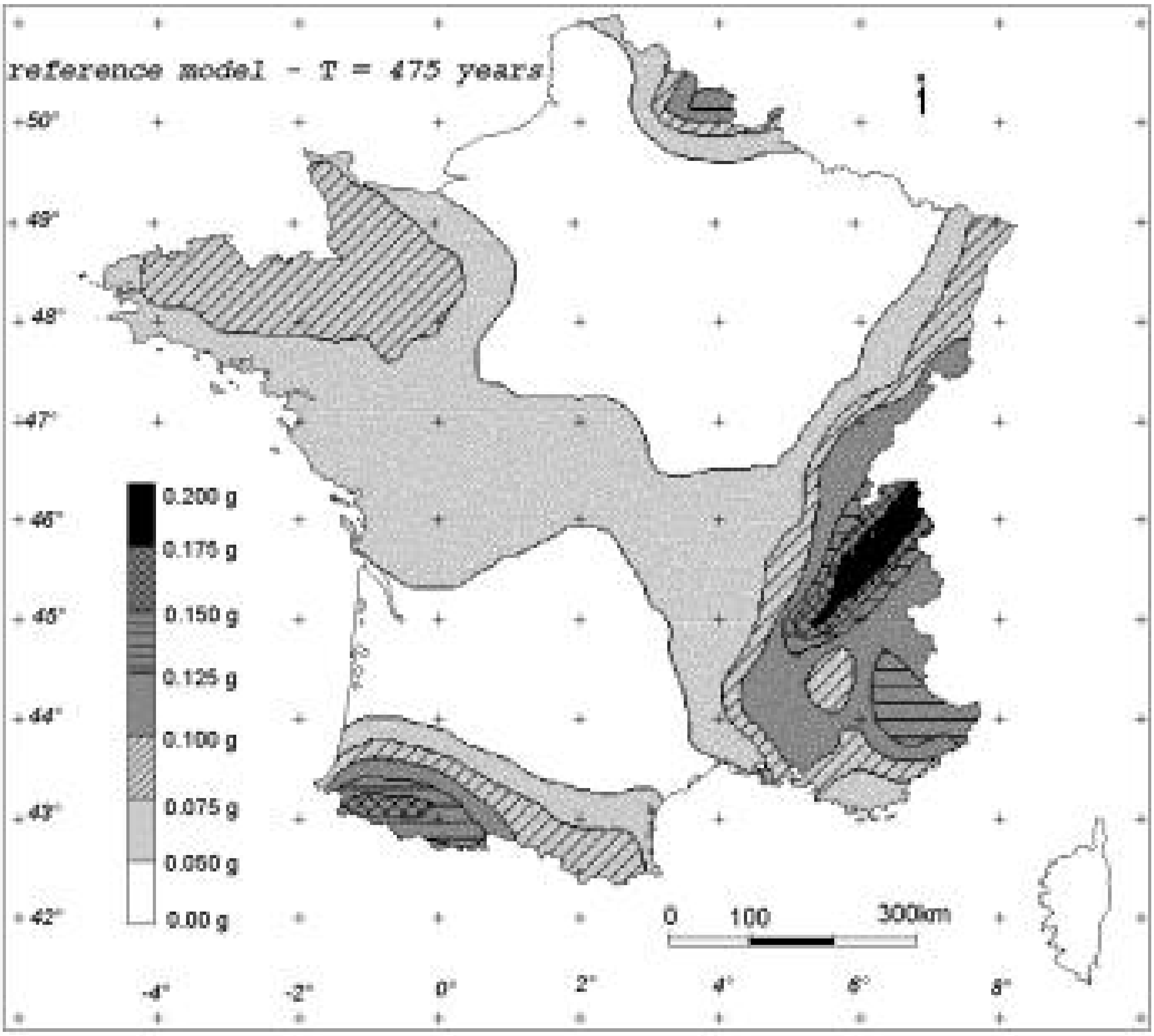
Déplacements corrigés - Direction B

SITE : Fourneaux (73)   Projet : Remplacement viaduc du Charmaix
INCLINOMETRE : 605-1   Code : 605-1
Coordonnées :
Mesure origine 26/08/2016



