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Geotechnical state-of-the-art in Guatemala -- Ground stabilization

Fernando Rafael Callejas Benitez

Department: Civil, Architectural and Environmental Engineering

GEOTECHNICAL STATE-OF-THE-ART IN GUATEMALA – GROUND STABILIZATION

by

FERNANDO RAFAEL CALLEJAS BENITEZ

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Approved by:

Ronaldo Luna, Advisor

Neil Anderson

Bate Bate

David Rogers

Rodolfo Semrau

Richard Stephenson

ABSTRACT

History provides an understanding of the present and in a way, a guide for the future. The state-of-the-art is often referred to as a snapshot of the most significant works and contributions made in a field of study; In this case the focus is on Guatemalan geotechnical engineering. This study presents a comprehensive description of the different events that impulse geotechnical engineering in Guatemala.

As part of the description of the state-of-the-practice an investigation of the different human (engineers/contractors) and physical resources (laboratories/field equipment) and their capabilities are presented. An overview of the civil engineering programs is presented. Then, the advances in the graduate program in geotechnical engineering are discussed.

Select comprehensive case studies are presented. Project selection was based on relevance and project's interest for geotechnical engineers. Privileged data and information, such as design, construction, and performance data are presented, giving a full range of point of views of the selected projects. Selected projects include topics such as slope stabilization, ground improvement, liquefaction, deep excavations, dams, grouting and foundations. A perspective on the quality of the solutions adopted in tropical and volcanic areas is discussed. For each case study, a review of the degree at which geotechnical engineering processes were followed: subsurface investigation, analytical or computational tools, empirical relationships, field testing, and/or measurement of performance (monitoring behavior). The study concludes by identifying the lessons learned; areas of improvement and recommendations in the different fields of education, resources, and practice.

ACKNOWLEDGMENTS

Life is a journey, this part of the journey could be call the pursuit of a dream. I would like to start giving thanks to God for giving me the opportunity of complete this part of the journey. I would like to thank my wife and two princesses for giving me their love, support and patience during this time. I would like to express my gratitude to my advisor, Dr. Ronaldo Luna, whose expertise, understanding, patience, and dedication transformed this dream in a reality. I appreciate his knowledge, guidance and assistance during the whole process. I would like to thank the other members of my committee, Dr. Neil Anderson, Dr. Bate Bate, Dr. David Rogers, Dr. Rodolfo Semrau, and Dr. Richard Stephenson for their assistance and collaboration. I would like to thank to Jeannie Werner for her help and terrific attitude. I would also like to thank to Rodolfo Alvarado for his help and knowledge share, a prayer for you Rodolfo. Finally, I would like to thank José García who gave me inspiration and support to start this adventure.

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1. INTRODUCTION

1.1. SIGNIFICANCE

The significance of this work revolves around two main goals framed by the author. First, the development of the geotechnical engineering trade in Guatemala is examined from its inception to the current state of the art. This places in the right context a series of geotechnical engineering case studies in the same country. The second goal is to present a detailed description of three select case studies of industrial and civil infrastructure projects. These projects were unique in that access to detailed information was made available to the author, including performance monitoring of the constructed infrastructure. It is the hope of the author that this manuscript serves the scholars and engineers interested in the progress made in geotechnical engineering in Guatemala and the possibilities to improve current practice.

1.2. MOTIVATION

The author has taken the liberty of writing this section in first person, as this manuscript was a motivated by his personal experiences in his professional career.

During my early years as a practicing engineer I noticed a knowledge gap in Guatemalan geotechnical engineering practice. Guatemala has a very limited number of geotechnical professionals and related resources. At that time, I had the opportunity to study and engage in an internship in Spain, which gave me an overview of the geotechnical advances and standard of practice at a global level. I realized that there was a need to improve the current geotechnical engineering practice in my country, Guatemala. Once I had identified this gap in the profession, I saw it as a great opportunity for professional development and improvement of the geotechnical trade. I started this pursuit for improvement by establishing the Guatemalan Society of Soil Mechanics and Geotechnical Engineering or “Asociación Guatemalteca de Mecánica de Suelos e Ingeniería Geotécnica” (AMSIG) as the founding president. I have been an instructor at the Universidad del Valle de Guatemala and I noticed that is important to teach students about engineering good practices and not just the technical concepts and skills, but also a complete perspective of responsibility, duties, and ethics. I also noticed the importance of sharing and recording the engineering history that we may get loose when each great engineer retires or passes away. It is from this perspective that I have developed my personal goals, which guided my research with a particular significance for the future of geotechnical engineering in Guatemala.

1.3. RESEARCH OBJECTIVES

This research has two general goals, one is to contribute to Guatemalan engineering practice, providing a historical framework of geotechnical engineering practice, and to seek suggestions for improvement in the development of geotechnical engineering and geo-professionals for the future. The second objective is to present three (3) select case studies related to the performance of ground stabilization in Guatemala. The specific objectives of this study are the following:

1. Present a historic background of geotechnical engineering practice in Guatemala.
2. Examine the current state of the practice as well as the educational opportunities for geoprofessionals.
3. Present case studies of the performance of ground stabilization techniques through actual projects in Guatemala. The selected case studies are:
   a) Soil Nailing Walls Performance in Guatemala City Volcanic Soils
   b) Performance of Grouting Intensity Method, GIN, for a Cutoff Curtain for Santa Teresa Dam in Guatemala.
   c) Seismic Ground Improvement: Stone Columns Performance for a Power Plant in the Southern Alluvial Plains of Guatemala.

4) Provide recommendations and guides for the development of the geotechnical profession in Guatemala.

1.4. DISSERTATION ORGANIZATION

This DE dissertation is organized in six chapters. This Chapter 1 presents an introduction with the significance, motivation, goals and objective of this research. Chapter 2 is the state-of-the-art of geotechnical engineering in Guatemala, with a geologic and historical overview of the geotechnical developments, including the educational and professional resources. Chapter 3 is a literature review of the geotechnical techniques used in the case studies. Chapter 4 presents three select case studies located in different geologic regions. The case studies are all related to ground stabilization: (1) highway underpass and slope cut, (2) hydro-power dam, and (3) power plant. Finally, the conclusions and recommendations are presented in Chapters 5 and 6, respectively. The appendix includes a directory of the current geo-professionals in Guatemala.

2. GEOTECHNICAL STATE-OF-THE-ART IN GUATEMALA

2.1. GUATEMALA GEOLOGIC AND SOIL CONTEXT

Guatemala is located on the Northern edge of Central America. Guatemala has a complex geological and geotechnical setting varying from alluvial plains in the pacific coast passing through volcanic formations in the highlands to thin layer of clay derived of siltstone weathering. Ground materials suffered different weathering processes due to its tropical area marked by two seasons, dry and rainy seasons.

The Guatemala seismic scenario is unique, with three tectonic plates converging, producing several seismic sources with a variety of movements, faults and rupture mechanisms. The overview of Central America tectonic scenario is well represented in Figure 2.1 (Mann, et al, 2007).

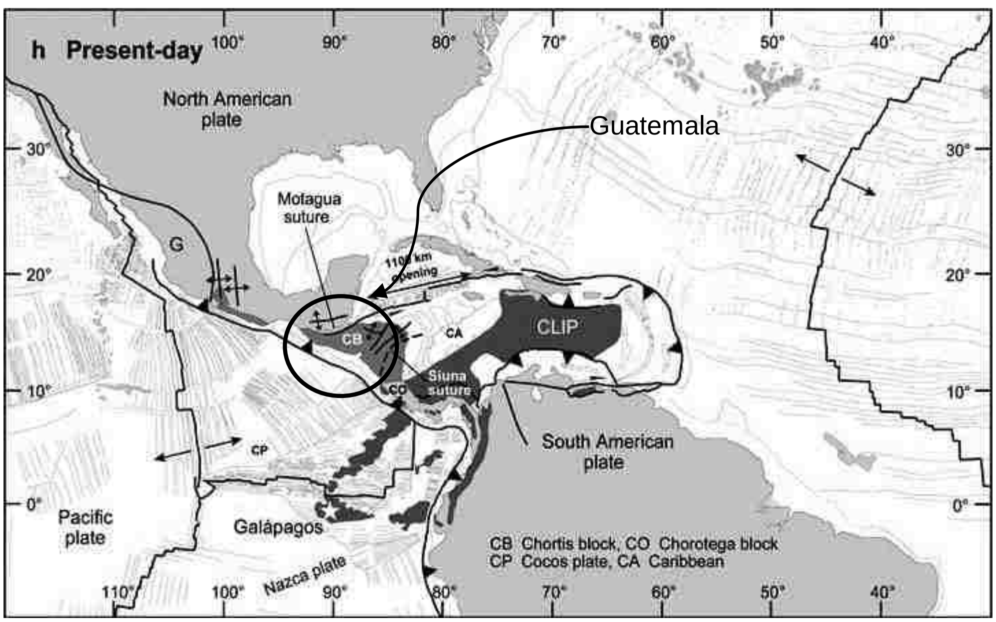


Figure 2.1. Central America tectonic setting (Mann et al, 2007)

The geological map of Guatemala, shown in Figure 2.2 (Weyl, 1980) includes the location of the main faults and volcanoes. Volcanism is a result of the subduction areas from the Pacific coast. The Pacific coast of Guatemala is part of the “Ring of fire” with 26 volcanoes, including three active volcanos named Santiaguito, Pacaya and Fuego.

The transcurrent fault system Motagua-Polochic is located on the map been one of the main seismic sources.

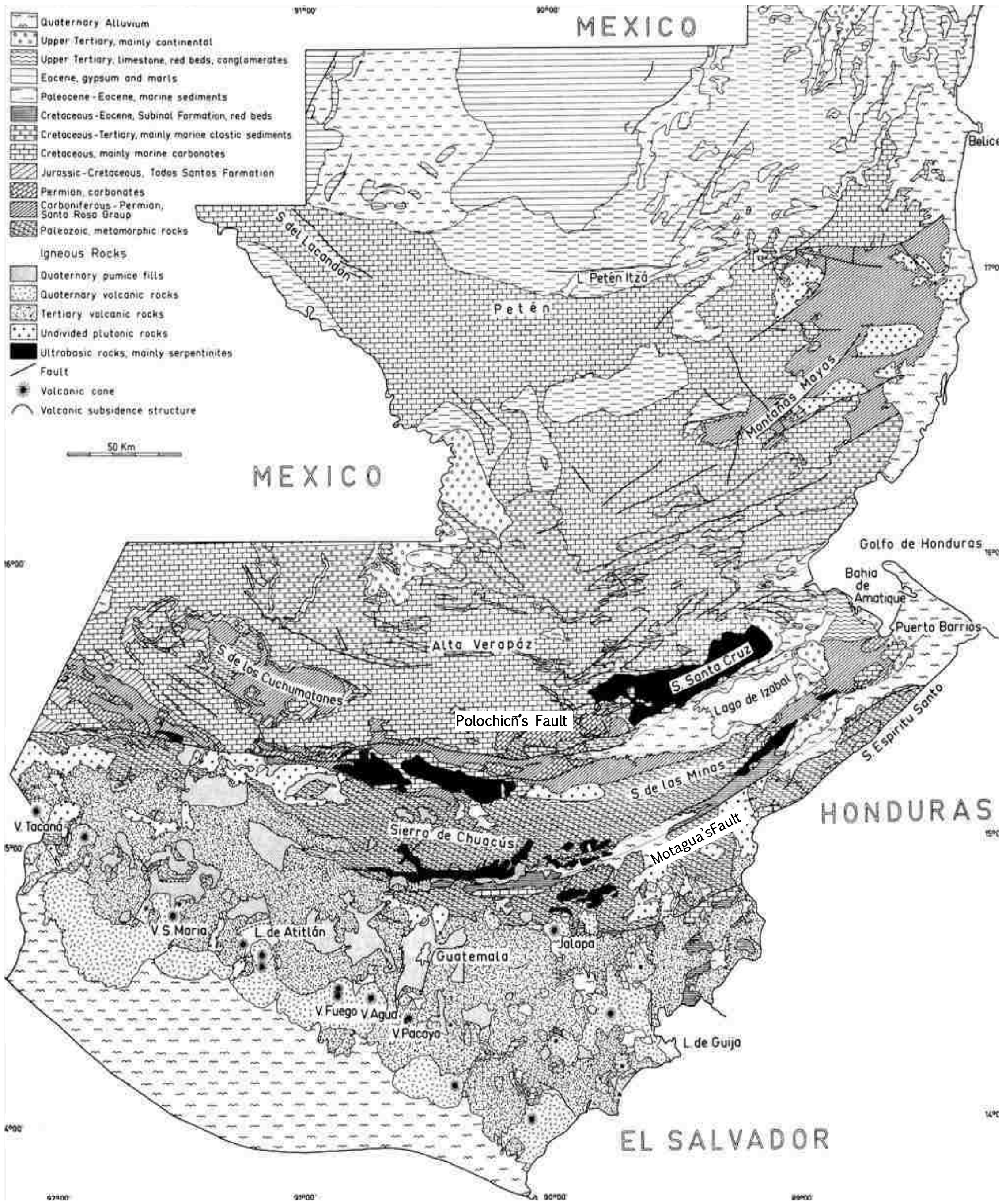


Figure 2.2. Geological map of Guatemala (Weyl, 1980)

The particular tectonic plate setting for Guatemala includes subduction in the pacific border, where the Cocos plate is subducting below the Caribbean plate. The arrows in Figure 2.2 show the subduction direction along the border of two plates. Also, the tectonic setting includes a transcurrent fault system, Motagua-Polchic, between North American plate and the Caribbean plate. Most likely as result of these two movements, transcurrent and subduction, there are several faults and ruptures areas within Guatemala.

2.1.1. Overview of Guatemala Seismicity History. Historical records of Guatemala's seismicity begins in the Colonial period in 1538, with the Catholic Church records. This records describe different effects and damage produced by ground motions. From these descriptions, the intensity and moment magnitude of the events can be inferred (White, 1984); a summary of this information is presented in Table 2.1.

Table 2.1. Historical seismicity records

Event Date	Location	Magnitude (M)	Event Date	Location	Magnitude
1538	Alta Verapaz	3 to 5	1942 August 2	Alta Verapaz, Quiche	M_L 7.9
1590	Alta Verapaz	3 to 5	1959 February 20	Alta Verapaz, Quiche	M_L 6.5
1733	Chiquimula	6.7 to 7.2	1976 February 4	Motagua Valley	M_w 7.5
1785 Jan 6	Alta, Baja Verapaz, Izabal	7.3 to 7.5	1991 September 18	Chimaltenango	M_w 5.3
1816 July 22	Verapaz to Chiapas	7.5 to 7.7	2007 June 13	Pacific Coast	M_w 6.7
1820 June 6	Baja Verapaz	5.5 to 6.5	2012 November 7	Pacific Coast	M_w 7.4

Where M is Richter scale magnitude, M_w is moment magnitude and M_L is local magnitude.

The 1976s earthquake is listed as one of the world's strongest ground motion (Douglas, 2001), mostly due to its dead toll.

The geological and seismic scenario presents a complex and challenging situation for geotechnical and civil engineering.

2.2. HISTORY OF GEOTECHNICAL AND GEOLOGICAL ENGINEERING IN GUATEMALA

Formal geologic and geotechnical engineering education and practice in Guatemala has a relative brief history. Transcendental events as international cooperation programs, natural disasters or national necessities are milestones that changed the way of the engineering in the country. These events can be grouped in three areas, the modern geotechnical engineering practices that include the (1) influence of contractors and professional in the development of the practice, (2) the 1976 Guatemala earthquake and the (3) contemporary and most relevant engineering projects in the country.

2.2.1. Modern Geotechnical Engineering Practice. The modern history of Guatemala's geotechnical engineering could start with 1928 Karl von Terzaghi visit, but this was just a scouting visit for him. The beginning of formal geotechnical engineering practice goes back to 1955 when Tippetts, Abbett, McCarthy and Stratton, (TAMS) as part of an agreement with Guatemala's government, sent personnel to develop capabilities in different areas. This event changed the practice of engineering in Guatemala particularly for geotechnical engineering and it's the start of a walk through human and natural events that marked the history for this engineering branch in Guatemala. As part of the crew, Professor John Barber from University of Maryland trained a young Guatemalan engineer named Roberto Lou. The training consisted of basic field investigation techniques including drilling, sampling, and compaction testing. This simple training motivated Rorberto Lou to study geotechnical engineering, travelling to Birmingham, England in order to earn a master degree in geotechnical engineering, Figure 2.3 shows Roberto Lou beside a drill rig. He became the first real geotechnical engineer per se of Guatemala. In the late 60's, Armando Lopez attended the University of California, Berkeley and Federico Koose took some courses at the L'Ecole Polytechnique-Université Paris-Saclay, France. They came back to install the first private laboratories and geotechnical engineering consultant firms in Guatemala.

In the late 70s Dr. Rodolfo Semrau earned his PhD degree from Northwestern University and then joined to the geotechnical engineering community. Since that time to early 2000s the geotechnical community have very few changes and incorporations as Rodolfo Hermosilla with a some courses in soil mechanics in Harvard University, Carlos Cordon with a Master degree from Rensselaer Polytechnic Institute, Franklin Matdzdorf from Georgia Institute of Technology and Daniel Gonzalez a local educated engineer partner of Carlos Cordon. In the late 1990s and early 2000s the geotechnical community suffered a very drastic change with the death of Armando Lopez in 1999, Daniel Gonzalez also in 1999 and Federico Koose in 2001. The space left by this three professionals was occupied partially by Dr. Semrau, opening a huge gap that have been fill by young professionals, mostly civil engineers without formal training.

The other main influence in geotechnical engineering practice was the participation of geotechnical contractors. The geotechnical construction was dominated by foreign companies since the establishment of Swissboring Overseas Corp. Ltd. in 1961.

Figure 2.3. Roberto Lou beside a drill ring in 1963, Lake Atittlán, Guatemala

The majority of complex projects such as, deep foundations, marine foundations, grouting, diaphragm walls, anchors, ground improvement, and even large geotechnical investigation campaigns were usually performed by foreign companies but mostly by Swissboring. Swissboring initially settled in the Central America region in El Salvador in 1959 and then moved in 1961 to Guatemala to work in the drilling and grouting campaign for Los Esclavos and Jurun Marinala dams. In the beginning, Swissboring was devoted to carry out soil and rock exploration drilling and grouting works for hydroelectric power plants. In the first years in Central America, their experienced technical and administrative personnel were from Spain. During the 70's and 80's the company had an important participation in the construction of the main hydroelectric projects in Central America. In the 90´s, the company extended its activities widely to other fields of civil and geotechnical engineering, participating in foundation works with piles and micropiles, slope stability works with tie-back, soil nailing and diaphragm walls, mineral exploration, geothermal exploration and marine works (Rosenberg, 2010).

The presence of the well trained Spanish/European personnel of Swissboring allowed the knowledge transfer to local engineers, as well as the implementation of new techniques with new equipment. Some of the most important milestones of the geotechnical practice were performed by Swissboring and are covered in the project section of this work. In 2001 the former general manager of Swissboring started his own company as the first formal local competitor to Swissboring. Since then, several local contractors have entered the market such as: Geocimsa, Soiltec, Soluciones Tecnicas de Ingeniería, (STI) Pilotecmar, Prodecsa and Geocon.

2.2.2. The 1976 Guatemala Earthquake. According to the National Oceanic and Atmospheric Administration, (NOAA) database, the 1976 Guatemala earthquake struck on February 4 at 03:01:43 local time with a moment magnitude, Mw, of 7.5. The shock was centered on the Motagua Fault, about 160 km northeast of Guatemala City at a depth of 5 kilometers (3.1 mi) near the town of Los Amates in the department of Izabal.

Many cities throughout the country suffered damage, and most adobe type dwellings in the outlying areas of Guatemala City were destroyed. The earthquake struck during the early morning (at 3:01 am, local time) when most people were asleep. This contributed to the high death toll of 23,000 and approximately 76,000 injured, and many thousands left homeless. Many areas went without electricity and communications for days. The main shock was followed by thousands of aftershocks, some of the larger ones causing additional damage and loss of life (United States Geological Services, 2016).

The most heavily affected area covered some 30,000 km², with a population of 2.5 million. Approximately 258,000 houses were destroyed, leaving about 1.2 million people homeless. 40% of the national hospital infrastructure was destroyed, while other health facilities also suffered substantial damage (Olcese, et al., 1977).

Several geotechnical related failures were observed as landslides, settlements and liquefaction, Figures 2.4 to 2.10 illustrates some of the most critical and representative failures occurred during the earthquake.

The Figure 2.10. One of many large cracks in a delta at Lake Amatitlan (20 kilometers south of Guatemala City) opened as a result of Earthquake-induced liquefaction of a near surface layer of saturated pumice sand and lateral spreading of the surficial deposits towards the lake. Such cracks caused serious damage where they intersected structures such as the one in the foreground. The front portion of the house in the background sank into the liquefied sand, tilting the brick chimney. (Plafker, 1977)

Figure 2.4. One of many landslides blocking the main highway from Guatemala City to El Progreso (Plafker, 1977)

Figure 2.5. Landslide along the edge of a steep walled valley in Guatemala City (Plafker, 1977)

Figure 2.6. Landslides in steep roadcut of stratified pumice and ash deposits at San Cristobal (Plafker, 1977)

Figure 2.7. Sand mound deposited by a sand blow (Plafker, 1977)

Figure 2.8. Landslides and extensive headwall cracks developed along the edge of a steep walled valley in a Guatemala City suburb (Plafker, 1977)

Figure 2.9. Puerto Barrios wharf, destroyed by the February 4 earthquake. Arrows point to the large warehouse partially submerged ((Plafker, 1977)

Figure 2.10. Cracks at Lake Amatitlan (Plafker, 1977)

This event changed the practice of engineering in Guatemala creating awareness about seismic and construction risks. Since then the concern about construction site assessment increased fostering the use of geotechnical engineering services. Despite all the damage no regulatory advance were observed until 2005 when a guidelines for geotechnical investigation, Guía para dictámenes geotécnicos, PE-01-2005, published by Colegio de Ingenieros de Guatemala (CIG) and Asociación Guatemalteca de Ingeniería Estructural y Sísmica (AGIES), were issued.

The next advance in the engineering practice regulatory frame was in 2010 when the recommend construction and design coded, "Normas Recomendas", (NR) published by AGIES were officially approved as mandatory construction code, Code of Structural Safety for Building and Infrastructure Projects, (NSE). The code content is presented in the Table 2.2.

Table 2.2. Code of structural safety for building and infrastructure projects content

Chapter	Description
NSE 1	General Notes, Code Administration and Use, and Technical Supervision
NSE 2	Structural Demands, Site Conditions and Protection Levels.
NSE 2.1	Geotechnical Site Characterization and Site Assessment
NSE 3	Buildings Structural Design
NSE 3.1	Structural Design for Standard Use Buildings
NSE 3.2	Structural Design for Special Buildings
NSE 4	Housing and One and Two Story Buildings Requirements
NSE 5	Infrastructure and Special Projects Requirements
NSE 6	Existing Facilities Requirements: Risk Reduction, Assessment and Retrofit.
NSE 7.1	Reinforced Concrete
NSE 7.4	Reinforced Masonry

The author have the opportunity to collaborate in both publication.

Other earthquakes and natural disasters as hurricanes and tropical storms have hit Guatemala but none had have the impact of 1976 earthquake.

2.2.3. Relevant Engineering Projects since 1955. Many relevant projects have been performed in Guatemala since 1955, the projects presented herein were selected based on their overall relevance and particularly geotechnical relevance. The first project is Guatemala City Government Center, Centro Civico, is an office complex composed of several buildings, its description is presented in the Table 2.3.

The most important features of this complex are:

- The largest government center in Central America.
- The tallest steel building in Guatemala, Ministerio de Finanzas Publicas.
- The largest basement for that time, Corte Suprema de Justicia.
- The majority of the complex survived the 1976 without collapse, most of the facades were damaged.
- The excavations were performed without any special protection.
- The building have a maximum of two basement levels.
- The in many cases the level differences were handle using slopes.
- The retaining walls majority were built with using cantilever concrete walls.
- The buildings used direct foundations, isolated footings.

Table 2.3. Buildings of Guatemala Civic Center

Building	Architects / Engineers	Construction Period
Municipalidad de Guatemala	Pelayo Llarena and Roberto Aycinena	1954 to 1958
Instituto Guatemalteco de Seguridad Social	Jorge Montes and Roberto Aycinena	1956 to 1959
Crédito Hipotecario Nacional	Carlos Haeusler	1961 to 1965
Banco de Guatemala	Jorge Montes and Raúl Minondo	1962 to 1966
Centro Cultural de Guatemala, Miguel Ángel Asturias	Efraín Recinos	1961 to 1978

Table 2.3 Buildings of Guatemala Civic Center (cont.)

Ministerio de Finanzas Publicas	René Minera Pérez	1973 to 1977
Corte Suprema de Justicia	Mario Flores Ortiz and Associates	1974 to 1976
Instituto Guatemalteco de Turismo	José María García de Paredes and Antonio Sandoval Martínez y Urrutia	1974 to 1977

(Asociación Amigos del Pais, 2014)

The Figure 2.11 shows images of the construction of Credito Hipotecario building, Figure 2.12 shows Corte Suprema de Justicia building excavation.

At the same period of this constructions the energy infrastructure was also developed. Energy infrastructure was the sole responsibility of the government until 1992, so all the projects until that date were developed and managed by the Instituto de Electrificación (INDE).

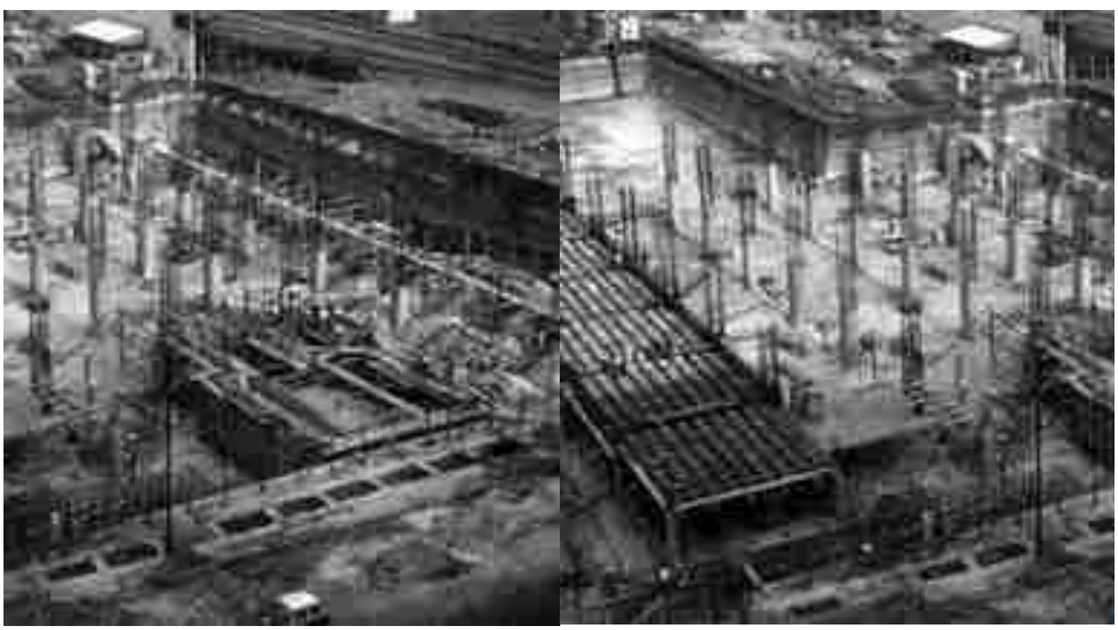


Figure 2.11. Construction of Credito Hipotecario building (Gatonelblu, 2010)

Figure 2.12. Excavation of Corte Suprema de Justicia building (Gatonelblu, 2010)

According to the Comisión Nacional de Energía Electrica, (CNEE), Guatemala National Electricity Board, Los Esclavos dam was completed in1966 with a power of 15 MW, 1.34 km channel and 175.00 m of pressure pipe. This was the first dam that implemented a grouting curtain for seepage control, also the largest hydro electrical project at that time. In the same period Jurun Marinala dam was completed by1970 with a power of 60 MW, a 4.03 km of pressure tunnel and 2.44 km of pressure pipe. This project became the largest hydro electrical project of Guatemala also the first with a conduction tunnel. For the tunnel, a consolidation and filling grouting campaign was performed, also a grouting curtain for seepage control was performed. The next relevant project was Aguacapa dam, completed in 1982 with a power of 79 MW, a 12.04 km of pressure tunnel and 3.65 km of pressure pipe. This project became the largest hydro electrical project of Guatemala and also the one with the largest conduction tunnel. The last hydro-electric INDE project of INDE was Chixoy dam, completed in 1983 with a power of 280 MW, a 25.482 km of pressure tunnel and a dam height of 110.00 m. This project became the largest hydro-electric project of Central America for more than 30 years. Chixoy was the first project that used a Tunnel Boring Machine, (TBM), this fact has a particularly interesting history. The tunnel construction started with a TBM but due to ground karstic formations it got stuck in a sinkhole and was abandoned due to the impossibility of get it out of the sinkhole. Finally the tunnel cover was poured over the TBM. For the tunnel, an extensive consolidation and filling grouting campaign was performed, also a grouting curtain for seepage control was performed in the dam site. Also, post-tensioned anchors were built for first time in Guatemala in order to stabilize power house slope. The construction was performed by several companies as Impregilo S.p.A. (former Cogefar S.p.A.) from Italy and Hochtief Aktiengesellschaft from Germany.

A milestone for geotechnical engineering in Guatemala was the construction of the Arizona Power Plant, where two ground improvement techniques were used for first time. The project is locate near Puerto Quetzal in the alluvial plains of the Pacific coast prone to a high seismic risk. Its construction started in 2002 using a direct foundation solution, after the floor slab was poured significant settlements were observed. In order to improve the ground condition, initially a High Dynamic Compaction and e-quake drains treatment were proposed and implemented. This was the first time that high dynamic compaction have been used in Guatemala, and the treatment was performed by ITSA a local contractor, Figure 2.13 shows the site after high dynamic compaction performance and before e-quake drains installation. This treatment did not improved the deep ground against liquefaction risk, so an alternative ground improvement solution was adopted. The selected alternative was Vibroreplacement, Stone Columns, which was also the first time used in Guatemala. The solution consisted of 13.00 m depth stone columns and was performed by Hayward Baker the American subsidiary of the British company Keller.

The Santa Teresa dam part of the Hydro-electric power project in 2010 brought a new era in the performance of grouting in Guatemala. In order to control seepage under the dam a grouting curtain was installed. It included the implementation of Grouting Intensity Number method, (GIN) also the first time that this method was used in Guatemala, Figure 2.14 shows the dam construction.

Figure 2.13. Construction site of Arizona Power Plant

Figure 2.14. Construction site of Santa Teresa Dam

Puerto Quetzal is the most important marine project performed since 1955, according with the web site of Empresa Portuaria Quetzal. It was built from 1980 to 1983 by the French company Dragages Et Travaux Publics. The wharf consists of a sheet pile wall with a concrete slab in the top. It was the first time that a structure of this type was built in Guatemala. For its construction a small railroad was built in order to ease concrete blocks and rocks transportation. The other relevant projects are San Jose docking station and Barcaza Man Power Station this project are part of thermal electric power plants. Both were the first to use large diameter steel driven piles, 1.50 m. Both were performed from 1999 to 2000 in Puerto Quetzal harbor area by Swissboring Overseas Corp. Ltd. Figure 2.15 shows the handling of the steel piles of Barcaza Man and Figure 2.16 show the pile driving process of the steel pipe piles of San Jose Docking station.

Figure 2.15. Steel piles of Barcaza Man

Figure 2.16. Construction of San Jose Docking Station

The new era of vertical building construction was set by the construction of Centro Gerencial Las Margaritas, built in 1993. This started the era of deep basements with 5 levels with a 17.50 m depth excavation. The project is a milestone in the Guatemala geotechnical engineering because it was the first deep excavation stabilized with soil nailing walls. The walls were performed by Swissboring Overseas Corp. Ltd. Also during the construction of Centro Gerencial Las Margaritas the pullout test for soil nailing nails was carried out. The deepest excavations in Guatemala are Zona Pradera with 7 basement levels and 27.50 m depth, stabilized with temporary soil nailing walls. Meanwhile Torre Real is the deepest excavation with a permanent wall, it is stabilized with a mix of post-tensioned anchors and soil nailing walls with a depth of 24.50 m. Both were performed by Rodio-Swissboring (former Swissboring Overseas Corp. Ltd.). The first top-down structure, by means of columns and structural slabs were constructed prior to soil removal, was at the parking lot of Montufar shopping center. The upper slab is supported over piles that late works as columns when the soil is removed, the work was completed in 2007 by Geocimsa. The deepest project using top-down system is Plaza de la Republica parking a three levels basement, built in 2009 also by Geocimsa.

The first micropile construction was performed in 1995 for the underpinning of the neighboring building, Instituto de Recreación de los Trabajadores de Guatemala (IRTRA) building, of Banco Agro Mercantil headquarters. The largest micropiles were performed for the retrofit of Pradera Puerto Barrios shopping center in 2005. The micropiles used were 200 mm diameter steel pipe and are 23.00 m depth. A load test was also performed, being de first load test over micropiles. This project was a finalist in the infrastructure category for the “La Excelencia Awards”, sponsored by Guatemalan Construction Chamber.

The Incienso bridge, Ingeniero “Martín Prado Vélez” bridge, is the second largest bridge in Guatemala and was the first one to use a post-tensioned box girder system, its construction started in 1973 was inaugurated on June 1974. This is also one of the largest bridges in Central America with a total length of 390.00 m, a width of 25.00 m and height from the bottom of the creek to the deck of 135.00 m. It was built by Ingenieros Urruela y Sittendfeld, Cía. Ltda. from Guatemala, and Ingenieros Civiles Asociados, S. A., ICA, from Mexico (Arriola, J., 2007), Figure 2.17 and 2.18 shows different construction stages.

Figure 2.17. Construction of Incienso Bridge deck (Castillo, 2009)

Figure 2.18. Construction of Incienso Bridge piers (Rodas, 2012)

The Rio Dulce bridge is the largest bridge in Guatemala also used a post-tensioned box girder system. Its construction started on January 1977 and concluding in 1980, it has a total length of 900 m. (Matta, 2009). Its foundation consist of driven concrete piles. The bridge suffered damage during 2001 earthquake and it has to been reinforced. Its collapse was avoided due to construction supports that were not removed.

The bridge located in the km 11.5 of CA-9 highway is a milestone for geotechnical engineering it was the first project to use drilled piers and also the first project to use drainage wells to reduce pore pressure in order to stabilize a landslide. The bridge was completed in 1996 by Grupo Fenix and Swissboring Overseas Corp. Ltd.

The bridges over Guacalte and Achiguate Rivers where built in 1999 as the first to use foundation drilled piers built in the alluvial plains. The Reloj de Flores underpass was built in 1999 and includes the construction of the first diaphragm wall in Guatemala, also post-tensioned anchors were used to support the wall. The underpass required a diaphragm wall because the area used to be a lagoon that was filled for the construction of the La Liberación Boulevard, Figure 2.19 shows the excavation stage.

Figure 2.19. Construction of Reloj de Flores Underpass

The clinker silo construction in 2000 of the Cementos Progreso cement plant in Sanarate included the performance of the first loading test. The load test was a full scale test of 1.20 m diameter drilled pier, using posttensioned anchors as reaction, Figure 2.20 shows the test setup. The test, anchors and pile construction were performed by Swissboring Overseas Corp. Ltd with advice of CPK a Salvadorian consultant firm. Other milestone of piling is also provided by Cementos Progreso in its new San Juan cement plant where 7 load tests were performed using Osterberg Load Cells. The test were performed by the American company Load Test meanwhile the piles were performed by Rodio-Swissboring Figure 2.21 shows drilled shafts and load cells installation.

Figure 2.20. Pile load test in Cementos Progreso Cement Plant

The construction of the shotcreted dome of the Magdalena sugar production facility in 2012 by Soiltec was relevant for geotechnical engineering because it was the first time that rammed aggregate piles were used in Guatemala. Figure 2.22 shows the inflatable form for the dome construction and the rammed aggregate pier equipment mounted on a conventional track-mounted excavator.

The wick drains performed in San Rafael mine in 2014 are the first wick drains performed in Guatemala. The drains were built in the tailing dam of the mine in order to accelerate consolidation process, drains are 20.00 m length in a grid of 1.50 by 150 m. The works were performed by Rodio-Swissboring.

Figure 2.21. Drilled shaft load test of San Juan Cement Plant

Figure 2.22. Magdalena Sugar Storage Facility (Soiltec, 2012)

2.2.4. Modern Geotechnical Engineering Practice. The first geophysical investigation, seismic refraction, was completed for Plan de Transporte para la Ciudad de Guatemala sponsored by the Japanesse International Coperation Agency, JICA, in 1996 and performed by Swissboring Overseas Corp. Ltd. The first ground resistivity assessment was required for the consultancy firm Dames & Moore for the site characterization of San Jose Power Plant in 1997 and was performed by Swissboring Overseas Corp. Lda. The first application of Multichannel Analysis of Surface Waves, MASW, was performed by Geo Ciencia Aplicada in 2013 for the site characterization of Veinticuatro building. The first application of Ground Penetrating Radar, GPR, was for the pipe location for the expansion of Pradera Concepcion shopping center in 2014 and was performed by Professor Neil Anderson of Missouri University of Science and Technology.

The first work that used a Cone Penetration Test for geotechnical investigation was Deca II power plant in Puerto Barrios it was performed by the Canadian company

Conetec. The campaign consisted of CPTs up to 40.00 m depth and was supervised by Essen Erdbaulaboratorium a German consulting firm.

2.3. GEOTECHNICAL AND GEOLOGICAL ENGINEERING EDUCATIONAL RESOURCES

The education is one of the main ways to improve the practice. The first step for improvement is to perform a reviews of what are the actual resources and capabilities in the Guatemalan geotechnical education. The state of geotechnical engineering education is available within undergraduate programs of civil engineering and geology and the graduate program of geotechnical engineering.

2.3.1. Civil Engineering Undergaduate Programs. There are several universities in Guatemala, including one of the oldest in the American continent, Universidad de San Carlos de Guatemala, (USAC), established on 1676, that also is the only public university, and is accessible to everybody (annual tuition fee of US$ 20.00), offering several engineering programs including Civil Engineering. Until the late 70’s the USAC used to have the most recognized engineering programs in the country, but during the cruelest part of the civil war in the 1980’s private universities took s stronger role in higher education. A brief history of the engineering programs in Guatemala is presented by Oropin, (2001) in his publication History of the Engineering in Guatemala presenting the most important events of the engineering education progress. A summary of the current universities that offer civil engineering programs in Guatemala is presented below based in the Guillen, (2015) publication about educational offering the construction field.

Universidad de San Carlos de Guatemala, USAC (Established in 1676)

Universidad del Valle de Guatemala, UVG (Established in 1966)

Universidad Mariano Galvez, UMG (Established in 1966)

Universidad Rafael Landivar, URL (Established in 1961)

The civil engineering programs typically consist of about 55 courses equivalent to a very spread number of credits, Table 2.4 shows the number of credits require to obtain the degree. The degree is called “Licenciatura” because when it is bestowed give the license to practice. The only requirement for professional practice in Guatemala is register the title in the Guatemala Engineers Society, no additional exams or qualifications are required.

Table 2.4. Number of credits required to complete civil engineering program

University	Number of credits required to complete a Civil Engineering Degree
Universidad de San Carlos de Guatemala	250
Universidad del Valle de Guatemala	221
Universidad Mariano Galvez	281
Universidad Rafael Landivar	513*

*URL credits could divided by two in order to normalize its value.

The approximate duration of a civil engineering program to award a degree is about five years. A bachelor in sciences, (B.S.), degree could be also completed with 44 courses, approximately four years duration, this program is only offered by the UVG. Other universities as Universidad Galileo offers a construction engineering programs, that is similar to a Bachelors degree.

In 2009 USAC received accreditation from Agencia Centroamericana de Acreditación de Programas de Ingeniería y Arquitectura (ACAIA) for their engineering programs and in 2010 UVG got the same accreditation. The four universities included in their programs soil mechanics and foundations courses as required courses for graduation. UVG used to have Soil Mechanics 1 and 2 in its curricula until 2011, but due to program restructuring both classes were merged into one. Until the early 2000s the only two universities with soil mechanics laboratory facilities were USAC and UVG, lending their installations to UMG and URL respectively. Presently, the four universities have soil mechanics laboratory facilities, the largest facility is the USAC Central Laboratory. A description is available for each laboratory is presented in the professional practice section of this work.

Presently the instructors of soil mechanics course are: Andres Fernando Herrera (UVG), John Arthur Sandoval (UMG), Juan Francisco Calderon (URL), and Dagoberto Alfredo Bautista (USAC).

2.3.2. Geology Undergraduate Program. The Centro Universitario del Norte, CUNOR, the USAC University Council approved the creation of the CUNOR based in Cobán, Alta Verapaz, on November 27, 1975, authorizing serve among other courses at the intermediate level in the Technical Analyst of Mineral Resources, Technical

Exploration of Mineral Resources, initiating the activities of the Centre in January 1976. (Cunor, 2016)

On July 1988 a new curriculum with a defined orientation to field geology, suggesting the name change for the Career Exploration Technician by Geology was approved raising the degree to a Bachelor level. So in 1988 opens the first year of the Bachelor of Geology (5 years) career, always keeping the Geology Technician (3 years).

This was a big step for geo-science related professional practice, bringing the natural complement for geotechnical engineering and not depending of foreigner geologist.

2.3.3. Graduate Program in Geotechnical Engineering. Since geotechnical engineering is such a new area of professional development the demand of training aroused the interest of different institutions. In 2009 USAC started to offer a Master in Science in Geotechnical Engineering. The program is sponsored by the Graduate Studies School part of the USAC Engineering School. The only formal requirement for admission is a title on Civil or Geological engineering or at least a certification of full courses completion with a maximum of one year to present the title. The enrollment process requires fill the admission form, 2 personal photographs, curriculum vitae, legal copy of the title, copy of an identification document, and payment receipt. The courses are tough once a week on Saturday. The program have a fixed mandatory curricula, by means all the course have to be approved in order to opt for the degree. The program have the following courses:

First Quarter

Seminar 1: Investigation Methodology

Geophysics and Geomorphology

Soil Mechanics 1

Second Quarter

Structural Geology and Geotectonics

Hydrology and Hydrogeology

Soil Mechanics II

Third Quarter

Seminar 2: Protocol

Rock Mechanics and Rock Mass Characterization

Fourth Quarter

Special Foundations

Geologic Risks Assessment and Environmental Impact

Fifth Quarter

Applied Geophysics

Geographic Information Systems

Slope Stability and Design

Sixth Quarter

Seminar 3: Final Report

Well Drilling

Underground Design and Excavation

Seventh Quarter

Geosynthetics Engineering

Hydraulic Projects Design and Construction

Master in Sciences Thesis

This program was a great advance to the field of geotechnical engineering and very well structured. One clean improvement could be to add a course in geotechnical earthquake engineering or soil dynamics. According with the secretary of Graduate Studies of USAC 10 students have earned their degree. In 2012 thirty two students get enrolled in the program by 2016 none enrollment was registered until today.

2.4. GEOTECHNICAL AND GEOLOGICAL ENGINEERING PROFESSIONAL PRACTICE

2.4.1. Professional Associations. Geotechnical engineering practice relates to three professional associations:

1. Guatemala Engineers Society (Colegio de Ingenieros de Guatemala), CIG.
2. Guatemalan Society of Soil Mechanics and Geotechnical Engineering (Asociación Guatemalteca de Mecanica de Suelos e Ingeniería Geotécnica), AMSIG.
3. Guatemalan Geological Society (Sociedad Geologica de Guatemala), SGG.

2.4.1.1. Guatemala Engineers Society (Colegio de Ingenieros de Guatemala), CIG. On September 9, 1930, the President of Guatemala, General Jorge Ubico, approved the statutes of the Association of Engineers of Guatemala, which aims to improve the knowledge gained in the military career; the development of the activities of engineering in all its manifestations. This, taking into account the scientific standards and plans that meet the onward march of nations, cultivation of professional ethics and effective link, as well as the help between partners, having been legally established on 10 May 1931. The founders of this society were the engineers Luis Aguilar Peláez, Luis Leonardo, Jorge Erdmenger, Carlos Benfeldt, Benjamin Solorzano and sixty other engineers. After its founding, the Board of Directors was formed, and for this the General Assembly is constituted with 99 voters resulting in the election of chairman of the Board of Directors to Juan de Dios Aguilar with 81 votes. The president Juan de Dios Aguilar, on behalf of the USAC, takes the oath of office to the members who constitute the first Board of the CIG, and the gentlemen who make up the Honor Board and the representative of the CIG to the University Council.

The Constitution of the Republic of Guatemala enacted by the National Constituent Assembly on May 31, 1985, Section Five. Universities, reads as follows:

Article 82. Autonomy of the University of San Carlos of Guatemala. The University of San Carlos, is an autonomous institution with legal personality. State University in character belongs exclusively to direct, organize and develop state higher education and vocational education state university, and the dissemination of culture in all its manifestations.

Article 83. The Government of the University of San Carlos of Guatemala. The Government of the University of San Carlos corresponds to the University Council, composed of the Rector, who presides; deans of faculties, a representative of the professional graduate school at the University of San Carlos, which corresponds to each faculty a full professor and a student per faculty.

Article 90. The licensing of university graduates is mandatory and shall end the moral, scientific, technical and material of the university professions and control of their exercise.

The Professional Association, as professional associations with legal personality, operating in accordance with the law of mandatory professional association the statutes of each school will be approved regardless of the university graduates who were members.

They will contribute to strengthening the autonomy of the University of San Carlos and the aims and objectives of all universities.

The CIG initially had its offices in Elena Avenue between 14th and 15th street where the other professional societies were located. These were attached to the Architects Society, but when architects already had quite a few members formed his own school. Currently the CIG is located at the 7th Avenue 39-60 zone 8 and has 14,417 members (data at 1st of February 2016) (Morales, F., 2009).

2.4.1.2. Guatemalan Society of Soil Mechanics and Geotechnical Engineering (Asociación Guatemalteca de Mecanica de Suelos e Ingeniería Geotécnica), AMSIG. In the pursuit of the improvement of geotechnical engineering practice in Guatemala and with the aim to gather all geo-professionals, the Guatemalan Society of Soil Mechanics and Geotechnical Engineering (Asociación Guatemalteca de Mecanica de Suelos e Ingeniería Geotécnica), AMSIG, was established on August 10th 2010. Its statutes and bylaws are based on the statutes and bylaws of the International Society for Soil Mechanics and Geotechnical Engineering, ISSMGE, including the 2009 Alexandria meeting amendments. This statutes were provided by Paloma Peers from ISSMGE. The initial board was formed by:

President:	Fernando Rafael Callejas Benítez
Vice-president:	Rodolfo Semrau Lago
Secretary:	Rodolfo Francisco de Guadalupe Alvarado Valverde
Treasurer:	Jose Julio Pantoja Prera
Pro-Secretary:	Hector Arturo Valdez Arandi
Vocal 1:	Wilma Siomara De Leon Marroquin
Vocal 2:	Bidkar Manuel Monterroso Rivas

The next step was to become a member of ISSMGE, for this purpose 31 member were required to meet in 2014 and then AMSIG officially became a member of the ISSMGE. This membership opened a wide range of opportunities for geotechnical engineering practitioners in Guatemala, receiving invitations to different events as well as the opportunity to be part of the technical committees. In July of 2015 conversations started about be the representative arm of the CIG for geotechnical matters, an initial agreement document is being discussed at the present time. Since its beginning AMSIG pursues the improvement of professionalism of its member, is the challenge of the new board presided by Dr. Rodolfo Semrau to bring opportunities of training that also will strengthen the affiliation to the society.

2.4.1.3. Guatemala Geological Society (Sociedad Geologica de Guatemala), SGG. The geology profession is one of the oldest in Guatemala. On November of 1974 the geologist headed by Dr. Gabriel Dengo, Samuel Bonis and Otto Bonenberger formed the Guatemala Geological Society (Sociedad Geologica de Guatemala), SGG. The society has a total of 60 members been very active offering different professional development courses and conferences.

2.4.2. In-Situ Testing, Field Investigations, and Geotechnical Construction Capabilities. A very important part for the practice are the contractors, particularly the geotechnical site investigation contractors. A limited number of geotechnical contractors are available in Guatemala particularly for site investigation, based on this a list of the contractors and their capabilities are presented in the Tables 2.5, 2.6 and 2.7. A directory with the information of the different geotechnical contractors is included in the appendix.

Table 2.5. Geotechnical contractors field investigation capabilities

Test and/or Equipment / Contractor	Rodio-Swissboring	Dr. Rodolfo Semrau	Servicios Unificados de Ingeniería	Ingeotecnia	Pala	Suelos y	Geotecnica	Grupo Phi	Geocon
Standard Penetration Test (SPT)	X		X	X	X	X	X	X	X
Rock Core Drilling	X						X	X	X
Cone Penetration Test (CPT)	X								
Vane Shear Test	X	X							
Pressuremeter	X								
Point Load Tes (PLT)	X			X					
Plate Load Test			X	X					

Table 2.6. Geotechnical contractors geophysical field investigation capabilities

Test and/or Equipment / Contractor	Geo-ciencia Aplicada	Ingeo-tecnia	Geopetrol	Geocon
Seismic Refraction	X		X	X
Multichannel Analysis Surface Wave (MASW)	X			
Electrical Resistivity	X	X	X	
Ground Penetrating Radar (GPR)	X			

Table 2.7. Geotechnical contractors field compaction supervision capabilities

Test and/or Equipment / Contractor	Dr. Rodolfo Semrau	Oficina de Ingeniería y Geotecnia, OIG	Mecánica de Suelos y Pavimentos, MECSYPASA	Servicios Unificados de Ingeniería	Ingeotecnia	Pala	Geo Estudios	Suelos y Cimentaciones	Laboratorio de Materiales	Servicios de Ingeniería El Pilar	Geotecnica	Grupo Phi	Soiltest	Geocon
Density Determination Using Sand Cone Equivalent	X	X	X	X	X	X	X	X	X	X	X	X	X	X
Density Determination Using Nuclear Densitometer		X												

Also the capabilities of the contractors to execute geotechnical solutions are critical for the practice, Table 2.8 presents a list of the different contractors and their Capabilities.

Table 2.8. Geotechnical contractor construction capabilities

Technique / Contractor	Rodio-Swissboring	Terrasol (STI)	Pilotecmar	Prodecsa	Soiltec	Geocimsa	Geocon
*Drilled Shafts up to 600 mm diameter	X	X	X	X	X	X	
*Drilled Shafts up to 1800 mm diameter	X	X	X				
*Drilled Shafts larger than 1800 mm diameter	X		X				
Driven Piles	X		X		X		X
Sheet Piles	X		X		X		
Soil Nailing	X	X	X	X	X	X	X
Post tensioned Anchors	X	X	X	X	X	X	
Micropiles	X	X	X	X	X	X	
Diaphragm Walls	X						
Vibro compaction	X						
Vibro replacement	X						
Rammed Aggregate Piers					X		
Grouting	X	X	X	X	X	X	X
Dynamic Compaction	X		X				
Jet Grouting	X						

*Note: the capacity refers to piers performed using a drill rig.

2.4.3. Laboratory Testing Capabilities. The other important part of the practice for geotechnical engineering are laboratory testing Capabilities. Laboratory testing Capabilities are divide in the private soil testing laboratories and universities' laboratories, Tables 2.9 and 2.10 describes de Capabilities of both.

Table 2.9. Soil mechanics laboratory capabilities

Laboratory Name /Resource	Dr. Rodolfo Semrau	Oficina de Ingeniería y Geotecnia, OIG	Mecánica de Suelos y Pavimentos, MECSYPASA	Servicios Unificados de Ingeniería	Ingeotecnia	Pala	Geo Estudios	Suelos y Cimentaciones	Mecanica de Suelos	Servicios de Ingeniería El Pilar (Quetzaltenango)	Geotecnica	Grupo Phi	Geocon
Grain Size Distribution	X	X	X	X	X	X	X	X	X	X	X	X	X
Hydrometer		X											
Liquid / Plastic Limit	X	X	X	X	X	X	X	X	X	X	X	X	X
Direct Shear	X	X		X			X	X	X		X	X	X
Triaxial Cell	X	X							X	X	X		
Oedemeter / Consolidation	X	X									X		
Unconfined Compression	X	X	X			X	X	X	X	X	X	X	X
California Bearing Ratio, CBR, Test	X	X	X	X		X	X	X	X	X	X		X
Compaction Test, Proctor	X	X	X	X		X	X	X	X	X	X		X

Table 2.10. University laboratories resources

Laboratory Name /Resource	Universidad de San Carlos De Guatemala	Universidad del Valle de Guatemala	Universidad Mariano Galvez	Universidad Rafael Landivar
Grain Size Distribution	X	X	X	X
Liquid / Plastic Limit	X	X	X	X
Direct Shear	X		X	
Triaxial Cell	X	X	X	
Oedemeter / Consolidation	X	X		
Unconfined Compression	X	X	X	X
California Bearing Ratio, CBR, Test	X	X		
Compaction Test, Proctor	X	X	X	X

3. LITERATURE REVIEW

3.1. SOIL NAILING

First applications of soil nailing are dated in early 60's at that time the technique was known as the New Austrian Tunneling Method, since was used as support system for tunnel excavations. One of the first applications of soil nailing was in 1972 for a railroad widening project near Versailles, France, where an 18.00m (59-ft) high cut-slope in sand was stabilized using soil nail walls (Rabejac and Toudic, 1974). Applications on United States are register about late 1970's in different projects mainly as temporary excavation support system. In 1984, a prototype soil nail wall 12.00m (40-ft) high was built near Cumberland Gap, Kentucky, as part of a demonstration project funded by the U.S. Department of Transportation Federal Highway Administration (FHWA) (Nicholson, 1986).

Basically soil nailing technique consists to perform a drill in the soil or rock mass, introduce a reinforcement, typically steel rebar, grouting the drill and finally facing construction. The relative narrow spacing of the inclusion reinforces the soil transforming it in a mass working together, similar to a gravity wall.

Soil nailing or soil nail walls are usually apply for slope and excavation stabilization but also can be applied in variety of situations as structure repair, soil and rock mass reinforcement, tunnel stability, factor of safety improvement. Generally soil nailing is applied in soil or weathered rock but also can be used as rock bolts for rock mass stabilization. Is very important to determine application duration, temporary applications are defined as a service life not large than 18 months, any duration further this limit is considered as permanent.