Déplacements corrigés - Direction B

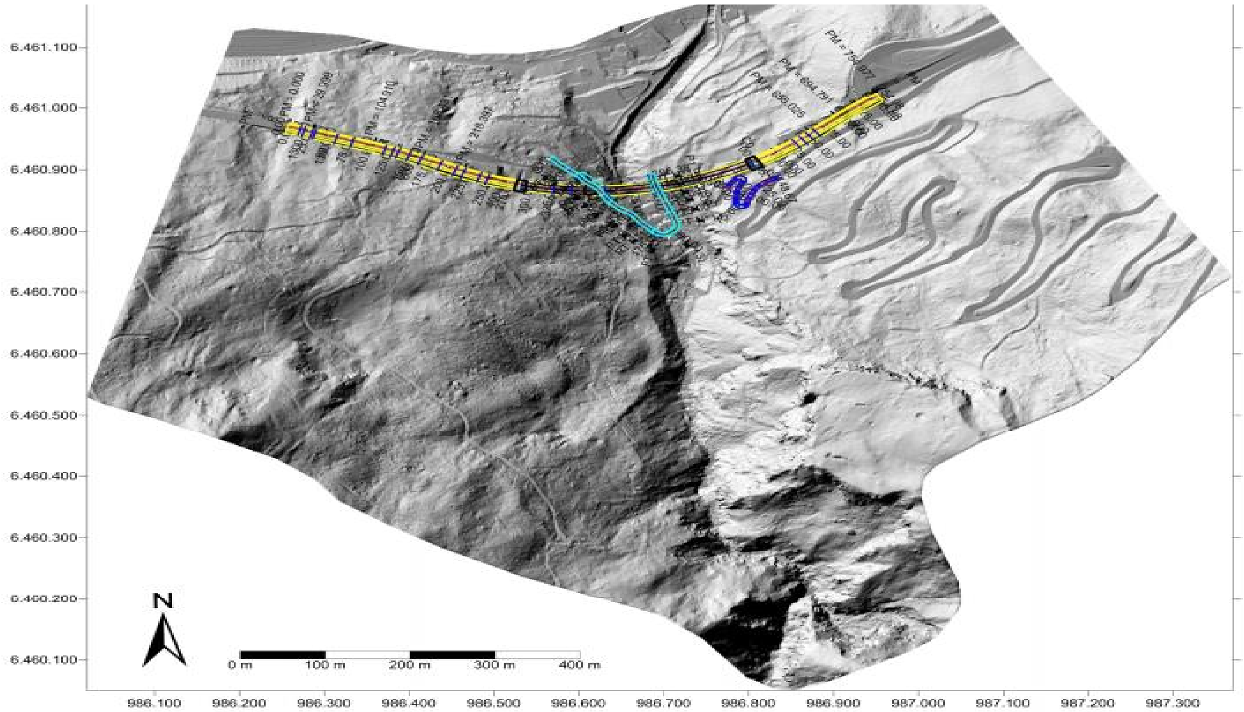
Appendix 5: Map of seismic acceleration isovalues in France



Appendix 6: Tensile strength catalogue for different materials.

Material	Yield strength (MPa)	Ultimate tensile strength (MPa)	Density (g/cm^3)
Steel, structural ASTM A36 steel	250	400–550	7.8
Steel, 1090 mild	247	841	7.58
Chromium-vanadium steel AISI 6150	620	940	7.8
Steel, 2800 Maraging steel[6]	2617	2693	8.00
Steel, AerMet 340[7]	2160	2430	7.86
Steel, Sandvik Sanicro 36Mo logging cable precision wire[8]	1758	2070	8.00
Steel, AISI 4130, water quenched 855 °C (1570 °F), 480 °C (900 °F) temper[9]	951	1110	7.85
Steel, API 5L X65[10]	448	531	7.8
Steel, high strength alloy ASTM A514	690	760	7.8
Acrylic, clear cast sheet (PMMA)[11]	72	87[12]	1.16
High-density polyethylene (HDPE)	26–33	37	0.85
Polypropylene	12–43	19.7–80	0.91
Steel, stainless AISI 302 – cold-rolled	520[citation needed]	860	8.19
Cast iron 4.5% C, ASTM A-48	130	200	7.3
"Liquidmetal" alloy[citation needed]	1723	550–1600	6.1
Beryllium[13] 99.9% Be	345	448	1.84
Aluminium alloy[14] 2014-T6	414	483	2.8

Appendix 7: LIDAR images



3 grands types d'anomalies :

- Zone d'éboulis ou de remblais
- Zones glissées avérés
- Zone de glissement probable