3.1.1. Basic Elements of a Soil Nail Walls. Soil nailing walls have five basics elements as described herein.

3.1.1.1. Reinforcement or inclusion. Reinforcement, steel or fiberglass, is installed within the drill, usually takes tensile stress gradually during construction. Rebar is normally treaded in the exterior tip. Different kind of corrosion protection could be applied to the reinforcement, depending of the soil aggressivity.

3.1.1.2. Grout. Grout is placed in the drill usually thru an injection hose or sacrificial pipe from the bottom of the drill. The grout main function is transfer stresses from the soil to the reinforcement, also provides corrosion protection.

3.1.1.3. Bearing plate, hex nut and washer. These elements transmit stresses form the reinforcement to the facing. Normally is a square steel plate about 13 to 20 mm thick and 200x200 mm or 250x250 m side.

3.1.1.4. Facing. Facing could be temporary or permanent, depending of the final support of the wall. Facing distributes stress from the reinforcement conceding one of most important characteristics of the soil nailing walls, structural redundancy.

3.1.1.5. Drainage. In order to avoid pore water pressure increase drainage shall be provided. Drainage could be from geocomposite drainage strips or drilled sub horizontal drains with a PVC pipe covered with geotextile in order to avoid fine migration.

3.1.2. Applications of Soil Nail Walls. As mentioned before soil nail walls most common applications are:

a. Slope stabilization
b. Excavation stabilization
c. Structure repair (e.g. retaining walls)
d. Structure reinforcement (e.g. bridge abutments)
e. Tunnel stabilization
f. Factor of safety improvement

The Figure 3.1 shows a vertical soil nailing wall for a basement construction.

3.1.3. Feasibility Evaluations of Soil Nail Walls. As any engineering, geotechnical-structural, solution feasibility depends on two factor technical suitability and economical accordance. For technical suitability is defined by two factor, ground conditions and ground water level location, there also other factor like adjacent constructions and loads that could affect. Economical suitability should be determined doing a comparison with other alternatives and a specific costs evaluation.

Figure 3.1. Vertical soil nailing wall for excavation stabilization

3.1.3.1. Ground conditions. The most important ground condition for soil nail walls performance is the temporary stability during each excavations stage, normally 24 to 72 hour. Is recommended do each stage with some alternates berms as shown in the Figure 3.2.

The following soils types are more favorable to perform soil nail walls:

a. Sandy soil in a dense to very dense conditions with some apparent or temporary cohesion.
b. Stiff to hard fine grained soil. Creep risk should be evaluated in these kinds of soils.
c. Rock or weathered rock. Joints, joints fill, and joint inclinations should be assess.
d. Glacial soil.

The following soil types are not recommended for soil nail walls performance:

a. Dry, poorly graded, cohesionless soils.

b. Soft to very soft fine grained soil.

c. Organic soils.

d. Loess.

e. Concrete aggressive soils.

f. Soil with high water content.

Figure 3.2. Berm utilization during construction

3.1.3.2. Ground water level. Sites with a superficial o high ground water level, within excavation depth, should not be suitable for soil nailing performance. Also saturated soils are not recommended for soil nail walls performance.

Ground water level should be at least 1.00 m below excavation bottom level (including capillarity zone).

An extensive site investigation is recommended for the correct ground conditions assessment.

3.1.3.3. Site investigation, laboratory testing, and recommendations. As any construction project an extensive site investigation should be carry out for soil nail walls design. Geotechnical Engineering Circular No. 7 (Lazarte et al, 2003), Soil Nailing Walls, published by the Federal Highway Administration, FHWA in March 2003 presents a detailed minimum investigation for this kind of retaining structures. Site´s geotechnical investigation must include a review of existing geotechnical information, site reconnaissance, subsurface investigation and laboratory testing program.

3.1.3.4. Site reconnaissance. Field visit should be mandatory when a soil nail walls will be designed. The following aspect should be verified during site reconnaissance:

a. Site accessibility;
b. Traffic conditions and control during investigation and construction;
c. Overhead space limitations;
d. Drainage and erosion patterns;
e. Nature and condition of above-ground structures;
f. Identification of underground utilities;

Behavior of similar works performed in the area.

g. Response of nearby cuts, slopes, and excavations; and
h. Evidence of surface settlement.

3.1.3.5. Field testing and sampling. Borings shall be performed and N of Standard Penetration Test (SPT) or qt of Cone Penetration Test (CPT) values have to be provided. Also stratigraphic profile and soil classification should be provided. There are many values that can be correlated to SPT and CPT results, this why are the preferred testing methods. Ground water level has to be located. Depending of ground conditions intact and disturbed samples shall be taken. Test pits are a good investigation option but are limited in depth. Depth and boring location are recommend by Cheney 1988 and Sabatini et al 1999 and are shown in the Figure 3.3.

Many alternative explorations methods are available, field testing and sampling intend to assess ground conditions, so the above stated investigation should be the minimum. The Table 3.1 presents a reference of common geotechnical field procedures and tests recommend by Lazarte et al 2003.

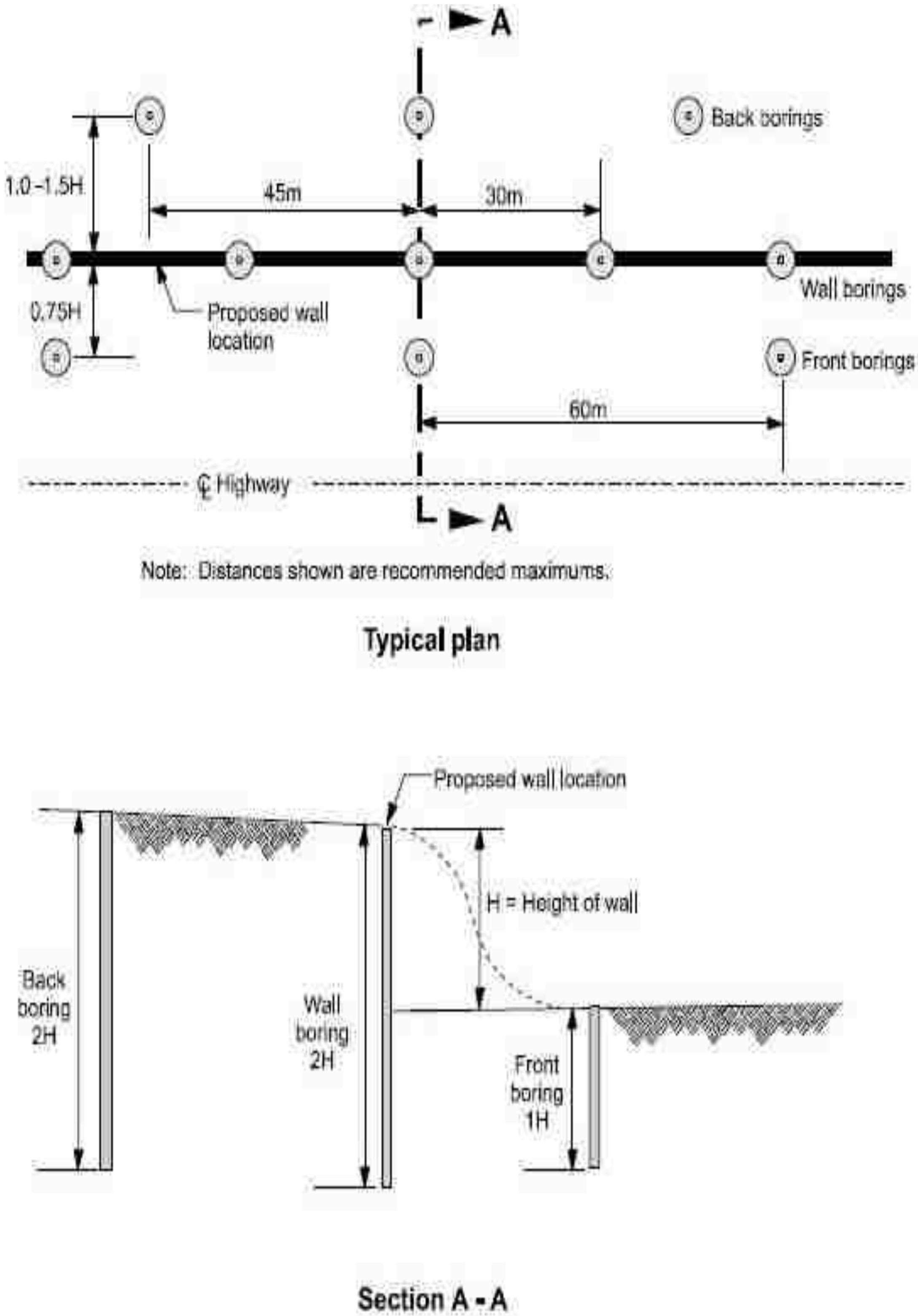


Figure 3.3. Preliminary geotechnical boring layout for soil nailing walls

Table 3.1. Common geotechnical field procedures and test

	Activity	Standard(1) /FHWA Reference	Most suitable for	Not suitable for	Obtained from field activity
Field Proce-dure	Preservation and Transportation of soil Samples	ASTM D4220-95	ALL	NA	Representative Samples
	Thin-Walled Tube Sampling	ASTM D1587-00	Clays, Silts	Sands, Gravel	Undisturbed Samples
	Subsurface Explorations (Soil and Rock)	ASTM D5434-97	ALL	NA	Various
Field Test	Standard Penetration Test (SPT)	ASTM D1586-99 ASTM(2) D6066-96e1	Sand, Silt	(2)	Stratigraphy, SPT N-values relative density, groundwater, samples
	Cone Penetration Test (CPT)	ASTM D5778-95 Briaud (1992)	Sand, Silt, and Clay	Gravel, bouldery soil	Continuous stratigraphy, soil, type, strength, relative density(3), K_0 , pore, pressures, no sample
	Field vane Shear Test (VST)	ASTM D2573-94	Soft to Medium Clay	Sand and Gravel	Undrained shear strength
	Pressuremeter Test (PMT)	ASTM D4719-00 Briaud (1989)	Soft Rock, Dense Sand, Non-Sensitive Clay, Gravel, Till	Soft Clays, Loose Silts and Sands	Soil type, strength, K_0(3) , OCR(4), compressibility, soil modulus, no sample
	Flat Plate Dilatometer Test (DMT)	ASTM D6635-01, Briaud and Miran (1992)	Sand and Clay	Gravel	Soil type, K_0 ,OCR, undrained shear strength, soil modulus, no sample
Notes:	(1) Individual ASTM standards can be found in ASTM (2002). Arman et al. (1997) and Sabatini et al. (2002) present general discussions on these field procedures (2) SPT can be used whit limitations in clays and gravels. (3) K_0 is the at-rest earth lateral pressure coefficient. (4) OCR is the over consolidation ratio. (5) ASTM D6066-9e1 for the use of SPT in liquefaction resistance evaluation				

3.1.3.6. Laboratory testing program. Laboratory testing for soil classification and index properties should be performed, but the main feature to be determined is long and short term strength parameters. For permanent applications triaxial compression tests should be performed, Consolidated Drained (CD) or more often use Consolidated Unconsolidated Undrained (CU) with pore water pressure measurements. Strength short term parameters will rule construction and long term parameter will be essential for a permanent soil nail wall design.

3.1.3.7. Soil creep potential. Creep in the interface between nail and soil should be evaluated in any permanent application in fine grained soils. The following parameters could be indicative of creep potential.

a. Fine-grained soils with a liquid limit (LL) $\geq$ 50;

b. Fine-grained soils with plasticity index (PI) $\geq$ 20;

c. Fine-grained soils with undrained shear strengths $\leq$ 50 kPa (1,000 psf);

d. A liquidity index (LI) $\geq$ 0.2; and

e. Organic soils. (Lazarte et al, 2003)

Creep potential could be directly determine performing a creep test during a nail load test.

3.1.3.8. Bond strength. Bond strength, pull out resistance, is one of the most important parameters in nail internal design. It, conditions drill diameter and/or nail length. In order to verify bond strength pull out tests have to be carry out before or during soil nail wall performance. Detailed procedure for this test is presented by Lazarte et al (2003). Many studies have been performed to determine this value. In Guatemala there is only one reference in this matter "Bond strength determination between nails and four different kinds of soil in Guatemala" prepared by Callejas (2001).

The Table 3.2 presents reference values presented by Elias and Juran (1991) for bond strength of soil nails in different types of soil and rock.

Table 3.2. Common procedures and laboratory test for soils

Material	Construction Method	Soil Rock Type	Ultimate Bond Strength, q_u (kPa)
Field Procedure	Rotary Drilled	Marl/limestone	300 - 400
		Phillite	100 - 300
		Chalk	500 - 600
		Soft dolomite	400 - 600
		Fissured dolomite	600 - 1000
		Weathered sandstone	200 - 300
		Weathered shale	100 - 150
		Weathered Schist	100 - 175
		Basalt	500 - 600
		Slate-Hard shale	300 - 400
Cohesionless Soils	Rotary Drilled	Sand/gravel	100 - 180
		Silty sand	100 - 150
		Silt	60 - 75
		Piedmont residual	40 - 120
		Fine Colluvium	75 - 150
	Driven Casing	Sand/gravel	
		low overburden	190 - 240
		High overburden	280 - 430
		Dense Moraine	380 - 480
		Colluvium	100 - 180
	Augered	Silty sand fill	20 - 40
		Silty fine sand	55 - 90
		Silty clayey sand	60 - 140
	Jet Grouted	Sand	380
		Sand/gravel	700
Fine-Grained Soils	Rotary Drilled	Silty clay	35 - 50
	Driven Casing	Clayey silt	90 - 140
	Augered	Loess	25 - 75
		Soft clay	20 - 30
		Stiff clay	40 - 60
		Stiff clayey silt	40 - 100
		Calcareous sandy clay	90 - 140
Notes:	Convert values in kPa to psf by multiplying by 20.9 Convert values in kPa to psi by multiplying by 0.145		

3.1.4. Construction Materials and Methods. Different construction materials and methods can be used depending mostly of local equipment and materials availability. Construction method and materials are also directly related to ground conditions like, borehole stability, soil aggressivity, and erosinability.

3.1.4.1. Construction methods. There is a general procedure for soil nail walls construction whether or not the soil is already removed. In other words is indifferent if is an excavation or a slope.

a. Initial cut, typically with a height between 1.00 to 2.00 m.
b. Drilling, drill typically has an inclination, between 10 and 20°, in order to reach the most critical failure surface but also for construction in order to fill the drill easier.
c. Reinforcement installation inside the drill.
d. Grouting, normally grouting is performed throughout a disposable hose attached to the rebar. Grouting should be performed from the bottom of the drill in order to ensure full fill of the drill.
e. Facing, usually a shotcrete layer is applied over ground surface. Then bearing plate and hex nut are installed. If drainage strips are provided, should be placed before facing application. Also sub horizontal drains should be drilled before facing installation.
f. Construction of a final, permanent facing (if required).

These stages are repeated until final depth is reached.

In order to give support to wall dead weight transmitting to the ground strip footing construction is recommend.

3.1.4.2. Drilling methods. The different drilling methods are presented herein.

3.1.4.2.1. <u>Drilled and grouted soil nails.</u> It is the most commonly used method, consist to drill a borehole with auger, "fish tail", drilling hammer or driven casing, and then install the reinforcement and grout the nail. This method usually requires borehole temporary stability.

3.1.4.2.2. <u>Driven soil nails.</u> This installation technique is used when a fast application is need and is suitable for soft soils. An extra thickness has to be provided due to corrosion.

3.1.4.2.3. <u>Self-drilling soil nails.</u> This is a very fast installation method used very often in tunnel and mining applications. In this case nails works as drilling tool and final reinforcement at the same time. Bar has an inner central hole that allow drilling fluid circulation in many cases grout is used as drilling fluid also is equipped drilling toe. Normally installation is performed using rotation and percussion.

3.1.4.2.4. <u>Jet-grouted soil nails.</u> Drilling is performed using jet grouting technique then reinforcement is introduced inside drill hole (jet grouting column).

3.1.4.2.5. <u>Launched soil nails.</u> In this method, bare bars are "launched" into the soil at very high speeds using a firing mechanism involving compressed air. (Lazarte et al, 2003)

3.1.4.3. Materials of components of a soil nailing walls. As mentioned above soil nail walls have three main components, nail, grout and facing with a nail head as connection element. Materials for each component may vary upon specific project requirements. The entire system has to be congruent, all components have to fit together and look for the similar fail mechanism.

3.1.4.3.1. <u>Nails.</u> Normally nails are just as simple steel rebar with a tread in the tip, pre tread rebar is also commonly used. Fiberglass nails, tendons or bars, are used for corrosive environments. Different yield stresses are available, Grade 420 MPa (G60) or 560 MPa (G80). Centralizer shall be provided in order to protect rebar from corrosion averting nail contact with soil.

3.1.4.3.2. <u>Nail head.</u> Nail head is composed of steel bearing plate, hex nut and washer. Usually bearing plate is made of Grade 250 MPa (A36) steel, and hex nut and washer from Grade 420 MPa (G60) or Grade 520 (G75) steel. For temporary application minimum bearing plate recommend thickness is 13 mm, and for permanent applications 19 mm.

3.1.4.3.3. <u>Grout.</u> Grout for soil nails is commonly a neat cement grout, which fills the annular space between the nail bar and the surrounding ground (Lazarte et al, 2003). Other grout type as epoxy cement grout or simply epoxy are also used, mostly when a reduced setting time is required. Any Portland cement types I, II, III and V can be used. Water / Cement ratio is usually in the range of 0.4 to 0.5, in order to pump a grout with these w/c ratios additives are usually added. Grout common minimum 28-day unconfined compressive strength is 21 MPa (3,000 psi).

3.1.4.3.4. Wall facing. Facing connect all inclusions, making them work together. Normally facing is constructed of reinforced concrete, sprayed concrete or shotcrete. Weld wire mesh, WWM, is used as reinforcement in most of the cases due to ease of construction, also and additional shorter reinforcement bars (referred to as waler bars) around the nail heads. Other materials as double torsion wire mesh, cable net, or precast concrete or wood panels can be used.

3.1.4.3.5. Drainage systems. Drainage systems have to be provided; one or several of the following options can be used:

a. Sub horizontal drains, drilled or driven into the soil, usually cover with geotextile.
b. Drainage geocomposite strip.
c. Weep holes.

The drainage system also includes a footing drain to convey collected drainage water away from the wall face.

3.1.5. Analysis of Soil Nailing Walls. The analysis of soil nailing walls is based in load transfer between the different elements. Load transfer will occur gradually with the facing installation; also load transfer is directly related with deformation.

Tension forces are developed in each stage, cut phase, these forces are transmitted to the facing as shear in the support, nail, and flexion between supports. Nails also increase shear soil strength. During each excavation stage, each row nails will getting loaded incrementally as more mass is retained. So in many cases the last stage, cut, is the critical situation. Therefore, soil nail walls analysis and design have to consider final condition and constructions stages as well. The Figure 3.4 presented by Lazarte et al (2003) shows the potential failure surfaces and soil nail tensile forces.

3.1.5.1. Limit states. The analysis and design of soil nail walls must consider two distinct limiting conditions, strength limit states and the service limit states.

3.1.5.2. Strength limit states and the service limit states. The Strength limit states refer to failure or collapse modes in which the applied loads induce stresses that are greater than the strength of the whole system or individual components, and the structure becomes unstable. Strength limit states arise when one or more potential failure modes

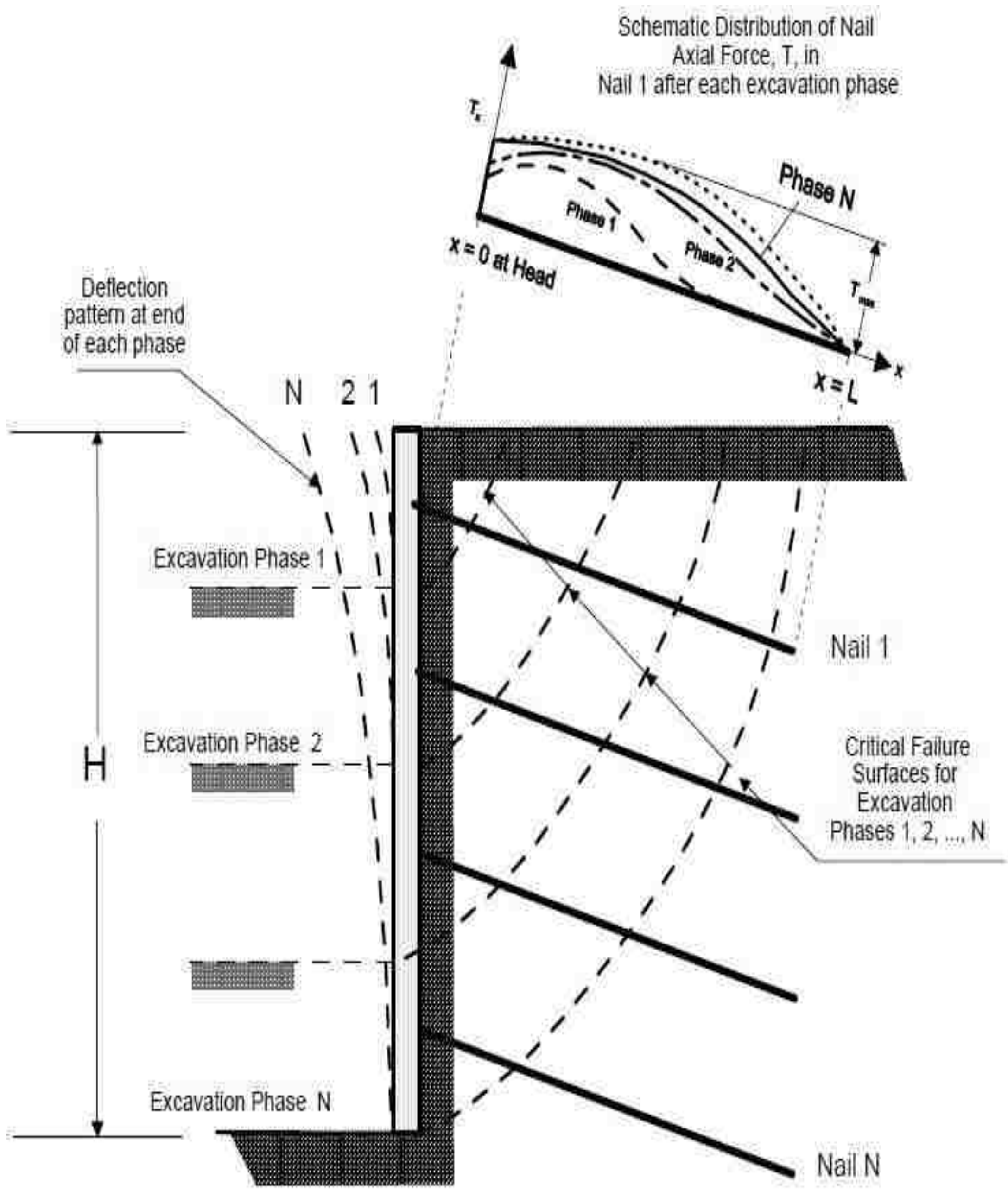


Figure 3.4. Potential failure surfaces and soil nail tensile forces

are realized. The design of a soil nail wall should ensure that the system is safe against all of the potential failure conditions. Lazarte et al (2003) prepared the Figure 3.5 where the failure modes are classified as:

1. External failure mode;
2. Internal failure mode; and
3. Facing failure mode.

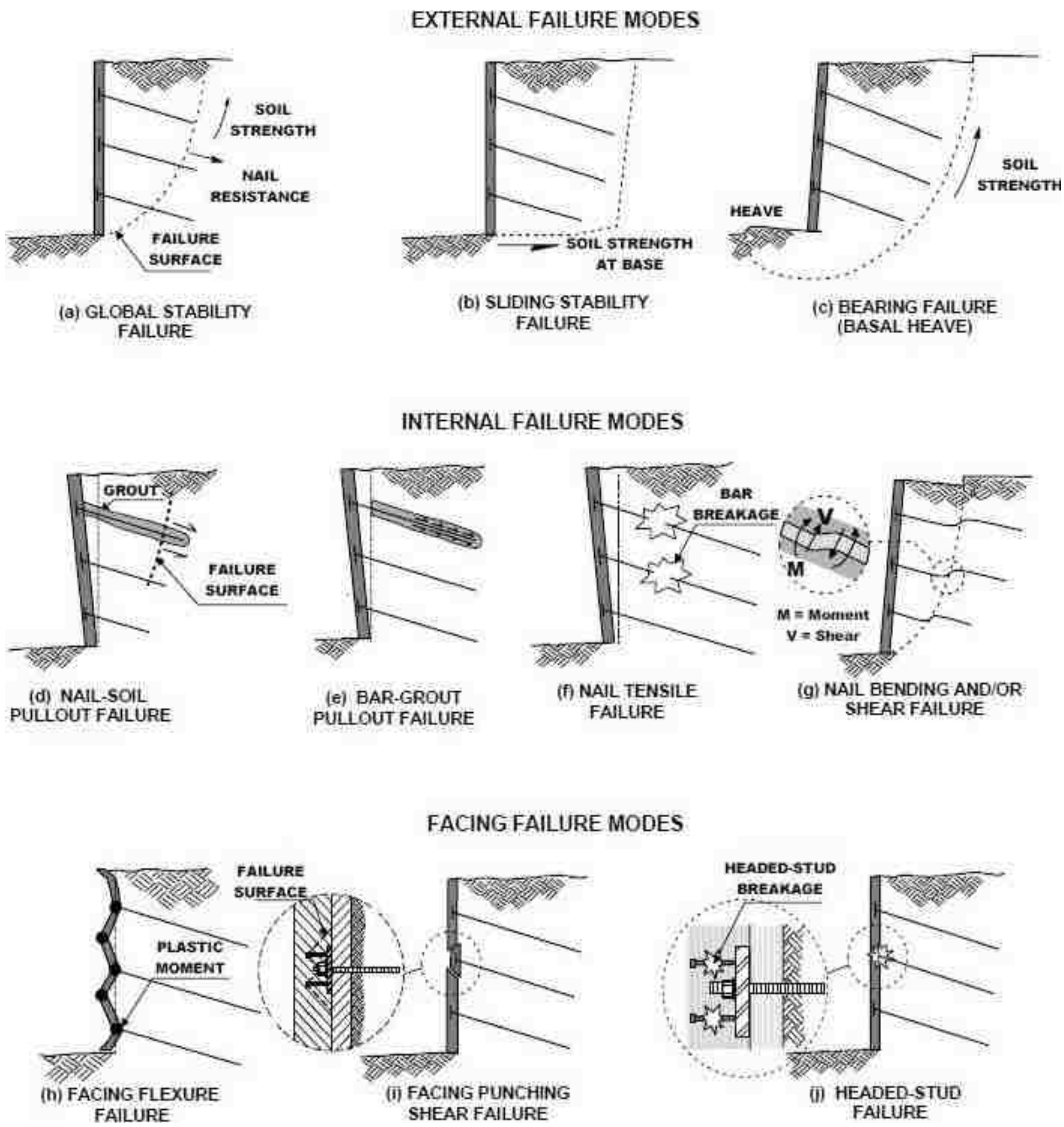


Figure 3.5. Principal modes of failure of soil nail wall systems

The service limit states refers to conditions that do not involve collapse, but rather impair the normal and safe operation of the structure. The major service limit state associated with soil nail walls is excessive wall deformation (Lazarte et al, 2003). Excessive deformation could induce settlements and/or fissures or cracks. In many cases service limitations conditions design, mostly when adjacent or close structure could be in risk of failure or damage.

3.1.5.3. External failure modes. External failure modes refer to the development of potential failure surfaces passing through or behind the soil nails (i.e., failure surfaces that may or may not intersect the nails). For external failure modes, the soil nail wall mass is generally treated as a block. Mainly external failure modes involves global stability of whole soil mass, sliding failure (shear at the base) and bearing capacity failure (basal heave), global stability usually is the critical condition to analyze.

Global stability and sliding failure conditions minimum nail's length. Global stability due to critical slippage surface and sliding due to equivalent "embedment" length to achieve necessary reaction. Final length is defined during internal design.

3.1.5.3.1. Global stability. Slope stability investigates potential failure mechanisms, and can assess slope sensitivity to different triggering mechanisms, designing of optimal slopes with regard to safety, reliability and economics. Slope stability analysis can be performed with simple limit equilibrium analysis and/or for more complex situations numerical modeling (finite element method, FEM). "In addition, the use of the risk assessment concept is increasing today. Risk assessment is concerned with both the consequence of slope failure and the probability of failure (both require an understanding of the failure mechanism)". (Eberhardt, 2003) Limit equilibrium are the most commonly used and simple solution methods. Some of the most limit equilibrium methods are:

a. Bishop
b. Felleneius
c. Jambu
d. Modified Bishop

In limit equilibrium analysis, the potentially sliding mass is modeled as a rigid block, global force and/or moment equilibrium is established, and a stability factor of safety that relates the stabilizing and destabilizing effects is calculated. As with traditional slope stability analyses, various potential failure surfaces are evaluated until the most critical surface (i.e., the one corresponding to the lowest factor of safety) is obtained (Lazarte et al, 2003).

Limit equilibrium analysis is simple a forces sum and provides a factor of safety as final result. The factor of safety is defined as the ratio between the sums of the resisting forces divided by the sum of driving forces.

$$F.S. = \frac{\sum Resisiting\ Forces}{\sum Driving\ Forces} \quad \text{(Eq. 3.1)}$$

Many probable failure surfaces have to be calculated to find the critical one, the one with the lowest factor of safety. Due to complexity of this calculations computer programs are usually used.

3.1.5.3.2. <u>Sliding.</u> Sliding stability analysis considers the ability of the soil nail wall to resist sliding along the base of the retained system in response to lateral earth pressures behind the soil nails. Sliding failure may occur when additional lateral earth pressures, mobilized by the excavation, exceed the sliding resistance along the base.

Concepts similar to those used to assess sliding stability of gravity retaining structures (in which Rankine or Coulomb theories of lateral earth pressures are used) can be applied to assess the sliding stability of a soil nail wall system, in the Figure 3.6 Lazarte et al (2003) presents an acting forces diagram for a typical soil nailing wall.

The terms in Figure 3.6 are identified as: H = wall height; H = slope rise up to bench (if present); β = backslope angle; βeq = equivalent backslope angle [for broken slopes β_{eq} = tan-1(ΔH/H), for infinite slopes $\beta_{eq} = \beta$]; α= face batter angle;θ = inclination of wall face from horizontal (i.e., $\theta = \alpha + 90°$); c_b = soil cohesion strength along the base; B_L = length of the horizontal failure surface where c_b is effectively acting; W = weight of soil nail block; Q_T = total surcharge load; φ'_b = effective angle of internal friction of the base (remolded or residual values may be needed if significant movement takes place); φ'= effective friction angle of soil behind soil nail block; δ = wall-soil interface friction angle [for a broken slope, $\delta = \beta_{eq}$, for infinite slope, $\delta = \beta$]; γ = total unit weight of soil mass; and H_1 = effective height over which the earth pressure acts [$H_1 = H + (B + \tan \alpha) \tan \beta_{eq}$].

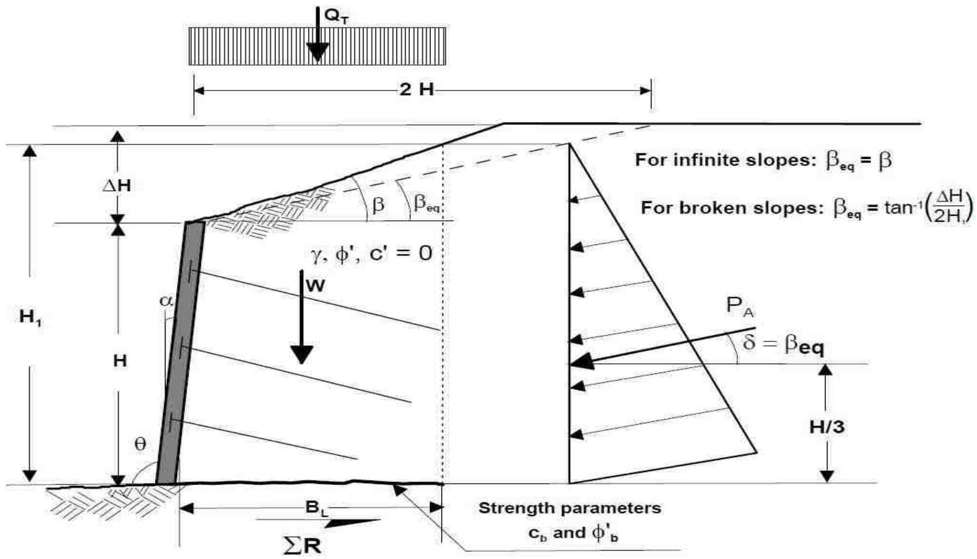


Figure 3.6. Sliding stability and acting forces in a soil nail

3.1.5.3.3. <u>Bearing capacity.</u> Bearing capacity failure is not common, soft layer underneath wall toe and at the same time a stiffer layer underneath this soft layer is the typical conditions that trigger this failure mechanism. Water flows, can also contribute to this kind of failure, arrows in the Figure 3.7 show slope toe heave.

Figure 3.7. Ground heave due to active landslide

3.1.5.4. Seismic considerations in soil nail wall stability. Seismic effect assessment is very important in seismic regions as Central America particularly Guatemala. Soil nail walls have performed remarkably well during strong ground motions, in contrast to the generally poor performance of gravity retaining structures. After the 1989 Loma Prieta, California; 1995 Kobe, Japan; and 2001 Nisqually, Washington earthquakes, it was reported that soil nail walls showed no sign of distress or significant permanent deflection, despite having experienced, in some cases, ground accelerations as high as 0.7g (Felio et al., 1990; Tatsuoka et al., 1997; and Tufenkjian, 2002).

Pseudo static methods as Monobe Okabe or Seed and Whitman modification can be used to include seismic effect in slope stability.

In general, it is acceptable to select a seismic coefficient for soil nail walls between:

$$k_h = 0.5\ A_m \text{ to } 0.67\ A_m \qquad \text{(Eq. 3.2)}$$

The coefficient k_h is a fraction of the normalized horizontal acceleration (A_m), which acts at the centroid of the wall-soil mass (AASHTO, 1996). A_m is a function of the normalized peak ground acceleration coefficient (A), which is the actual peak ground acceleration normalized by the acceleration of gravity (g), and is defined as:

$$A_m = (1.45 - A)\ A \qquad \text{(Eq. 3.3)}$$

This range has provided wall designs that yield tolerable deformations in highway facilities (Kavazanjian et al., 1997).

3.1.5.5. Internal failure modes. Internal failure modes refer to failure in the load transfer mechanisms between the soil, the nail, and the grout. Soil nails mobilize bond strength between the grout and the surrounding soil as the soil nail wall system deforms during excavation. The bond strength is mobilized progressively along the entire soil nail with a certain distribution that is affected by numerous factors. As the bond strength is mobilized, tensile forces in the nail are developed.

Depending on the soil nail tensile strength and length, and the bond strength, bond stress distributions vary and different internal failure modes can be realized. Typical internal failure modes related to the soil nail are shown in the Figures 3.5d–g.

The nail pullout failure is a failure along the soil-grout interface due to insufficient intrinsic bond strength and/or insufficient nail length, Figure 3.5d.

The strength against slippage along the grout and steel bar interface (Figure 3.5e) is derived mainly from mechanical interlocking of grout between the protrusions and "valleys" of the nail bar surface. Mechanical interlocking provides significant resistance when threaded bars are used and is negligible in smooth bars. The most common and recommended practice is the use of threaded bars, which reduces the potential for slippage between the nail bar and grout.

The tensile failure of the nail is the fail in tension if there is inadequate tensile strength, Figure 3.5f.

Soil nails work predominantly in tension, but they also mobilize stresses due to shear and bending at the intersection of the slip surface with the soil nail (Schlosser, 1983; Elias and Juran, 1991), Figure 3.5g. The shear and bending resistances of the soil nails are mobilized only after relatively large displacements have taken place along the slip surface. Some researchers have found that shear and bending nail strengths contribute no more than approximately 10 percent of the overall stability of the wall. Due to this relatively modest contribution, the shear and bending strengths of the soil nails are conservatively disregarded in the guidelines contained in this document.

3.1.5.5.1. Nail pullout failure. Pullout failure is the primary internal failure mode in a soil nail wall. This failure mode may occur when the pullout capacity per unit length is inadequate and/or the nail length is insufficient, Figure 3.8 by Lazarte et al (2003) shows single nails stress-transfer mode. In general, the mobilized pullout per unit length, Q, (also called the load transfer rate) can be expressed as:

$$Q = \pi \, q \, DDH \qquad \text{(Eq. 3.4)}$$

Where: q = mobilized shear stress acting around the perimeter of the nail-soil interface; and DDH = average or effective diameter of the drill hole.

The pullout capacity, Rp, is mobilized when the ultimate bond strength is achieved and is expressed as:

Rp = Tmax = Qu Lp (Eq. 3.5)

with:

Qu = π qu DDH (Eq. 3.6)

Where: Qu = pullout capacity per unit length (also referred to as load transfer rate capacity); an qu = ultimate bond strength.

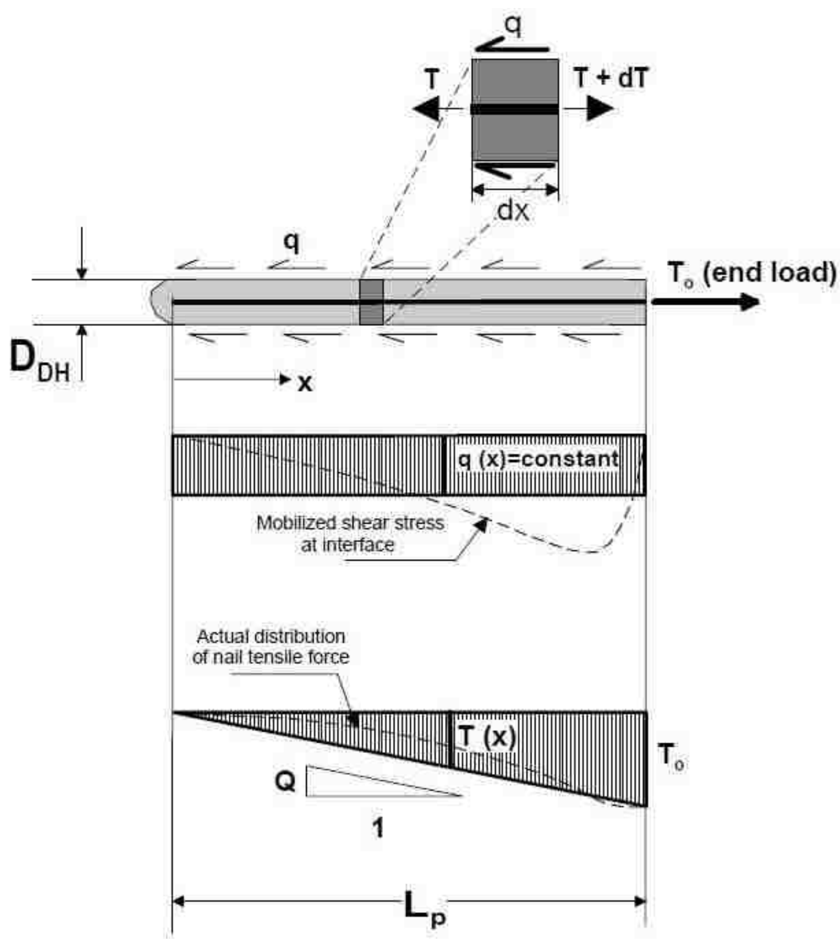


Figure 3.8. Single nails stress-transfer mode

3.1.5.5.2. Facing connection failure modes. The most common potential failure modes at the facing-nail head connection are presented in Figure 3.3 and are shown by Lazarte et al (2003) in detail in Figure 3.9.

The flexure failure is a failure mode due to excessive bending beyond the facing's flexural capacity. This failure mode should be considered separately for both temporary and permanent facings.

The punching shear failure mode occurs in the facing around the nails and should be evaluated for both temporary and permanent facings.

The headed-stud tensile failure is a failure of the headed studs in tension. This failure mode is only a concern for permanent facings.

3.1.5.5.3. Tensile forces at the wall facing. The nail tensile force at the wall face, To, is smaller than or equal to the maximum nail tensile force, as shown in Figure 3.10 (Byrne et al., 1998). The Figure 3.10 presents the in-service normalized values of the nail tensile forces measured at the facing of actual soil nail walls. These values are related to long-term soil nail forces and do not include freezing (or other) forces at the face. The normalized nail forces at the facing, also referred to as the nail head force, are comparable in distribution to the normalized maximum nail tensile forces shown in Figure 3.11. By comparing these two figures, the ratio of normalized nail head force to the maximum nail force varies from 0.6 to 1.0. In the upper half of the wall, the mean, normalized nail head force ranges between 0.4 and 0.5; in the lower half, the normalized forces decrease gradually and tends to zero at the bottom. Considering the normalization and influence area described above, this trend shows that that head nail tensile force typically varies from To = 0.60 KA γ H to 0.70 KA γ H.

These observations are consistent with those made on experimental walls in Germany and in France. In Germany, actual earth pressure measurements, recorded via total stress cells located at the shotcrete-soil interface, indicate that the equivalent earth pressure on the facing between 60 to 70 percent of the Coulomb active earth pressure for most conditions (Gässler and Gudehus, 1981).

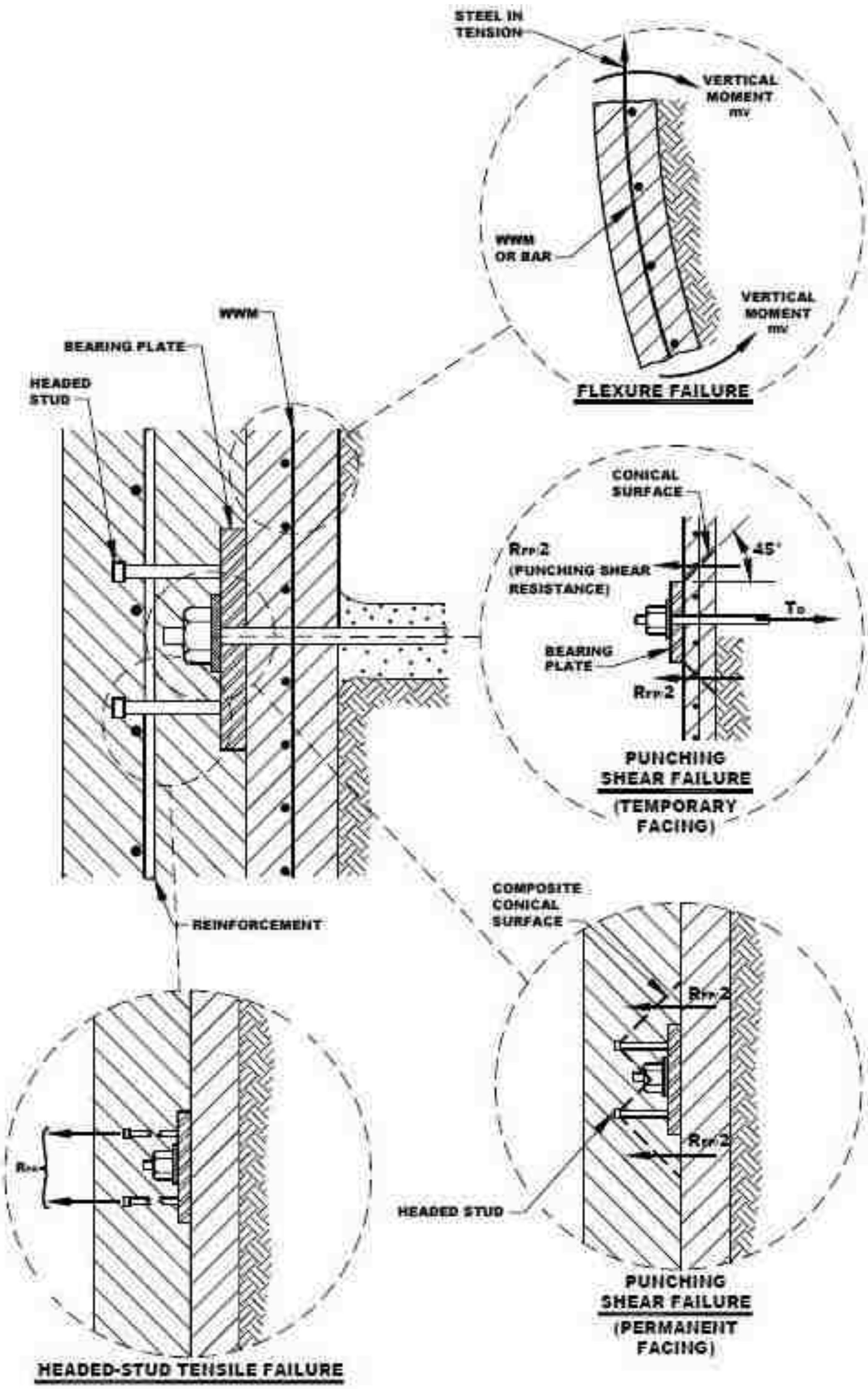


Figure 3.9. Facing connection failure modes

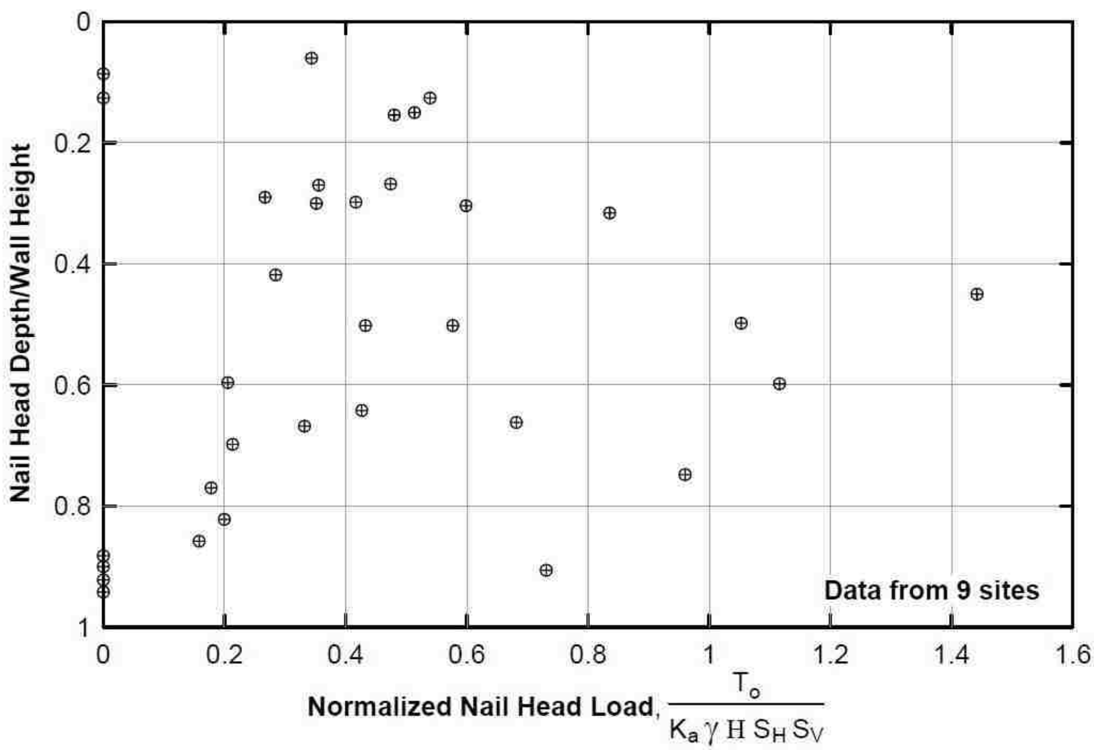


Figure 3.10. Summary of facing tensile forces measured in walls

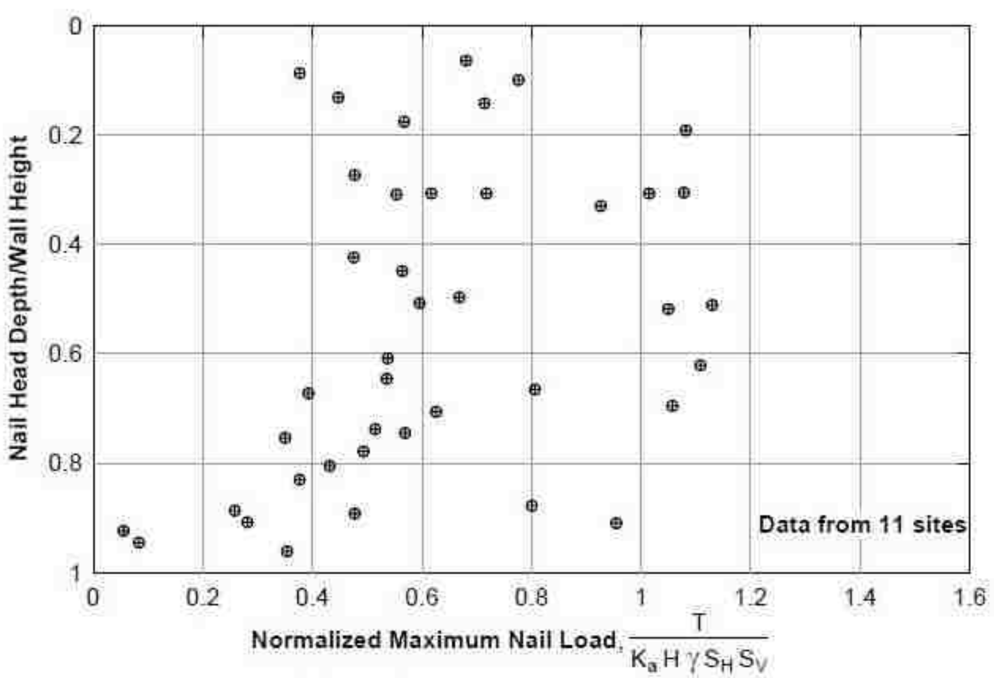


Figure 3.11. Summary of maximum nail tensile forces measured in walls

In the French tests, the ratio of the nail head force to the maximum nail force generally varied between 0.4 and 0.5 in the upper portion of the walls (FHWA, 1993). In addition, these test results showed that due to the effect of soil arching between nails, a closer spacing of the nails caused a reduction in the measured forces on the wall facing as compared to what would be expected using simple tributary area contributions.

Based on these results, the Clouterre (1991) design guidelines recommend adopting in-service values of the head nail tensile force as:

60 percent of the maximum nail service load for a nail vertical spacing of 1.00m or less; 100 percent of the maximum nail service load for a nail vertical spacing of 3.00m or more; and a linear interpolation for intermediate nail spacing, this recommendation is formally expressed as follows:

$$T_o = T_{max\text{-}s}[0.6 + 0.2 (S_{max} [m] - 1)] \qquad \text{(Eq. 3.7)}$$

Where: To = Design nail head tensile force; Tmax-s = Maximum design nail tensile force and Smax = Maximum soil nail spacing. Use maximum of SV and SH, the vertical and horizontal nail spacing, respectively, in Equation 3.7. (Lazarte et al, 2003)

3.1.5.5.4. <u>Flexural failure.</u> As with other reinforced concrete/shotcrete structures, flexural failure is achieved progressively. After the first yield of the facing section, Figure 3.12c, progressive cracking takes place on both sides of the facing as the lateral earth pressure increases. As the lateral pressure increases, fractures grow and deflections (δ) and nail tensile forces increase. Individual fractures indicate where the flexural capacity is achieved. Eventually, an ultimate stage of the structure is achieved when all fractures connect, act as hinges, and form a mechanism referred to as the critical yield line pattern. Yield line patterns are dependent on various factors including the soil lateral pressures, horizontal and vertical nail spacing, size of bearing plate, facing thickness, reinforcement layout, and concrete strength (Seible, 1996) and are associated with a maximum soil pressure.

In theory, the soil pressure that causes facing failure (i.e., the critical yield line pattern) can be applied to an influence area around the nail head, and a nail tensile force (“reaction”) is obtained. This force is designated as the facing flexure capacity, RFF, and is related to the flexural capacity per unit length of the facing. The flexural capacity per unit length of the facing is the maximum resisting moment per unit length that can be mobilized in the facing section. Based on yield-line theory concepts, RFF can be estimated as the minimum of:

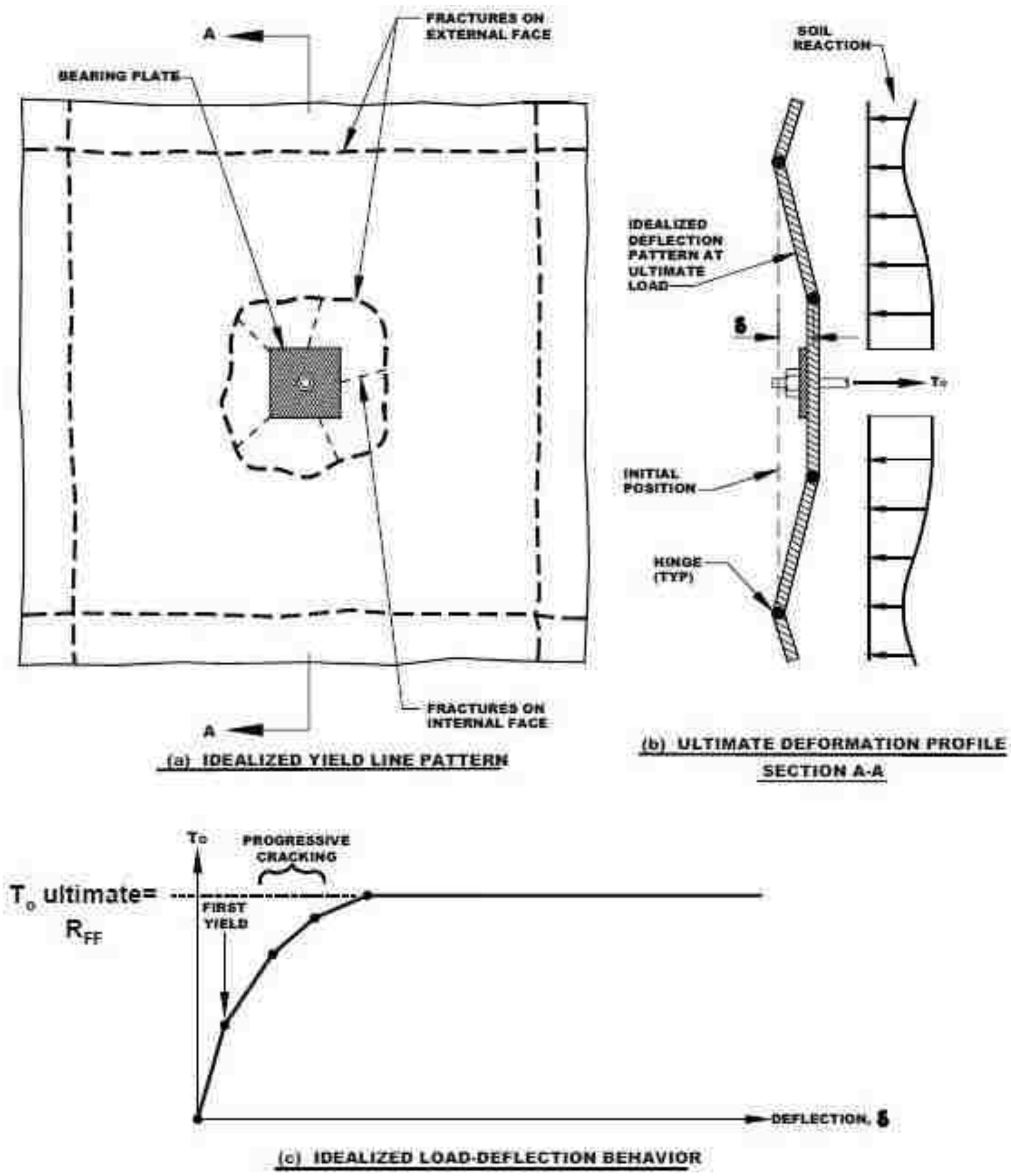


Figure 3.12. Progressive flexural failure in wall (Lazarte et al, 2003)

$$R_{FF}[kN] = \frac{C_F}{265} x(a_{vn} + a_{vm})[mm^2/m]x\left(\frac{S_H h[m]}{S_V}\right) x f_y [MPa] \qquad \text{(Eq. 3.8)}$$

$$R_{FF}[kN] = \frac{C_F}{265} x(a_{hn} + a_{hm})[mm^2/m]x\left(\frac{S_V h[m]}{S_H}\right) x f_y [MPa] \qquad \text{(Eq. 3.9)}$$

Where: C_F = factor that considers the non-uniform soil pressures behind the facing (Byrne et al., 1998); h = thickness of facing (Figure 3.13); d = half-thickness of facing; a_{vn} = reinforcement cross sectional area per unit width in the vertical direction at the nail head; a_{vm} = reinforcement cross sectional area per unit width in the vertical direction at midspan; a_{hn} = reinforcement cross sectional area per unit width in the horizontal direction at the nail head; a_{hm} = reinforcement cross sectional area per unit width in the horizontal direction at midspan; S_H = nail horizontal spacing; S_V = nail vertical spacing; f_y = reinforcement tensile yield strength; and f_c' = concrete compressive strength. (Lazarte et al, 2003)

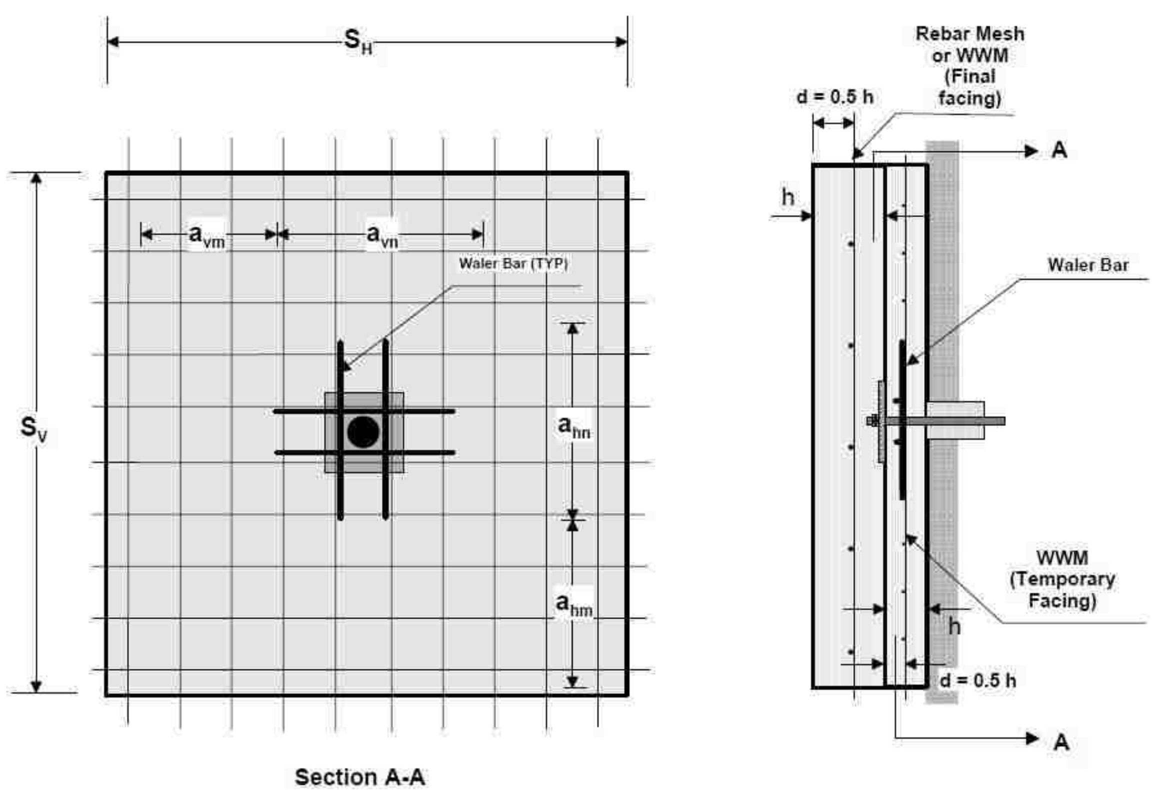


Figure 3.13. Geometry used in flexural failure mode

3.1.5.5.5. <u>Punching shear capacity.</u> Punching shear failure of the facing can occur around the nail head and must be evaluated at:

a. Bearing-plate connection (used in temporary facings), and

b. Headed-stud connection (commonly used in permanent facings).

As the nail head tensile force increases to a critical value, fractures can form a local failure mechanism around the nail head. This results in a conical failure surface are presented by Byrne et al (1998) in the Figure 3.14. This failure surface extends behind the bearing plate or headed studs and punches through the facing at an inclination of about 45 degrees, as shown schematically in Figure 3.14. The size of the cone depends on the facing thickness and the type of the nail-facing connection (in example: bearing-plate or headed-studs).

As is common for concrete structural slabs subjected to concentrated loads, the nail-head capacity, Figure 3.12, must be assessed in consideration of the punching shear capacity, R_{FP}, and can be expressed as:

$$R_{FP} = C_P V_F \qquad \text{(Eq. 3.10)}$$

Where V_F is the punching shear force acting through the facing section and C_P is a correction factor that accounts for the contribution of the support capacity of the soil.

The punching shear force can be calculated considering both SI and English units using standard equations for punching shear. These equations consider the size of a conical failure surface (with diameter D'_C at the center of the facing and height h_c, as shown in Figure 3.14) at the level of the concrete slab as:

$$V_F\,[\text{kN}]\ 330\ f'_c\,[\text{MPa}]\ \pi\ D'_c[\text{m}]h_c[\text{m}] \qquad \text{(Eq. 3.11)}$$

$$V_F\,[\text{kip}]\ 0.58\ f'_c\,[\text{psi}]\ \pi\ D'_c[\text{ft}]h_c\,[\text{ft}] \qquad \text{(Eq. 3.12)}$$

Where D'_C = effective diameter of conical failure surface at the center of section (i.e., an average cylindrical failure surface is considered); and h_c = effective depth of conical surface. (Lazarte, 2003)

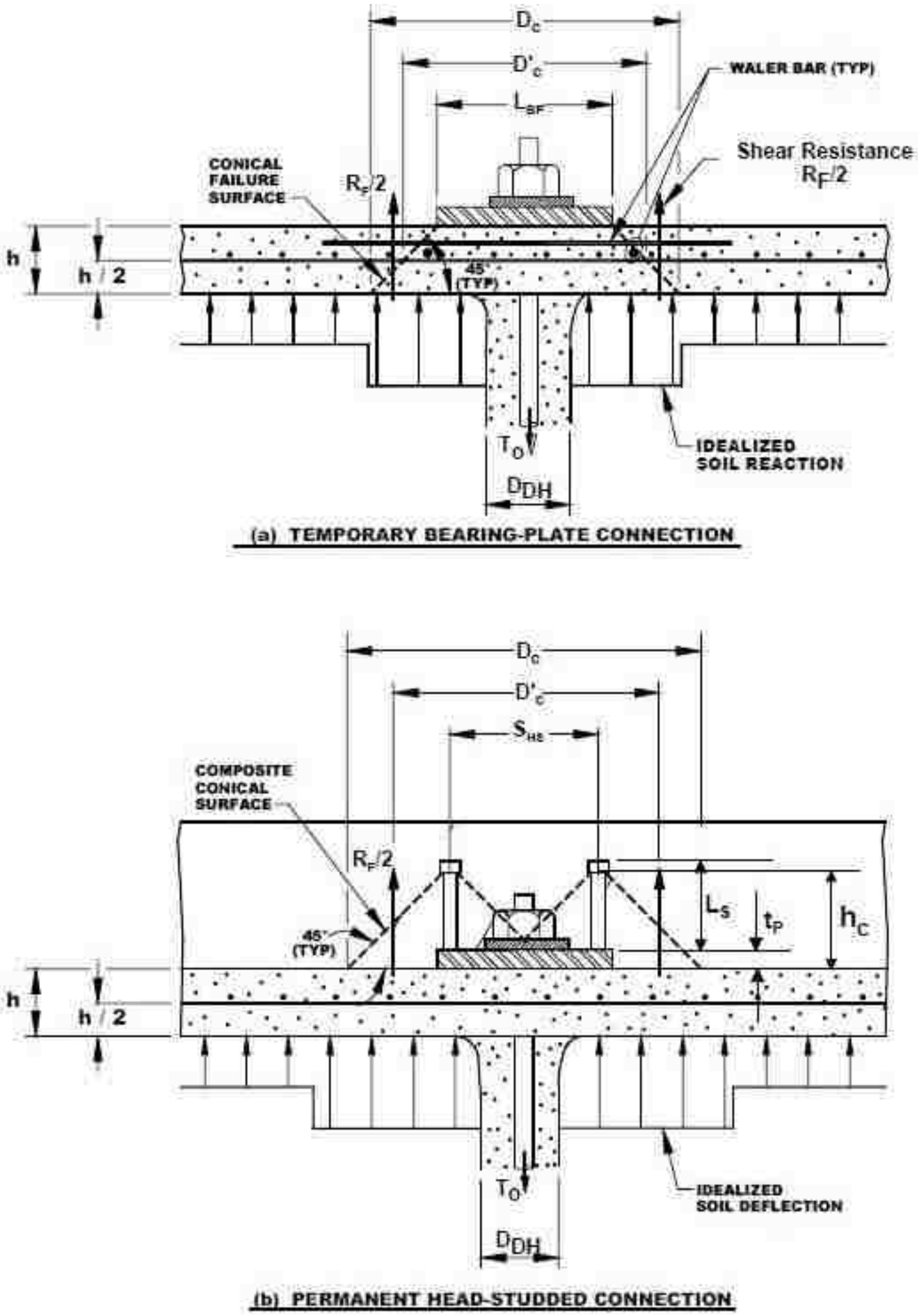


Figure 3.14. Punching shear failure modes (Byrne et al, 1998)

3.1.5.5.6. <u>Deformation behavior of soil nail walls.</u> As mentioned before, soil nail walls deforms gradually with the excavation progress. Soil nail wall tend to deform outwards, the outward movement is initiated by incremental rotation about the toe of the wall, similar to the movement of a cantilever retaining wall. This behavior is presented in the figure 3.15 by Thompson and Miller (1990).

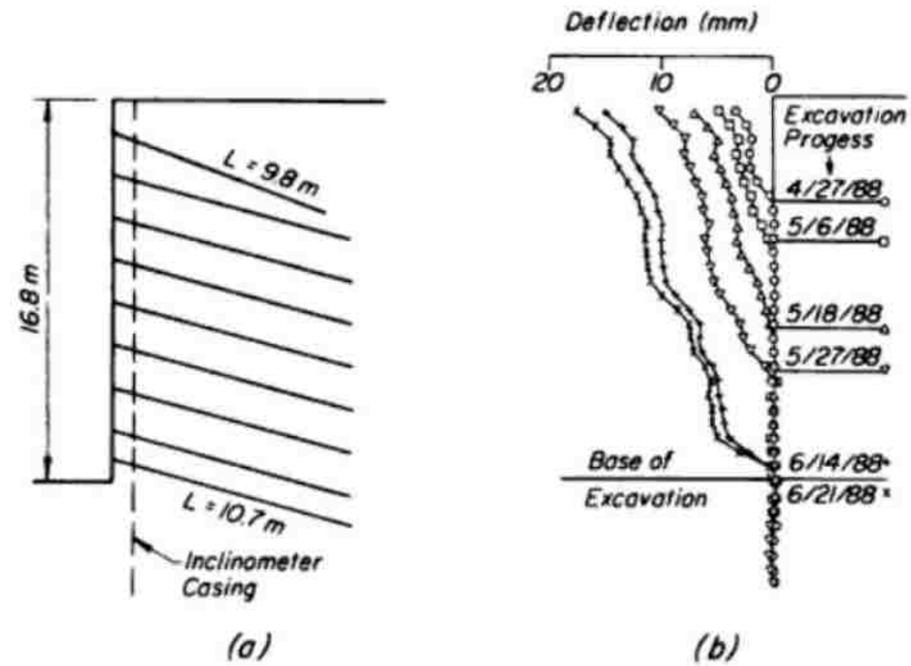


Figure 3.15. (a) Geometry of soil-nailed wall in heavily overconsolidated glacial deposits in Seattle (b) Deflected position of face of wall at various excavation stages

Most of the movement occurs during or shortly after excavation of the soil in front of the wall. Post construction deformation is related to stress relaxation and creep movement, which are caused by post construction moderate increases in tensile force in the soil nail described previously. Maximum horizontal displacements occur at the top of the wall and decrease progressively toward the toe of the wall. Vertical displacements (i.e., settlements) of the wall at the facing are generally small, and are on the same order of magnitude as the horizontal movements at the top of the wall. This behavior is presented in the Figure 3.16 by Plumelle et al (1990). In general, horizontal and vertical displacements of the facing depend on the following factors:

a. Wall height, H, (deformation increases approximately linearly with height);
b. Wall geometry (a vertical wall produces more deformation than a battered wall);
c. The soil type surrounding the nails (softer soil will allow more deformation);
d. Nail spacing and excavation lift heights (larger nail spacing and thicker incremental
e. Excavation lifts generate more deformation);

f. Global factor of safety (smaller FSG's are associated with larger deformation);

g. Nail-length-to wall-height ratio (shorter nail lengths in relation to the wall height generates larger horizontal deformation);

h. Nail inclination (steeper soil nails tend to produce larger horizontal deformation because of less efficient mobilization of tensile loads in the nails); and

i. Magnitude of surcharge (permanent surcharge loading on the wall increases deformation).

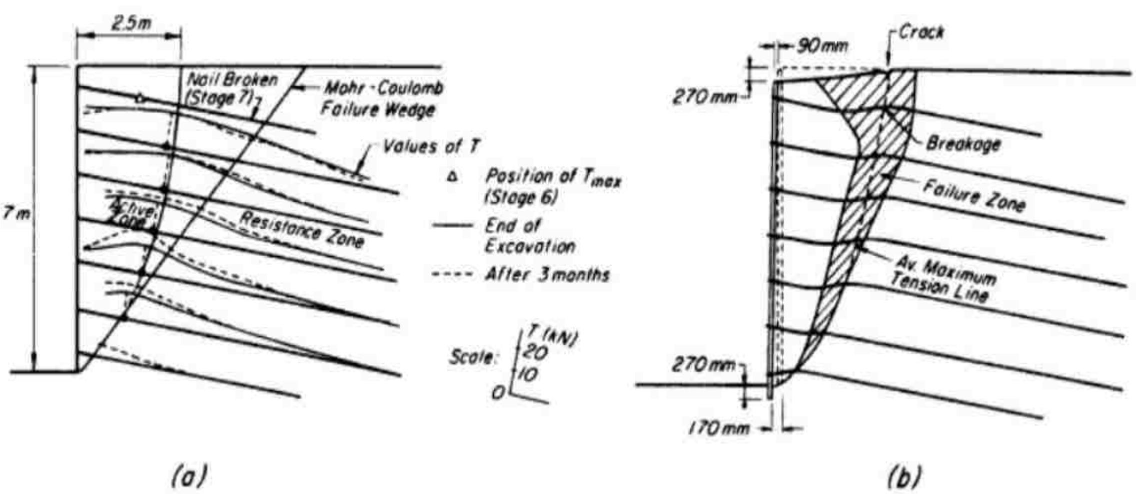


Figure 3.16. (a) Distribution of tensile forces in nails behind experimental wall (b) Distortion of nails behind wall and displaced position of wall at failure

Empirical data show that for soil nail walls with typical L/H between 0.7 and 1.0, negligible surcharge loading, and typical global factors of safety values of 1.5, the maximum long-term horizontal and vertical wall displacements at the top of the wall, δh and δv, respectively, can be estimated as follows:

$$\delta_h = \left(\frac{\delta_h}{H}\right)_i x\ H \qquad \text{(Eq. 3.13)}$$

Where $(\delta_h/H)_i$ = a ratio dependent on the soil conditions "i" indicated in the Table 3.3; and H = wall height. The Figure 3.17 illustrates this values.

Table 3.3. Values of (δb/H)i and C as functions of soil conditions

Variable	**Weathered Rock and Stiff Soil**	**Sandy Soil**	**Fine-Grained Soil**
δ_b/H and δ_v/H	1/1,000	1/500	1/333
C	1.25	0.8	0.7

Also Clough and O'Rourke (1990) propose various types of envelopes of excavations induced ground surface settlements for different soils. Ou and Hseih (2000 and 2005) improve these envelopes using spandrel and concave type curves.

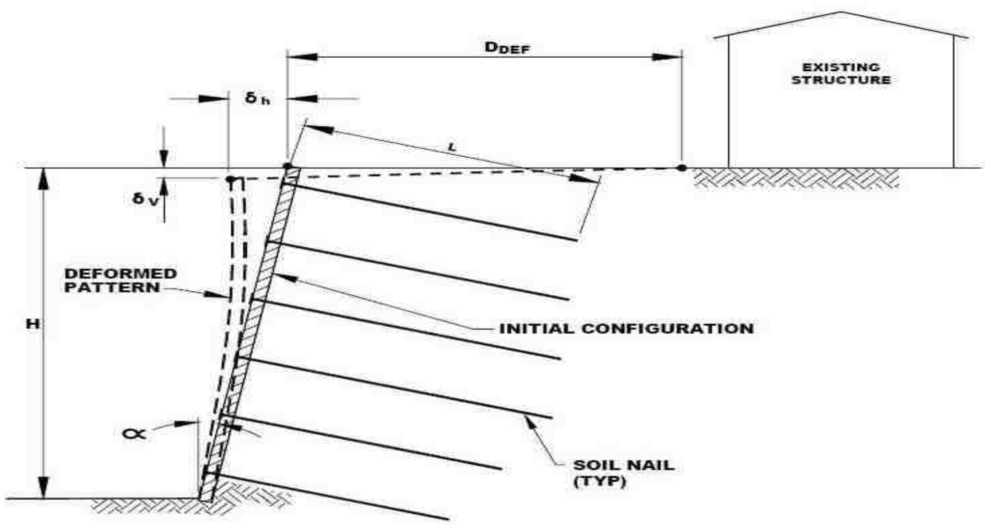


Figure 3.17. Deformation of soil nail walls (Byrne et al, 1998)

3.1.5.6. Drainage design considerations. Is mandatory to provide proper drainage to all soil nail walls. Uncertainties about water flow and seepage due to soil nail wall, barrier, construction, could result in ground water pore pressure increase, so drainage have to be provide in order to reduce this effect. As mentioned above there are many options to for drainage but at least weep holes have to be considered, Figure 3.18 shows typical drainage alternatives for soil nailing walls.

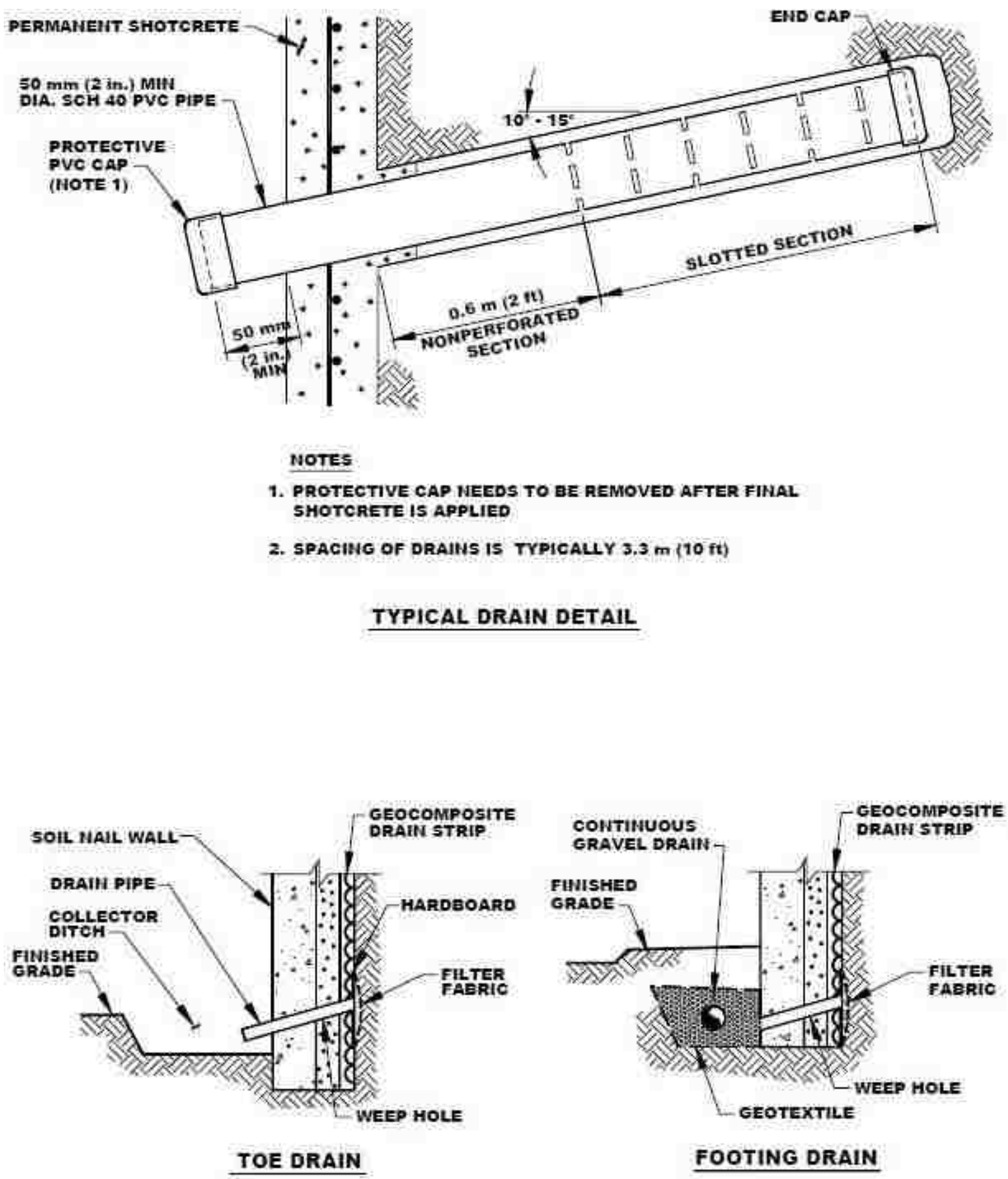


Figure 3.18.Typical drain pipe details to provide groundwater control in soil nail walls (Byrne et al, 1998)

3.1.5.7. Final design of soil nail walls. All the stated above is the base for a full soil nail wall design. In order to outline design methodology a step by step design process is summarized by Lazarte et al (2003) in the Table 3.4.

Table 3.4. Design steps four soil nail walls

Step 1.	Initial soil nail wall design considerations a. Wall Layout (e.g., Wall height, face batter) b. Soil nail vertical and horizontal spacing c. Soil nail pattern on wall face (e.g., square, staggered, other irregular patterns) d. Soil nail inclination e. Soil nail length and distribution f. Soil nail material type (e.g., selection of steel bar grade) g. Selection of relevant ground properties for design
Step 2.	Preliminary design using simplified charts These charts are used to preliminarily evaluate nail length and nail force
Step 3.	Final design a. External Failure Modes 1) Global stability 2) Sliding Stability 3) Bearing Capacity b. Seismic Considerations c. Internal Failure Modes 1) Nail pullout resistance 2) Nail tensile resistance c. Facing Design 1) Nail head load 2) Wall facing type and thickness 3) Facing materials 4) Flexural resistance 5) Facing punching shear resistance 6) Facing head stud resistance 7) Other design facing Considerations
Step 4.	Estimate maximum wall deformation
Step 5.	Other design considerations a. Drainage b. Frost protection c. External loads d. Support for facing dead load

3.1.6. Construction Inspection. Construction inspection consists of two main activities, inspection of construction materials and inspection of construction activities. Materials quality control and assurance is widely known and standardized. Principal aspects that should be inspected are:

Cutting of the slope: Should be a uniform cutting to guarantee the thickness of the wall according to the design.

Drilling: Length according to the final design and correct inclination.

Inclusions: Type of steel according to the design, protection against corrosion if required, correctly centered and with and adequate and functional centralizer, adequate thread or minimum development length, tensile resistance (at least 5% of the inclusions or 1 inclusion for each stratum of soil).

Grouting: Verify grouting mix, grouting from the bottom to the surface in order to expulse drilling detritus, verify complete grouting. Usually, a reduction in the level of grout occurs after a few minutes of the first grouting.

Facing Steel Reinforcement: Wire welded mesh correct position inside the wall, no ground contact, longitudinal reinforcement installation, verify continuity and overlap and steel plate installation.

Facing (Shotcrete): Should be concrete or mortar according to the design. If sampling is required, samples can be taken from the wall. Control of surface finish. Concrete must be cured with water or additives. The use of rebounded concrete is not allowed.

Additional works: Ditches to collect and transport water at the top of the slope. Longitudinal drains to avoid moisture and water pressure. Weep holes.

3.1.7. Load Testing. Soil nails should load tested in the field to verify that the nail design loads can be carried without excessive movements and with an adequate factor of safety. Testing is also used to verify the adequacy of the contractor's drilling, installation, and grouting operations prior to and during construction of the soil nail wall. If ground and/or installation procedures change, additional testing may be required to evaluate the influence on soil nail performance. It is typical practice to complete testing in each row of nails prior to excavation and installation of the underlying row. This requirement of completing all testing in the upper row may need to be relaxed, at the direction of the engineer, for very long walls. If test results indicate faulty construction practice or soil nail Capabilities are less than that required, the contractor should be required to alter nail installation/construction methods. Testing procedures and nail acceptance criteria must be included in the specifications. (Lazarte et al, 2003)

There are three types of load tests:

3.1.7.1. Verification or ultimate load tests. Verification or ultimate load tests are conducted to verify the compliance with pullout capacity and bond strengths used in design and resulting from the contractor's installation methods. Verification load tests should be conducted to failure or, as a minimum, to a test load that includes the design bond strength and pullout factor of safety. (Lazarte et al,2003)

3.1.7.2. Proof tests. Proof tests are conducted during construction on a specified percentage, typically five percent, of the total production nails installed. Proof tests are intended to verify that the contractor's construction procedure has remained constant and that the nails have not been drilled and grouted in a soil zone not tested by the verification stage testing. Soil nails are proof tested to a load typically equal to 150 percent of the design load. (Lazarte et al, 2003)

3.1.7.3. Creep tests. Creep tests are performed as part of ultimate, verification, and proof testing. A creep test consist of measuring the movement of the soil nail at a constant load over a specified period of time. This test is performed to ensure that the nail design loads can be safely carried throughout the structure service life. (Lazarte et al, 2003)

3.1.8. Performance Monitoring. As any structure in civil engineering, performance monitoring is desirable to assess behavior with actual loads, check design assumption and assure safety. The Figure 3.19 shows typical instrumentation options.

For vertical movements inclinometers and/or surveying points can be installed, meanwhile for horizontal movements settlement cells and/or surveying points can be used.

For nail load determination strain gauges attached directly to the reinforcement or load cells with double bearing plate.

Instrumentation shall be accompanied by a monitoring plan, this plan has to include at least parameters to be monitored and limit values.

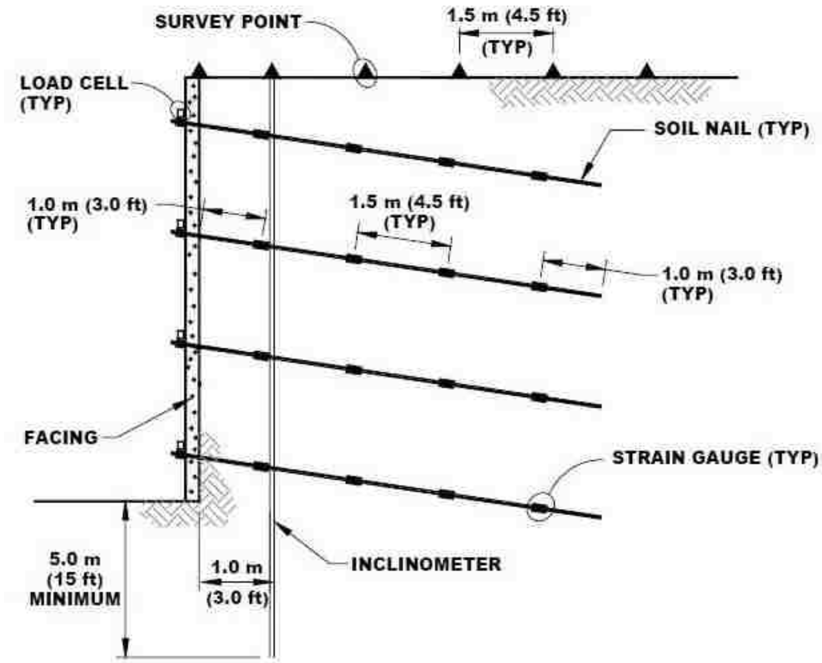


Figure 3.19. Typical Instrumentation (Byrne et al, 1998)

3.2. GROUTING FOR SEEPAGE CONTROL

According to Fenoux (1990), grouting is the technique of introducing into the voids of the ground a self-hardening liquid called grout. Grout is defined as a suspension of cement grains in water (Lombardi, 1997), and is usually described by the water-to-cement ratio (by weight). Currently, there are many grout alternatives besides a water-cement mix, such as acrylate grouts, urethanes, and resins. Unlike Newtonian fluids, such as water, where rheological behavior can be characterized solely by the parameter viscosity, “stable” grout (defined as those that exhibit less than 5 percent of decantation of clear water at the top of a 1000 ml cylinder in 2 hours) slurry behaves as a Bingham fluid during flow, possessing both viscosity and cohesion. While viscosity and cohesion are both flow-resistance parameters, the viscosity governs the rate of flow, while the cohesion governs maximum travel distance (Lombardi and Deere, 1993). The potential improvements of the ground by grouting are: reducing the permeability, reducing the deformability, and increasing the strength. Grouting is usually performed through drilled boreholes, and it can be performed in a single or multiple stages. Also the distance between stages can be variable depending mostly of the influence of the stresses their relationship with permeability (i.e., reduction of the permeability due to increase of the stresses with the increase of depth). Normally the arrays of grouted boreholes are called grout curtains, specifically named cut-off curtains for seepage control.

The use of grouting has almost been considered a “black art” of geotechnical engineering due the relative unfamiliarity of the relationships between key performance parameters such as the mixes, grouting rates, pressures, monitoring and verification process, and its consequential cost uncertainty. Grouting success has largely relied upon the experience of the grouting contractor and observing engineer(s).

Until the mid-1990’s, rock grouting in the U.S. was technologically far behind practices employed in many other parts of the world, and especially in France, Germany, Italy and Switzerland. (Bruce, 2013a)

The great improvement of computers and sensor technology has reached the grouting industry. Since the beginning of the personal computers, PCs, many companies, such as the French company Soletanche-Bachy (formerly Soletanche), developed complex systems for grout monitoring to achieve a more satisfactory performance of the technique. Many improvements and optimizations have since been developed to increase the grout penetration, such as using finer cement, higher pressures, and/or adding plasticizer to the mix.

3.2.1. Grouting Applications. Applications for grouting are a very large list that possess a challenge to select the most suitable for each particular situation, applications includes:

Tunnel Treatment:

Before excavation

Contact

Consolidation

Advance: stage grouting

Repair Works

Deep Shafts

Dams:

Cutoff curtains

Foundation Consolidation

Containment of a reservoir of polluted water to protect the aquifer

Filling Abandoned Quarries Pits

Structures:

Foundations reinforcement

Impermeable barriers for are potentially harmful wastes

Compensation of settlements

3.2.2. Grout-hole Layout. The spacing of the grout-holes depends on the type of soils, the grout used and the objectives of the treatment: the finer the soil grain-size, the smaller the distance between the grout holes. The Table 3.5 shows a suggested layouts in various situations:

Table 3.5. Grouting suggested layouts

Structure	Ground Type	Grout Hole Layout
Grout Curtains	Alluvium	2 rows of grout holes minimum spacing between grout holes: 1.00 to 3.00 m.
	Rock	1 to 3 rows of grout holes spacing between grout holes: 1.50 to 6.00 m.
Mass Grouting	Alluvium	Grout hole layout: 1.00 x 1.00 to 3.00 x 3.00 m
	Rock	Grout hole layout: approximately 3.00 x 3.00 m
Impermeable Foundations	Alluvium	Grout hole layout: 1.50 x 1.50 to 3.00 x 3.00 m
	Rock	Grout hole layout: approximately 3.00 x 3.00 m

(Soletanche-Bachy, 2011)

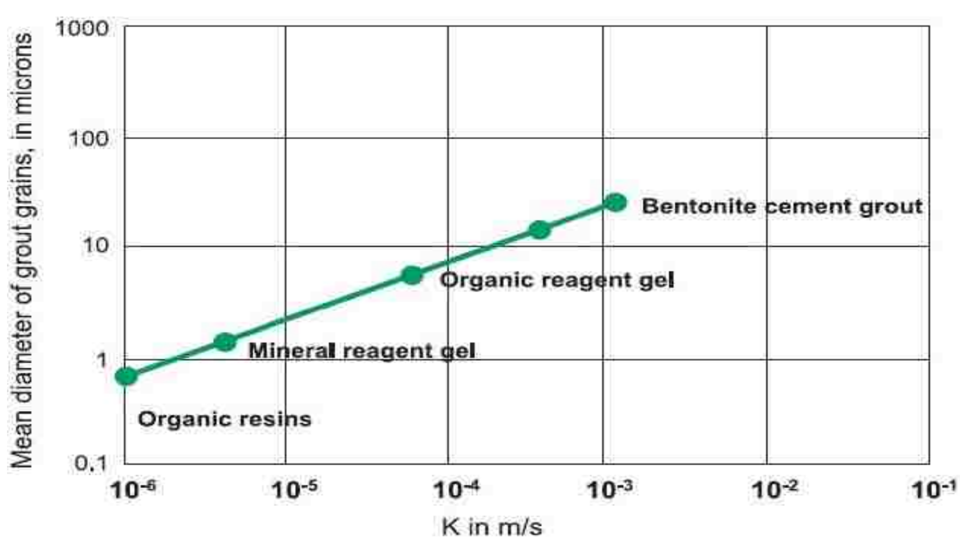
3.2.3. Grouts. Grout material depends on its final function, ground characteristics and its installation conditions. Some of the most common types of grouts are:

Liquid grouts: their ability to penetrate is a function of their viscosity, and the change in viscosity over time. These are the most broadly used and basically consist of cement-bentonite mixes; normally some additives are required due to pumpability requirements.

Suspensions: in addition to viscosity, these grouts possess rigidity or cohesion, which restricts their radius of action. The voids or pores that can be sealed with these grouts depend on the size of the grains in suspension. Broadly, it is considered that there should be a minimum ratio of three between the size of the void and the grain size of the suspension. The stability of a suspension (decantation, pressure filtration) is an important grouting parameter. An unstable grout behaves in the same way as hydraulic fill where the water, which provides the mobility of the mix, progressively bleeds out.

Mortars: mortar grouts have high rigidity and are used for filling large voids and cavities, or for grouting where soil displacement is the objective: solid or compensation grouting.

Grout penetrability versus soil permeability is shown in the Figure 3.20.



(Adapted from Soletanche-Bachy, 2011)

Figure 3.20. Grout penetrability limits based on soil permeability

3.2.4. Grouting Volume. Grouting effects can spread widely from the injection point, so volume estimation result difficult. Grout consumption depends on the pressure used but mostly from the type of ground. The Table 3.6 gives indicative grout percentages of volume required according to ground and treatment type:

Table 3.6. Range of grouting volumes

Ground and Treatment Type	Range of Grout Consumption
Sands and gravels	25 – 35% soil volume
Fine Sand	35 – 45% soil volume
Fisured Rock	5 – 15% soil volume
Base slab in chalk	8 – 25% soil volume
Sound Rock by Hydrofracturing	10 – 20% soil volume

(Adapted from Soletanche Bachy, 2011)

3.2.5. Traditional Grouting Methods. The older grouting methods used in the USA from 1920 to 1980, and in some cases also to the present time (i.e., since about the time of the grouting of the Hoover dam until today) can have major flaws and gaps. The main aspects of these older methods are:

1. Drilling of vertical holes to a target depth;
2. A "single row" curtain;
3. Relatively low grouting pressures;
4. Use of "thin" mixes;
5. "Thin to thick" mix grouting method;
6. Drilling higher order holes to sometimes "ridiculously close centers;" and
7. Use of thin mixes injected in karst cavities (Lombardi, 2011).

3.2.6. Grouting for Seepage Control. The use of grouting for seepage control and to reduce ground consolidation has shown to be a very effective solution, but in many cases the implementation poses a challenge. It is very important to consider some basic aspects of grout application, such as ground feasibility (i.e., soil and/or rock type), expansion due to grouting, durability of the beneficial effects, and the economics of the treatments. The main features to be specified in a grout curtain are the grout mix; borehole spacing and depth; grouting sequence; volume and pressure limits; and process monitoring.

3.2.7. GIN Grouting Method. The Grouting Intensity Number, GIN, method presents a solution for grouting control to achieve a satisfactory performance level, even with the inherit uncertainty of the technique itself. The method was developed specifically for dam grout curtains, but also can be applied for consolidation grouting. The aim of the method is reduce unwanted grout travel distance in large fissures by controlling grouted volume, and increasing grout penetration in small fissures by increasing pressures.

The mix used for the GIN is rich in cement, using superplasticizer to reduce bentonite and, therefore, reducing mix cohesion. Also, this thick mix exhibits greater strength, less shrinkage, less porosity, better binding to rock, lower permeability, higher chemical resistance, greater density, and predictable grouting results.

The GIN method has these particular characteristics:

A single, stable grout mix for the entire grouting process (water: cement ratio by weight of 0.67 to 0.8:1) with superplasticizer to increase penetrability;

A steady low to medium rate of grout pumping which, across the grouting stages, leads to a gradually increasing pressure as the grout penetrates further into the smaller rock fractures;

The monitoring of pressure, flow rate, volume injected, and penetrability versus time in real-time by computer; and

The termination of grouting when the grouting path on the displayed pressure versus total volume (per meter of grouted interval) diagram intersects one of the curves of limiting volume, limiting pressure, or limiting grouting intensity, as given by the selected GIN hyperbolic curve (Lombardi and Deere, 1993).

Basically, the Grouting Intensity Number is the product of the grouting pressure and the grouted volume that represents the energy expended during the grouting process. The typical units for the GIN are bar–liters/m. The use of a constant GIN value is recommend for both wide and thin fissures. A constant GIN value, when plotted on a pressure versus volume graph, yields a hyperbolic curve: the higher the grouting intensity or GIN value, the greater the distance of the curve from the origin. The GIN curve thus completes the missing ingredient for joining the pressure and volume limits (Lombardi and Deere, 1993). The Table 3.7 presents the five grouting intensities that are recommended:

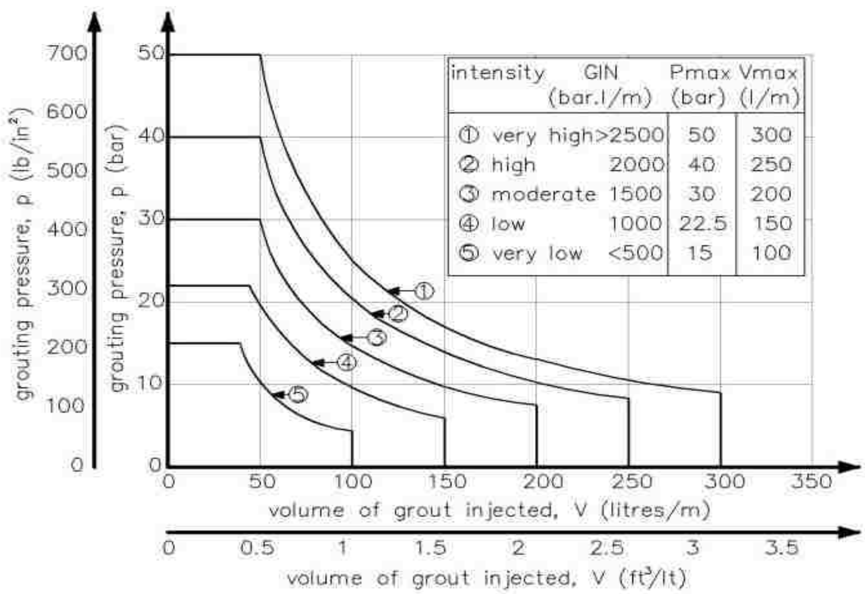
Table 3.7. Intensity and grouting parameters

Intensity	GIN (bar*l/m)	Pressure (bar)	Volume (l/m)
Very High	2,500	50	300
High	2,000	40	250
Moderate	1,500	30	200
Low	1,000	22.5	150
Very Low	<500	15	100

(adapted from Lombardi and Deere, 1993)

The GIN method limiting grouting envelopes, in Figure 3.21, help prevent the wasting of grout or damage (e.g., hydrofracture) from grouting pressures that are too high. If relatively large fissures are present, the grout will travel at lower pressures. If a contractor was just watching pressure, large volumes of grout could get pumped into areas where it is not needed. Conversely if only small fissures are present, the grout will need higher pressures to flow. If a contractor was only watching the grout volume, high grout pressures could build up and cause additional rock fracturing, defeating the purpose of the grout curtain. In the GIN method, the grout volume, pressure, and volume x pressure monitored. If any of these values hits the limit for the chosen envelope, then the grouting is stopped for that stage.

For example, if we have a rock formation with small, medium, and large fissures that need to be grouted, then during the first grouting stage, the grout will most easily flow into the large fissures at lower pressures until the volume limit is reached and grouting is stopped. When that grout has set and the large fissures are filled, the second stage grouting will require more pressure to travel into the medium fissures. Once the grout volume x pressure hits the limiting curve, the grouting will stop. When the second stage grout has set, the third stage grouting will require even more pressure to push grout into the small fissures. Once the maximum pressure is reached, the grouting will stop and all three sets of fissures should be adequately filled.



(adapted from Lombardi and Deere, 1993)

Figure 3.21. Limiting grouting envelopes

For intensity selection, a trial area is recommended, and a ‘moderate intensity’ can be used as a starting trial. Grouting pressure will increase with tighter borehole spacing As rule of thumb, a spacing that yields in about a 50% take reduction (typically 25%-75%), by volume, should lead to satisfactory performance. It is important to note that to some extent the grouting is a self-regulating procedure.

A summary of the GIN method is presented by Lombardi (Lombardi, 2003) as follows:

1. Define the scope of grouting
2. Design the grouting process

3. Determine the best mix
4. Use a single mix (the best)
5. Define the GIN limits
6. Confirm by tests
7. No water pressure tests
8. Split-spacing as self-adaptive process
9. Variable stage length
10. Previous saturation of dry rocks
11. New boreholes steered by grout take
12. Real-time grouting control

The GIN theory most probably has worked well and was an excellent option in the grouting interregnum in developing countries during the latter decades of the 20th Century. (Bruce, 2013b)

3.3. VIBROREPLACEMENT FOR LIQUEFACTION RISK MITIGATION

3.3.1. Liquefaction. Liquefaction is a very complex topic that groups several phenomena, normally its effects are catastrophic to civil edifications due to its high deformations. Liquefactions is associated with earthquakes due to the nature of the load produced by them, rapid and cyclic loading.

During an earthquake, the application of cyclic shear stresses induced by the propagation of shear waves causes the loose sand to contract, resulting in an increase in pore water pressure. Because the seismic shaking occurs so quickly, the cohesionless soil is subjected to an undrained loading (total stress analysis). The increase in pore water pressure causes an upward flow of water to the ground surface, where it emerges in the form of mud spouts or sand boils. The development of high pore water pressures due to the ground shaking and the upward flow of water may turn the sand into a liquefied condition, which has been termed liquefaction. For this state of liquefaction, the effective stress is zero, and the individual soil particles are released from any confinement, as if the soil particles were floating in water (Ishihara, 1985).

Liquefaction phenomena that result from this process can be divided into two main groups: flow liquefaction and cyclic mobility. In the field, flow liquefaction occurs much less frequently than cyclic mobility but its effects are usually more sever. Cyclic mobility, on the other hand, can occur under much broader range of soil and site conditions that flow liquefaction; its effects can range from insignificant to highly damaging. (Kramer, 1996)

3.3.2. Liquefaction Assessment. As in many engineering analyses, liquefaction assessment is based in the ratio between resisting parameters and the driving forces. Simplified procedures of assessment have been developed from empirical evaluations of field observations and field and laboratory test data. Different approaches as stress, strain or energy methods are available with a general trend of use in situ testing. Youd and Idris in 2001 summarized the procedures presented in proceedings from the 1996, Northwestern Center for Engineering Education Research, NCEER 1996, AND 1998 NCEER / National Science Foundation, NSF, Workshops on Evaluation of Liquefaction Resistance of Soils presenting a simplified procedure with uniform nomenclature as shown below.

3.3.2.1. Cyclic Stress Ratio (CSR) and Cyclic Resistance Ratio (CRR). As As mentioned above, calculation of two variables is required for evaluation of liquefaction resistance of soils: the seismic demand on a soil layer, expressed in terms of CSR; and the capacity of the soil to resist liquefaction, expressed in terms of CRR. The ratio between these two represents the factor of safety against liquefaction.

$$\text{Factor of Safety} = CRR/CSR \qquad \text{(Eq. 3.14)}$$

CRR is function of geologic history (deposit type, age, OCR), soil structure (relative density, clay content), groundwater conditions. Evaluation of CRR and CSR as summarized by Youd and Idris in 2001 is based in situ tests:

SPT blow count (N)

Corrected blow count

Need fines content

Corrected clean sand blow count – $N_{1(60)CS}$

The Figure 3.22 shows the SPT clean-sand base curve for magnitude 7.5 earthquakes with data from liquefaction case histories.

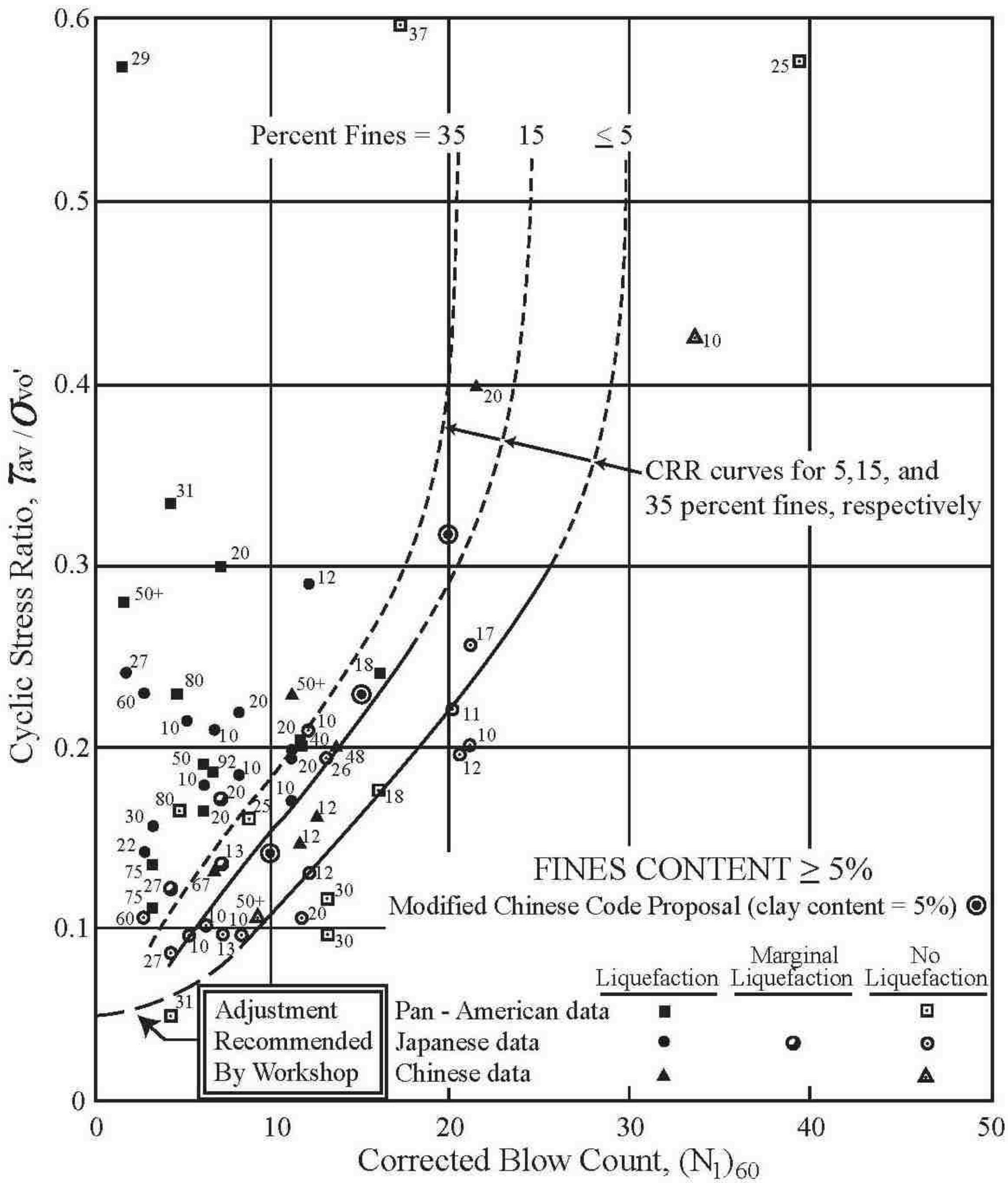


Figure 3.22. SPT Clean-sand base curve for magnitude 7.5 earthquakes with data from liquefaction case histories (modified from Seed et al. 1985)

The Figure 3.23. shows the curve recommended for calculation of CRR from CPT data along with empirical liquefaction data from compiled case histories.

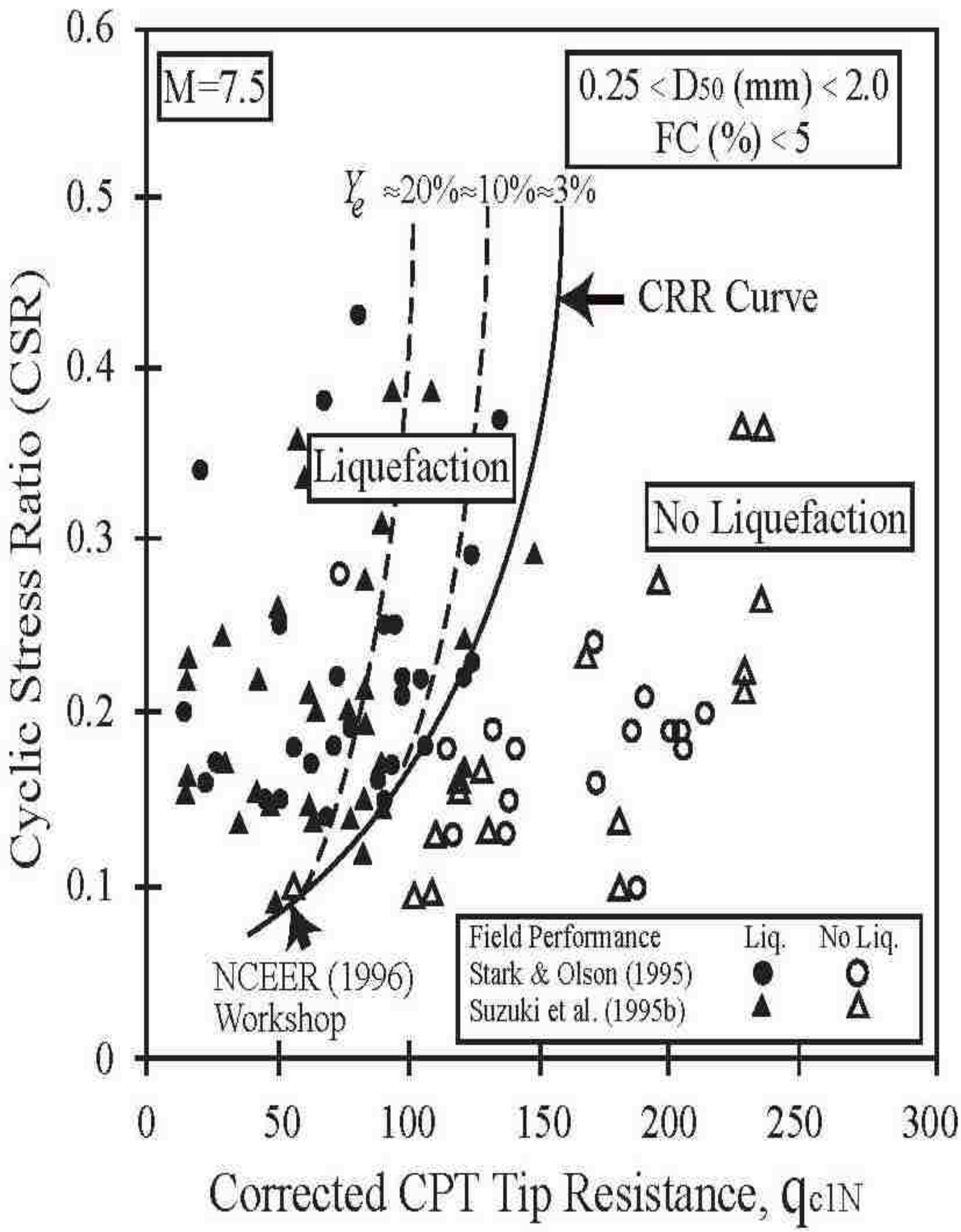


Figure 3.23. Curve recommended for calculation of CRR from CPT data along with empirical liquefaction data from compiled case histories (Robertson and Wride, 1998)

The Figure 3.24 shows the liquefaction relationship recommended for clean, uncemented soils with liquefaction data from compiled case histories.

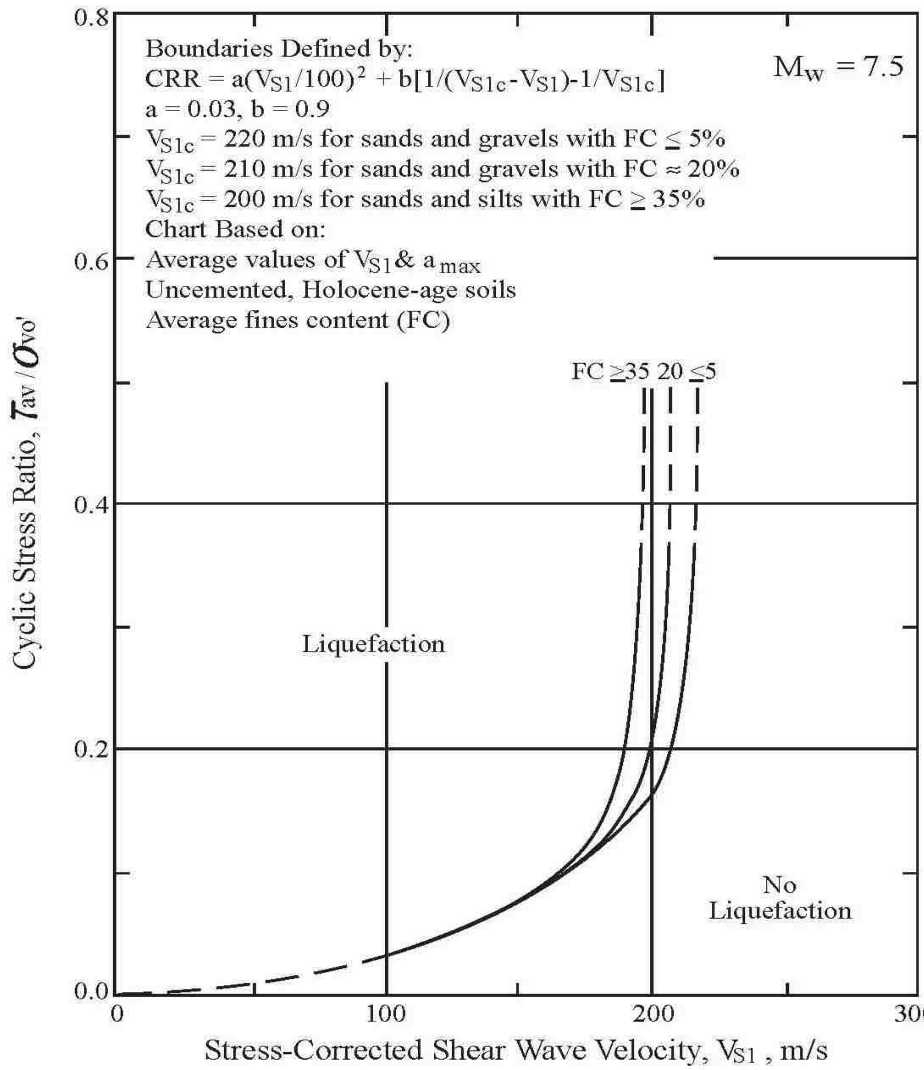


Figure 3.24. Liquefaction relationship recommended for clean, uncemented soils with liquefaction data from compiled case histories (Andrus and Stokoe, 2000)

3.3.2.2. Magnitude Scaling Factors (MSFs). The clean-sand base or CRR curves in Figures 3.21 (SPT), 3.22 (CPT), and 3.23 (V_{S1}) apply only to magnitude 7.5 earthquakes. To adjust the clean-sand curves to magnitudes smaller or larger than 7.5, Seed and Idriss (1982) introduced correction factors termed ‘‘magnitude scaling factors (MSFs).’’ These factors are used to scale the CRR base curves upward or downward on

CRR versus $(N_1)_{60}$, qc_{1N}, or V_{S1} plots. Conversely, magnitude weighting factors, which are the inverse of magnitude scaling factors, may be applied to correct CSR for magnitude, Figure 3.25. Either correcting CRR via magnitude scaling factors, or correcting CSR via magnitude weighting factors, leads to the same final result. (Youd and Idris, 2001).

$$FS = (CRR7.5/CSR)MSF \qquad \text{(Eq. 3.15)}$$

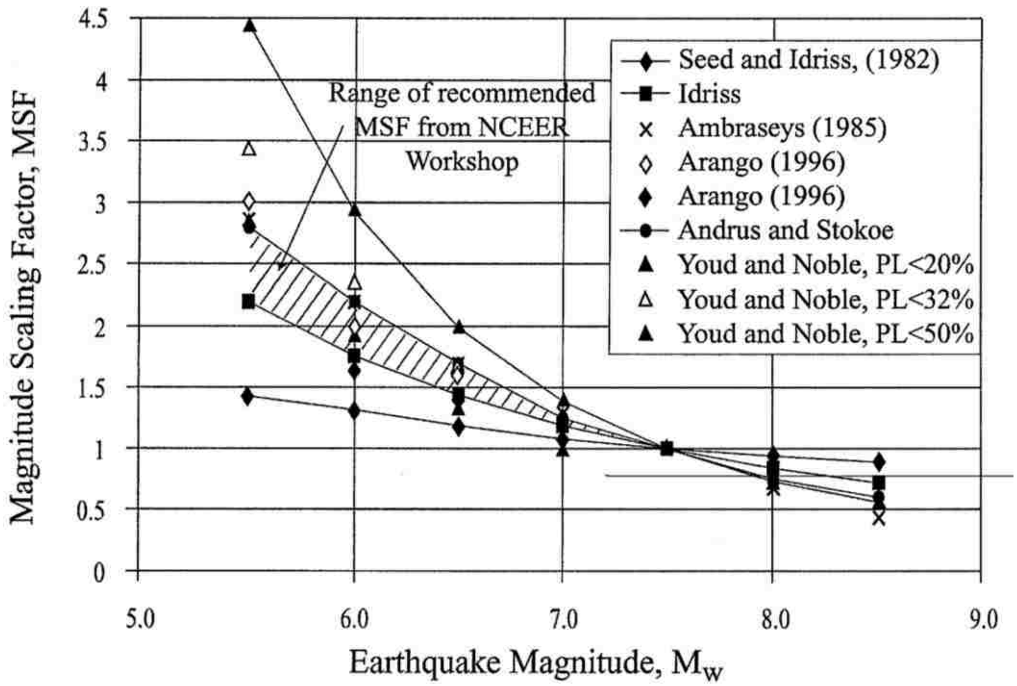


Figure 3.25. Magnitude scaling factors derived by various investigators (Youd and Noble, 1997)

3.3.3. Liquefaction Mitigation. Many alternatives for liquefaction risk mitigation are available, a suitability analysis has to be performed for every case. This analysis should include soils characterization, equipment, materials and contractors availability. In general mitigation has three approaches increase strength (increase CRR, i.e. Ground improvement (densification or grouting)), decrease driving stress (decrease CSR, i.e. Reinforcement / Shear reinforcement with 'stiffer' elements within soil mass) and decrease excess pore pressure quickly (i.e. Reduce drainage path distance with tightly spaced drains).

Ground densification techniques increases cyclic shear strength, CRR, by increasing relative density of cohesionless materials. These techniques have several advantages as field verifiability conducting field testing before and after treatment, have been used for a long period passing through several large magnitude earthquakes. Also several peer-reviewed documents describing the methods, efficiency, and mechanics of densification have been published.

Some methods of densification are:

Vibrocompaction

Vibroreplacement

Dynamic Compaction

Blasting Compaction

Compaction Grouting

Ground reinforcement techniques reduce cyclic shear stress applied to liquefiable soil by installing 'stiffer' elements within soil matrix that attract stress. It can be used in non-densifiable soils (silts, silty sands). Design Methodology I based in a shear stress reduction factor (K_G) as presented by Baez and Martin in 1993 with an area inclusion factor or area replacement ratio.

Some methods of reinforcement are:

Deep soil mixing

Stone Columns

Rammed Aggregate piers

Jet Grouting

Ground drainage techniques limit excess pore pressure increase and duration of increased pore pressure during cyclic shearing by providing short drainage paths in cohesionless materials.

Some methods of drainage are:

EQ drains / Wick drains

Vibro replacement, Stone columns (additional feature).

3.3.4. Vibroflotation. Vibroflotation techniques groups two ground improvement/stabilization techniques vibrocompaction and vibroreplacement. Vibroflotation principle is the introduction of vibratory energy into the ground to cause particles to rearrange themselves into tighter configurations. Vibratory ground improvement methods have been used over the years in different ways; from strength improvement (bearing capacity increase), settlement control, consolidation rate acceleration, seismic remediation, slope stability and construction time reduction. The application of each technique is directly related to the nature of the soil, cohesive or cohesionless. Vibrocompaction is suitable for cohesionless soils with fines less than the 10%. Meanwhile vibroreplacement, normally referred as Stone Columns, is suitable for soil with a fine content greater than 10%. For this purpose Brown (1977) developed a chart based mostly in the soils grain size distribution. This chart is presented in the Figure 3.26 and has the following zones.

Zone A: The soils of this zone are very well compactable.

Zone B: The soils in this zone are suited for Vibro Compaction. They have a fines content of less than 8 to 10 %.

Zone C: Compactable. Stone backfill is needed if the fines content is higher than 10%.

Zone D: Stone columns are a solution for a foundation in these soils. There is a resulting increase in bearing capacity and reduction on total and differential settlements.

Vibration by itself is not effective on cohesive soils, as the energy is typically absorbed. The solution for cohesive soils is stone columns, in which the soil is excavated out and replaced by stone and gravel forming a pillar under the ground. The goal of adding these pillars is to replace the existing weaker soils with the more competent constituents of the columns, typically to the effect of 15% to 35% of the affected area (Barksdale & Bachus, 1983). The columns consequently help to increase the bearing capacity of the site and reinforce the soil reducing liquefaction risk. Stone columns also function similar to sand or wick drains adding a radial component to drainage, accelerating the rate of consolidation. Lastly, stone columns help in slope stability by providing high friction angles and thus high shear resistance along potential slip surfaces.

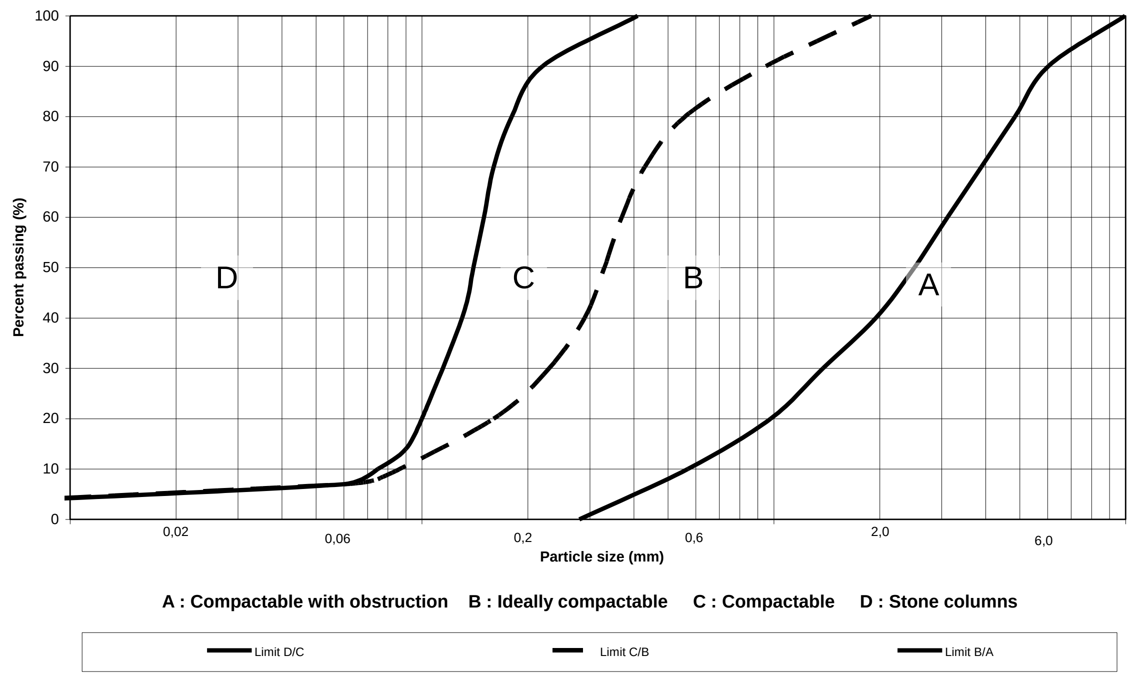


Figure 3.26. Vibrocompactability chart (modified from Brown, 1977)

3.3.5. Vibroreplacement. Vibro replacement is part of the deep vibratory compaction techniques whereby loose or soft soil is improved for building purposes by means of special depth vibrators. These techniques as well as the equipment required are comprehensively described by several authors and mainly by specialized contractors.

Contrary to vibrocompaction which densifies non cohesive soil by the aid of vibrations and improves it thereby directly, vibroreplacement improves non compactible cohesive soil by the installation of load bearing columns of well compacted, coarse grained backfill material. In many practical cases the reinforcing effect of stone columns installed by vibroreplacement is superposed with the densifying effect of vibrocompaction, i.e. the installation of stone columns densifies the soil between. Vibro replacement is suitable particularly for ground improvement in seismic areas since stone columns possess certain flexibility on one side and prevent liquefaction on the other side. (Priebe, 1995)

3.3.6. Design of Stone Columns. The determination of the quantities of stones to be installed and compacted to reach the required final improvement is based on the well-known and worldwide accepted calculation methods used for stone columns. As mentioned before, in 1995 Priebe presented a detailed design procedure that is globally accepted. In general the design process can be summarized as follow:

a. Estimate the settlement for the proposed loading conditions for the unimproved ground using conventional settlement calculations.
b. Determine the reduction of settlement that is required to meet the design requirements. This reduction factor which is expressed as a ratio of the amount of settlement of the unimproved soils to the amount of settlement of the improved soils is referred to as "settlement ratio," or "improvement factor." This concept was developed by Priebe.
c. Determine, based on contractor's experience and published empirical data, if stone columns can provide the required reduction of settlement. Typically, settlement ratios are between 2 and 3 (i.e., settlement can be reduced by a factor of between 2 and 3).
d. Determine the area replacement ratio (stone column area divided by the tributary area of the stone column) necessary to provide the required reduction of settlement.
e. Determine the stone column length, diameter and spacing. The stone column length is determined from evaluation of the settlement calculations. Stone column diameter and spacing are determined by contractor experience.
f. Assess the load-carrying capacity of the stone columns.

3.3.7. Constructive Methods.

3.3.7.1. Transfer platform. If the foundation element does not have enough inertia to distribute load bearing in a homogeneous way on the initial column grid, it is necessary to add a load transfer platform between the foundation elements and the treated soil. The purpose of this platform is to improve the load bearing distribution.

If the stone columns are being used for their draining properties, a drainage layer (with outlet) should be added at the top of the columns.

The minimum thickness for a gravel load transfer platform to distribute the load is 40 cm.

The load transfer platform can be partially or entirely installed before the stone columns and therefore can be used as a work platform.

However, any re-grading, final compacting, and re-treatment of subgrade, as well as any gravel additions to increase thickness should be performed after the installation of the stone columns so that the characteristics in compliance with the project remain consistent.

3.3.7.2. Wet, top feed method (replacement and displacement). In this technique, jetting water is used to remove soft material, stabilize the probe hole, and ensure that the stone backfill reaches the tip of the vibrator. This is the most commonly used and most cost-efficient of the deep vibratory methods. However, handling of the spoil generated by the process may make this method more difficult to use on confined sites or in environmentally sensitive areas.

3.3.7.3. Dry, bottom feed method (displacement). This technique uses the same vibrator probes as standard Vibro-Replacement Stone Columns, but with the addition of a hopper and supply tube to feed the stone backfill directly to the tip of the vibrator. Bottom Feed Vibro-Replacement is a completely dry operation where the vibrator remains in the ground during the construction process. The elimination of flushing water in turn eliminates the generation of spoil, extending the range of sites that can be treated. Treatment is possible up to a depth of 80 feet and is not inhibited by the presence of groundwater.

3.3.8. Quality Control and Acceptance.

3.3.8.1. Calibration tests (trial area). At the start of any stone column construction project, the contracting company must carry out calibration tests to validate the choice of material and verify the compliance of soil reactions with expected behavior (depths attained, consumption, possible swelling, effects from vibrations, etc.).

If the preliminary soil study shows remarkable heterogeneity in the depths, nature or characteristics of the layers to be treated, calibrations should be carried out for each of the different areas in question. These tests are preferably carried out in the vicinity of the soil sampling or borings for the geographical study.

3.3.8.2. Acceptance tests. Unless otherwise specified by the contractor, these tests are to be carried out by the project construction company and include:

Diameter verification

Columns outside the building footprint can be side-stripped at different layers deep enough for diameter verification. For feasibility reasons, this excavation is usually done from the top of the column to a minimum depth of 1.00m beneath the platform.

Checking continuity

Static cone penetration tests are suited to this verification, though dynamic CPTs can also be carried out.

Compaction verification

This verification is carried out with a static CPT. This test must be done down to 1 m below the tip of the column except in the case of refusal on the underlying layer.

In some cases complications can arise to make these tests difficult to carry out:

Blocking due to large pieces of column material

Deviation of the drill pipe string, which can slip out of the column

If this occurs, the contractor must provide the recorded data for the column in question and suggest a new quality control plan.

Load test

This load test is done at 1.5 times the service limit state load, SLS, for column service load increment, QN, on one column at the site. The load test requires installing a footing on the top of the column, preferably leveled off under the load transfer platform. The surface of the footing should be less than 2.5 times the planned column section. Testing frecuency is presented in the Table 3.8. (Aguado et al, 2011)

Table 3.8. Test frequency

Method	**Wet**		**Dry**	
	Recorded	**Unrecorded**	**Recorded**	**Unrecorded**
Checking diameter	1 per set of 50 columns up to 100, beyond that at least 3			
Checking continuity	1/50	1/20	only if an anomaly is detected	1/50

Table 3.8 Test frequency (cont.)

Method	**Wet**		**Dry**	
	Recorded	**Unrecorded**	**Recorded**	**Unrecorded**
Compaction verification	1/80 under concrete slab or raft foundation + 1/20 underground mass with a minimum of 5			
Load test*	1 test up to 800 m and 1 per section beyond 800 m.		1 test up to 2000 m and 400 columns, and at least one more beyond 2000 m.	

* For construction sites with less than 1,000 m of stone columns installed with the dry method (800 m by wet method), a load test may not be performed, but in this case the allowable stress must be reduced by a factor of 1.5. (Aguado et al, 2011)

4. CASE STUDIES

Case studies present a detailed description of an engineering project, including the most relevant information that lead to the performance of the solution adopted. Case studies are very important for the progress of the practice and an excellent opportunity to share experience and knowledge.

The geotechnical issues in Guatemala are not new, several recent tragedies related to ground failures have occurred in the recent past. Some examples are: the 273 deaths due to a landslide in 2015 in El Cambray residential project, the 5 deaths during the failure of an excavation for a shopping center in 2012 in zone 16, the 10 deaths due to a landslide that buried a bus in the CA1 highway in 2010 and the 5 deaths during the failure of an excavation for an office building in zone 13 in 2010. These tragic failures could yield many lessons learned about what is totally forgotten in construction projects, such as subsurface investigation and characterization, slope stability analysis, protection for temporary excavations, excavation limiting heights, or misuse of soil nailing walls. Despite the fact that these failure events were reported to the news, the technical information is restricted or not available mostly due to liability reasons.

The case studies presented herein were selected based on its relevance to ground stabilization, geotechnical conditions, and the availability of data and project information, However, none of the cases are an example of a failure or limited performance, where most of the learning can take place using principles of forensic engineering. In the selection process, it was also apparent that owners and local engineers were apprehensive to disclose information of failed projects or those that had compromised performance. Therefore, the selected case studies represent projects performed in Guatemala as stories of success. These cases leave useful information of the different solutions adopted for each situation, proving that good geotechnical solutions are possible in Guatemala. The following case studies are focused in the performance of ground stabilization techniques used between the years 2000 and 2015 in Guatemala, Central America.

4.1. PERFORMANCE OF SOIL NAILING IN VOLCANIC SOILS

4.1.1. Introduction. The soil nailing technique intends to reinforce soil “in situ” since its first applications in tunneling stabilization. Soil nailing combines three elements, inclusions (passive anchors), facing and soil itself. Soil nail walls increase the soil mass strength due to inclusion of passive anchors. This technique has more than 50 years of application. Its first application in Central America was in Guatemala City in 1991 for Centro Gerencial Las Margaritas, for a temporary excavation support. It was a 17.50 m deep vertical excavation. Since that time soil nail walls have become a widely used technique in Guatemala for cut into natural soil and excavation stabilization.

The aim of this case study is to illustrate soil nailing performance in the volcanic soil of Guatemala City and its effectiveness for increasing the factor of safety against sliding as well as its function as erosion protection. Also present a particular construction methodology in a zone of the project will be presented. The case of study allows access to all the information of the project by means geotechnical investigation and characterization, design, construction and performance monitoring.

Guatemala is particularly vulnerable to the climate change phenomenon. The first storm of the 2010 Pacific hurricane season, Agatha, hit Guatemala in late May. Agatha left 113 fatalities and $932 (USD) million of dollars on losses and damage. Infrastructure was seriously damaged by landslides and mud flows. One of the most important national roads, CA-9 North, was on risk of collapse. A landslide adjacent to the west side of the route was activated, falling over the lower exit lane closing the road and threatening highway global stability. Slope has heights from 8.13 m to 18.70 m and total final length of 120.00 m. One of the principal factor that contribute to the failure was the surface erosion. Additionally adjacent to the slope there was a planned underpass to start its construction few days after the failure. This situation left a soil mass slide, wedge, between the slope face and underpass interior face. The underpass is supported by a soldier pile wall that works as foundation as the same time as retention structure. This situation left a soil mass slide, wedge, between the slope face and underpass interior face that is one of the most interesting conditions of this case study.

The government awarded several emergency contracts to mitigate infrastructure damage. Contracts were design-build, so the final solution was under the contractor's responsibility. After several stabilization and protection options were analyzed, the soil nailing option arose as final solution.

As any geotechnical project, an expedited soil investigation was performed. Exploration and investigation plan included two boreholes with rock and soil core recovery, SPT, soil samples and undisturbed block samples from the slope´s face. A laboratory testing program was conducted including direct shear tests on intact samples. Local geology is composed by an alternation of volcanic genesis materials layers, pumitic tuffs and ashes. A simplified soil profile shows an interbedded silty sand, sands and clayey sands layers. Soil nailing analysis consists of two main failure modes, external and internal, external design refers to global stability and internal design to structural elements within the reinforced zone and wall. Final design was performed using computer software.

The final solution consists of 12.00 m length nails, spaced at 2.00 m in both directions. Shotcrete facing had a thickness of 0.16 m, reinforced with a welded wire mesh and waler bars in both directions. Two rows of 6.00 m length, 75 mm (3") horizontal drains were installed alternated with weep holes. The wall is supported over a 0.50m width strip footing.

Instrumentation and monitoring plan includes the installation of two inclinometers at different locations in order to monitor the slope long term performance.

4.1.2. Site Characterization.

4.1.2.1. Project location. The project site is located in Guatemala City, zone 18, at kilometer 4.5 km CA-9 North route, 100 m west to the "Puente Belize" (bridge), (14°38'56.49"N 90°28'59.04" W), and an elevation of 1451 meters above sea level. A general location of the project is shown in Figures 4.1 and 4.2.

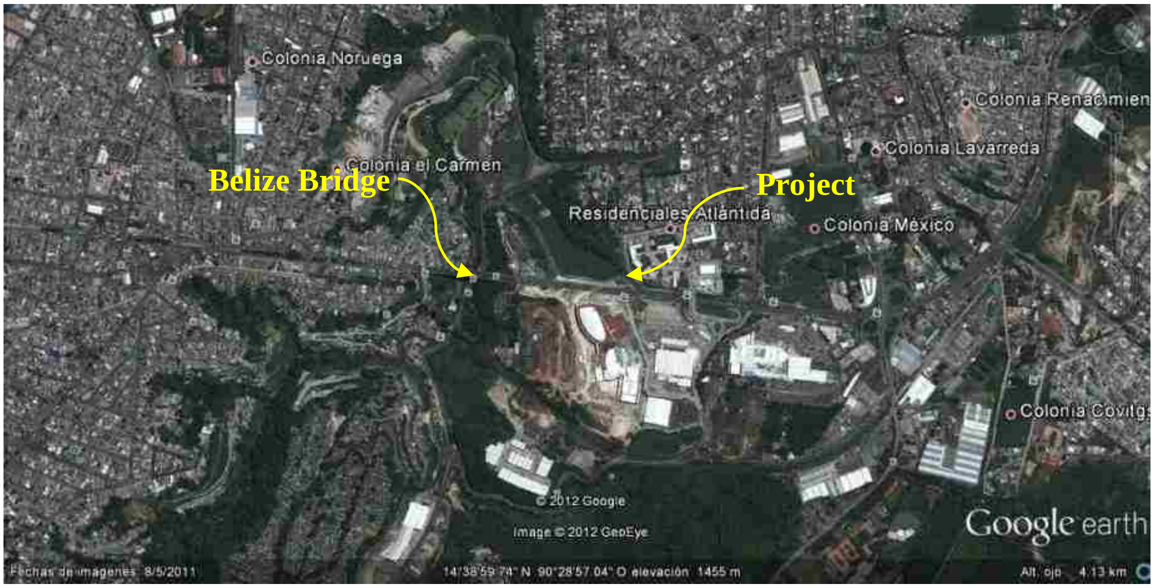
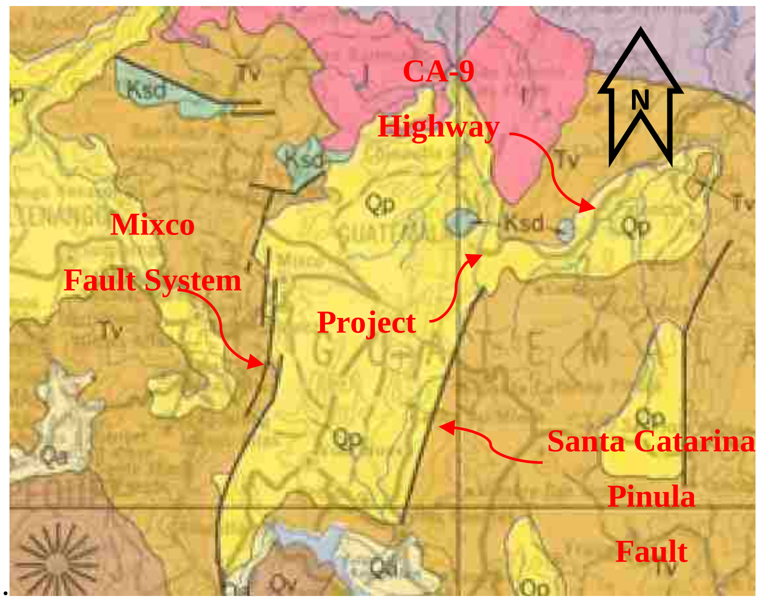


Figure 4.1. Project location

Figure 4.2. Project location

4.1.2.2. Geologic setting. Guatemala City is located on the Asunción valley, which is a result of a geologic graben. This graben is a depressed block of land bordered by parallel faults, bordered by Mixco's fault and Santa Catarina Pinula's fault. Guatemala City's graben is filled with volcanic eruption materials, ashes and tuffs. The project is located in a quaternary igneous pumice fill formation. A geological map of the area is show in Figure 4.3, with the faults and rock types shown.



Legend:
I: Undivided Plutonic Rocks
Ksd: Cretaceous. Neocomian-Campanian carbonates.
Qp: Quaternary. Thick pumice fills.
Qv: Quaternary. Volcanic rocks.
Tv: Tertiary. Undivided Tertiary volcanic rocks.

Figure 4.3. Area geological map (Instituto Geografico Nacional, 1970)

4.1.2.3. Geotechnical investigation. To investigate the surface and subsurface conditions at the site a geotechnical field program was planned. The field geotechnical survey consisted of two boreholes of 23.00 m depth with core recovery and performing Standard Penetrations Tests (SPT) at 1.00 m intervals and four block samples. The four block samples were taken from the slope face and were taken to the laboratory for direct shear testing, Figure 4.4 shows the sampling activities. Boreholes were drilled using wire line system (HQ diameter; 96.5 mm outside diameter; 63.5 mm sample diameter) as shown in Figure 4.5 for borehole 2. Boring logs and core boxes photos are included in the appendices. Figure 4.6 shows the borehole locations.

Figure 4.4. Slope face with sampling location

Figure 4.5. Boreholes drilling

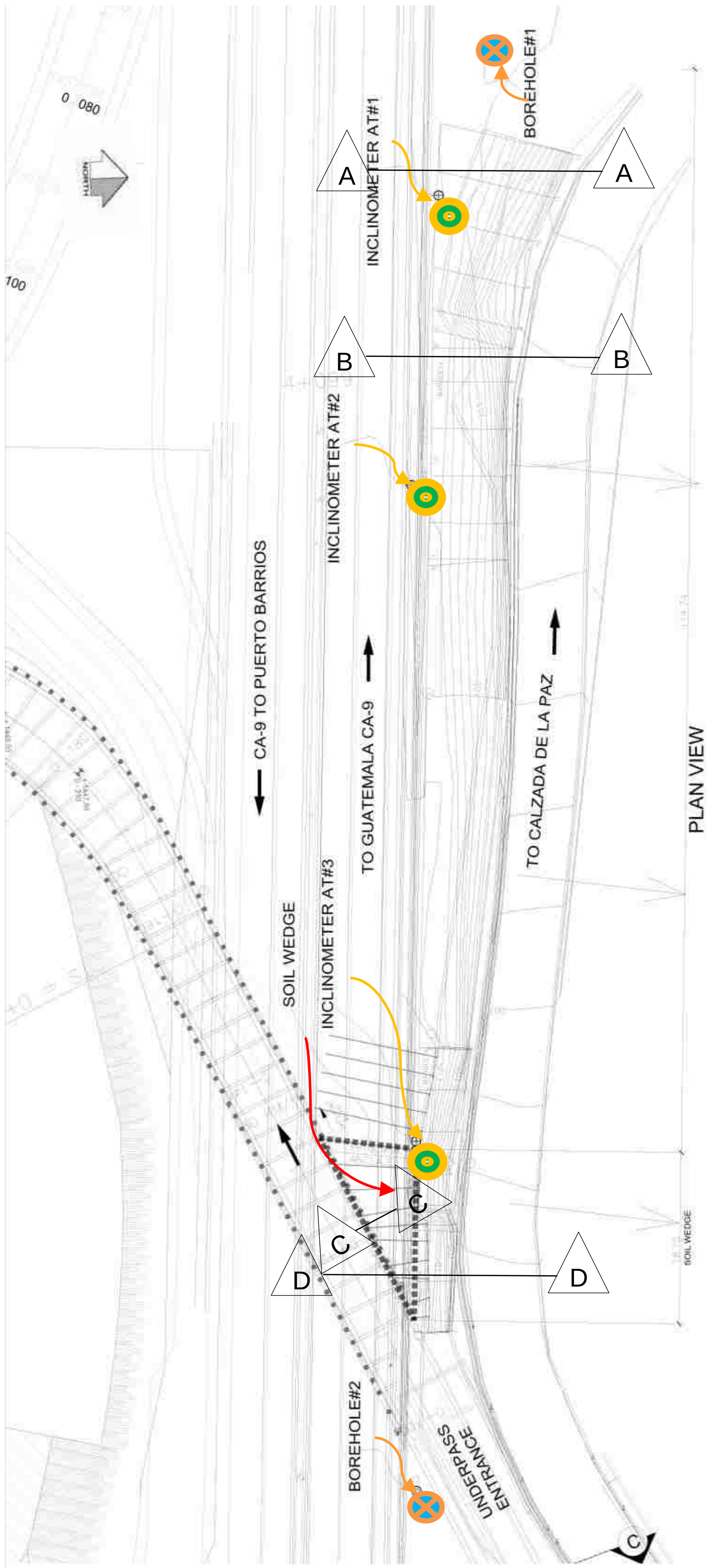


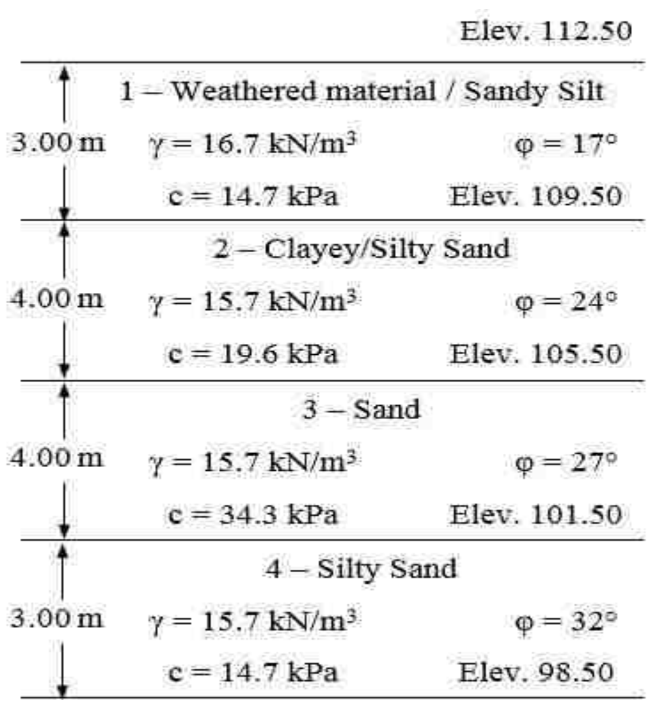
Figure 4.6. General plan view of the project, borehole and inclinometer location

4.1.2.4. Laboratory program. Laboratory testing program consisted of index tests such as moisture content, grain-size, Atterberg limits, and wet density. The strength parameters were determined using the direct shear test for block soil samples recovered. The summary of the laboratory testing program is presented in the Table 4.1.

Table 4.1. Laboratory test program results summary

Sample number	**Depth (m)**	**Soil Description**	**Natural moisture content (%)**	**Wet density (kN/m^3)**	**Cohesion (kPa)**	**Internal Friction Angle**	**Record No.**
M-1	3.90	Silty Sand	20.2	16.8	42.16	42°30’	13303
M-2	16.70	Silty Sand	33.8	14.0	23.96	34°30’	13364
M-3	23.00	Sandy Silt	25.6	15.4	30.66	28°30’	13365
M-4	24.80	Sandy Silt	39.5	14.2	71.87	43°00’	13366

4.1.2.5. Simplified soil profile. As result of site characterization program a simplified soil profile was determined for the entire slope.



Where: γ is soil’s unit weight, c is soil’s cohesion and φ is soil’s friction angle.

4.1.3. Stabilization Method Selection. Different stabilization methods and procedures were evaluated. Below is a bullet list of the analyzed options:

- Decrease slope inclination was discarded due to road proximity.
- Traditional gravity walls were discarded due to exit lane in slope toe.
- Mechanically stabilized earth walls were discarded due to road proximity.
- Shotcrete cover was an option for soil erosion protection, but not to increase slope stability.
- Soil nail wall provides erosion protection and increases slope stability.
- Tie back wall, post tensioned anchors wall, was also an option providing the same advantages that soil nailing but at a higher cost.

In the past, the slope had suffered local landslides but without signs of a global stability problem (i.e. cracks in the top of the slope, displacements or settlements). This supports the idea of erosion induced instability. Initial stability analysis was run in order to assess slope conditions and estimated the actual factor of safety to be marginal. This step also helps to refine the parameters that were selected in order to avoid be too conservative. If the factor of safety was too low (below 1.00) its means that the slope failed.

After analyzing the options including safety, technical viability, costs, construction time, local experience, and soil type, the soil nailing option was selected.

For the soil wedge between the slope and interior face of the under pass the following options were analyzed:

- Cut the soil wedge and uncover the structure. This was performed in the initial length but due to road proximity its application was limited.
- Substitute the soil wedge with a structure. This option just transfers the problem backward due to the underpass entrance angle.
- Gravity walls and/or Mechanical Stabilized Walls were discarded due to soil wedge height.
- Soil nail wall with inclined nails, stabilize the soil wedge. Soil wedge and underpass structure have different stiffness and probably will induce a failure plane or large deformations.

- Soil nail wall “sandwich” attached nails to the inner underpass soldier pile wall. This solution provides redundancy and reduces differential movements between soil nailing wall and underpass structure.
- Post tensioned anchors wall “sandwich” attaching anchors, strands, to the inner underpass soldier pile wall. This solution provides redundancy and reduces differential movements between anchored wall and underpass structure. Also provides compression within the soil wedge. Its cost is higher than soil nailing “sandwich”.

After analyzing the options including: safety, technical viability, costs, construction time and considering the experience in the area and with this soil type soil nailing was selected.

4.1.4. Solution Analysis and Design.

4.1.4.1. Soil nailing calculations. Soil nail walls improve soil mass stability with the inclusion of passive anchors or nails. These inclusions increase resisting forces, therefore factor or safety is increased as well.

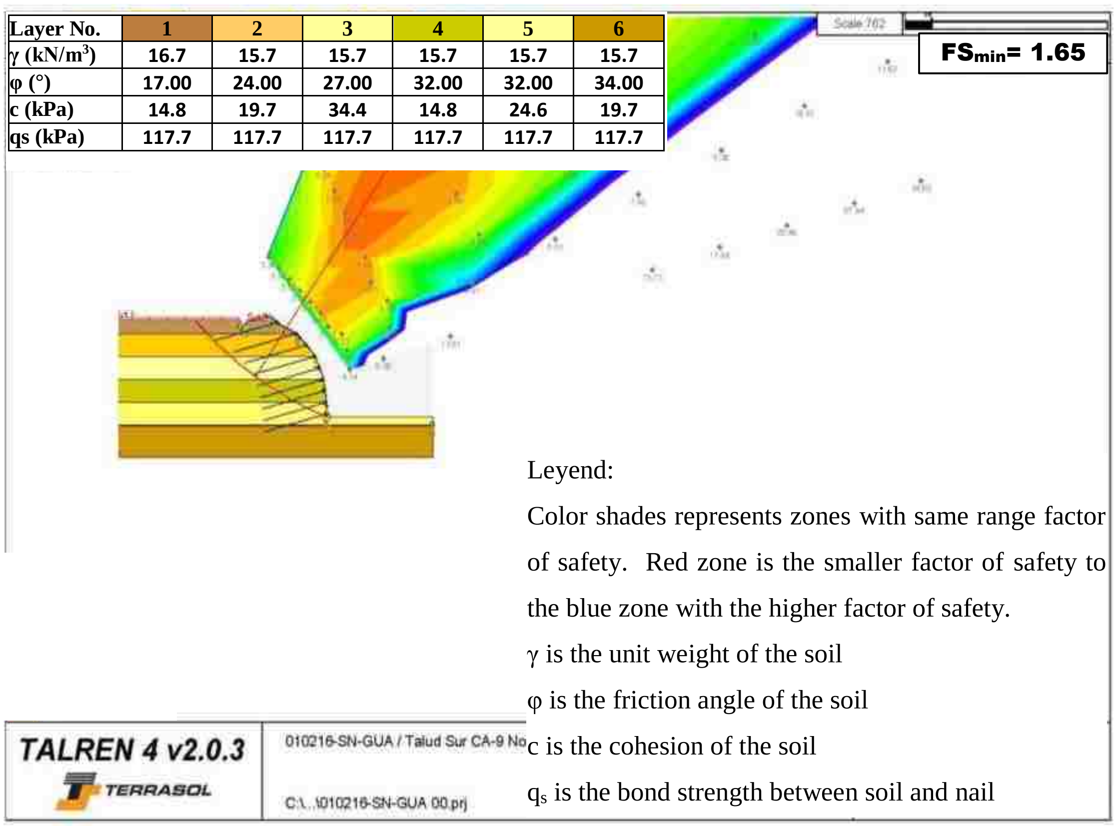
$$F.S. = \frac{\sum Resisiting\ Forces}{\sum Driving\ Forces} \qquad \text{(Eq. 4.1)}$$

The soil nailing facing distributes stresses from nails, given structural redundancy but also works as soil erosion protection. Global stability, external failure mode, was analyzed using simplified Bishop´s method. This limit equilibrium method satisfies forces equilibrium for each slice and overall moment equilibrium about the center of the circular trial surface. Since horizontal forces are not considered at each slice, the simplified Bishop method also assumes zero interslice shear forces. Calculations were performed using Talren 4 V2.0.3 (Terrasol, 2005) software by Terrasol selecting simplified Bishop’s method. Geometry and simplified soil profile were input, then a traffic load was added to the top of the slope and finally an automatic search for the critical slip surface was performed. Figures 4.7 and 4.8 shows results of the slope stability analysis for static and seismic load conditions, respectively. Figure 4.9 shows results for the underpass section.

Seismic considerations were evaluated using Monobe Okabe, MO, pseudo-static method for earth pressure. Maximum peak ground acceleration, PGA, used is 0.4g based on the recommendations presented by the Guatemalan Structural and Seismic Association, AGIES, 2009. A reduction coefficient of 0.5 was used according to Lazarte, et al. (2003).

Allowable Stress Design (ASD) method was selected. In this case a minimum Factor of Safety ($F.S._{min}$) equal to 1.50 for static loads, including traffic loads (critical loads in this case), and 1.10 for transitory loads as earthquake were used.

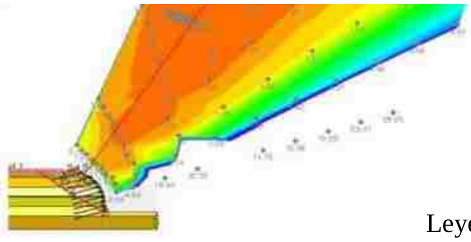
Internal failure modes calculations, including pull out resistance, facing and drainage, calculations were based on Lazarte, et al. (2003) guidelines for concrete and nails design.



Layer No.	1	2	3	4	5	6
γ (kN/m^3)	16.7	15.7	15.7	15.7	15.7	15.7
φ (°)	17.00	24.00	27.00	32.00	32.00	34.00
c (kPa)	14.8	19.7	34.4	14.8	24.6	19.7
qs (kPa)	117.7	117.7	117.7	117.7	117.7	117.7

Figure 4.7. Software output of slope stability analysis for static load condition

Layer No.	1	2	3	4	5	6
γ (kN/m^3)	16.7	15.7	15.7	15.7	15.7	15.7
φ (°)	17.00	24.00	27.00	32.00	32.00	34.00
c (kPa)	14.8	19.7	34.4	14.8	24.6	19.7
qs (kPa)	117.7	117.7	117.7	117.7	117.7	117.7



Leyend:

Color shades represents zones with same range factor of safety. Red zone is the smaller factor of safety to the blue zone with the higher factor of safety.

γ is the unit weight of the soil

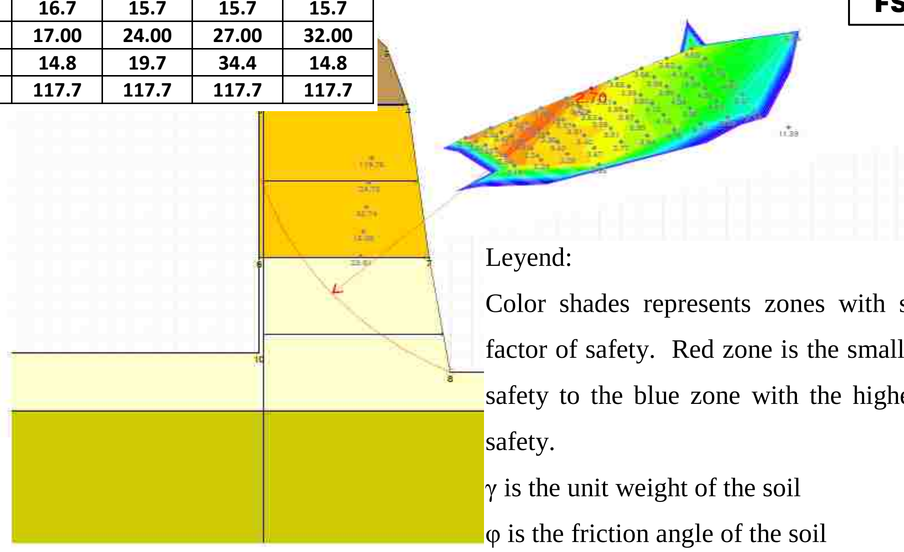
φ is the friction angle of the soil

c is the cohesion of the soil

q_s is the bond strength between soil and nail

Figure 4.8. Software output of slope stability analysis for seismic load condition

Layer No.	1	2	3	4
γ (kN/m^3)	16.7	15.7	15.7	15.7
φ (°)	17.00	24.00	27.00	32.00
c (kPa)	14.8	19.7	34.4	14.8
qs (kPa)	117.7	117.7	117.7	117.7



Leyend:

Color shades represents zones with same range factor of safety. Red zone is the smaller factor of safety to the blue zone with the higher factor of safety.

γ is the unit weight of the soil

φ is the friction angle of the soil

c is the cohesion of the soil

q_s is the bond strength between soil and nail

Figure 4.9. Software output of slope stability analysis for static load condition

The beam on non-linear Winkler´s foundation method was used to calculate deformation and therefore stresses in the soldier pile wall area interacting with the soil nail wall. The vertical beam models, the pile, and the horizontal springs connecting the beam and supporting soil model. The governing differential equation for the model can be expressed as:

$EI \frac{d^4 y}{dz^4} = -Dp$ (Takahashi, A., 2005) (Eq. 4.2)

And the p-y spring modeled by a hyperbolic function can be written as:

$p = C_i \frac{k_{hr}}{1+|^{y}/_{y_r}|} y$ (Takahashi, A., 2005) (Eq. 4.3)

Where EI = flexural rigidity of pile, y = relative displacement between pile (u) and soil in free field (u_g), z = depth form the pile head, D = width of pile (or width of footing, B), p = horizontal subgrade reaction, C_i = scaling factor for the p-y curve at i-th layer, k_{hr} = coefficient of initial subgrade reaction parameter, and y_r = reference relative displacement. As $p|_{y=\infty} = k_{hr}\ y_r$ when $C_i = 1$, y_r may be defined as follow using the Brom's (1964) ultimate pile resistance. (Broms, B., 1964)

$y_r = 3K_p\sigma'_v/k_{hr}$ (Eq. 4.4)

Where K_p = coefficient of passive earth pressure, and σ'_v = effective overburden pressure.

Deformation analysis of the soldier pile wall area was performed using RIDO (Fages, R., 2010) software by RFL. This software uses the beam on non-linear Winkler´s foundation method Figure 4.10 shows software output of deformation analysis for static load condition. Reaction modulus was calculated based on Chadeisson's (1961) abacus. Software input and results are included in the appendices.

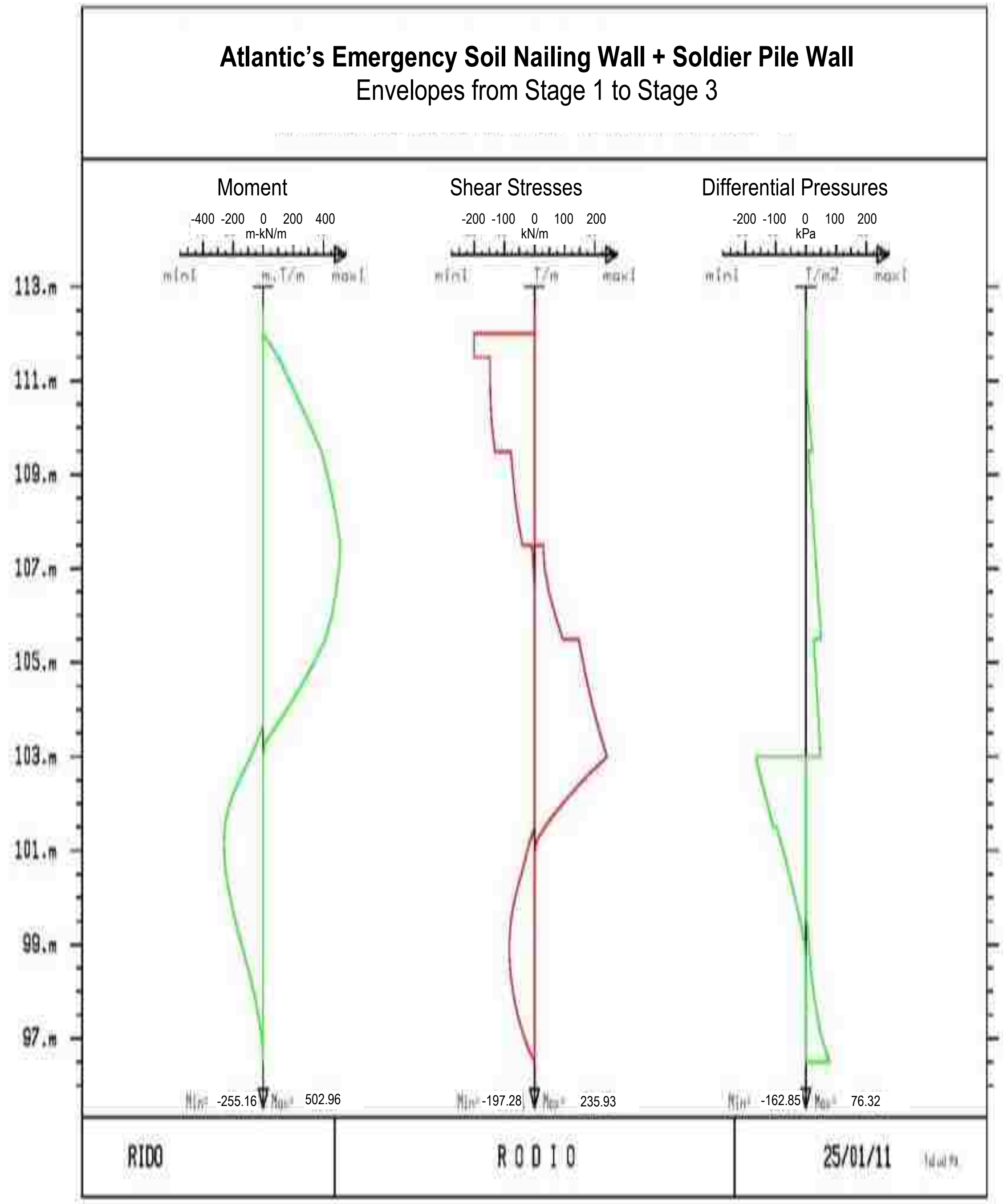


Figure 4.10. Software output of deformation analysis for static load condition

The Table 4.2 summarize the results of stability and deformation calculations.

Table 4.2. Summary of calculation results

Location / Section on Figure 4.4.	Height (m)	Factor of Safety in Static Conditions	Factor of Safety in Dynamic Conditions	Estimated Maximum Deformation (mm)	Maximum Flexural Moment (kN-m/m)
Slope / A-A	15.50	1.87	1.20		
Slope / B-B	18.70	1.65	1.11		
Underpass soldier pile wall / C-C	9.50			11.04	502.96
Underpass slope side / D-D	9.50	2.70	2.05		

4.1.4.2. Soil nail wall facing design (permanent). Facing design was based on the Geotechnical Engineering Circular No. 7, Soil Nailing Walls, FHWA, (Lazarte, et al, 2003), calculation are show below:

Materials

Wall

Rebar:

F_y:	420	MPa	60	ksi

Where F_y is steel rebar yield strength.

Welded Wire Mesh

F_y:	455	MPa		
Schedule:	3 /3			
Reinforcement area per square meter:			200.62	mm^2/m^2

Concrete:

f'_c:	21	MPa	3,000.00	psi

Where f'_c is specified compressive strength of the concrete.

Nail characteristics

Horizontal spacing (SH):	1.75	m	OK
Vertical spacing (SV):	1.75	m	OK

Influence area:		3.06	m^2	OK
Bar No.:	8			
Bar area:	0.79	in^2	506.71	mm^2
T_{max} Bar:	12.83	Ton	125.77	kN

Where T_{max} is the maximum tensile resistance of the bar.

(Based on Chapter D, Steel Construction Manual, American Institute of Steel Construction, 13th, 2005)

V_{max} Bar:	6.43	Ton		
T_{max} Analysis:	12.83	Ton	125.77	kN

Facing nail head tension force (T_o):

T_o = Facing bar tension.

$T_o = T_{max\text{-}s}\,[0.6 + 0.2\,(S_V[m]-1)]$ (Eq. 4.5)

T_o =	9.62	Ton	94.33	kN

Thickness (h)

h:	0.15	m	150.00	mm

Flexural resistance:

Minimum reinforcement

$$\rho_{min[\%]=20*\sqrt{f'_c[MPa]/F_y[MPa]}}$$ (Eq. 4.6)

ρ_{min} = 0.20%

Maximum reinforcement

$$\rho_{max}[\%] = 0.5f'_c[MPa] * 600 \Big/ \left(F_y[MPa] * \left(600 + F_y[MPa]\right)\right)$$ (Eq. 4.7)

ρ_{max} = 1.31%

Reinforcement area

Reinforcement area square meter:

Waler bars (both directions)

Bar length: 1.20 m

Bar No.:	5		
Bar Area:	0.31	in^2	197.93 mm^2
Bar quantity:	2		
a_n:	226.21	mm^2/m^2	
a_m:	200.62	mm^2/m^2	
a_{sn}:	426.83	mm^2/m^2	

ρ_n =[a_n/(bh/2)]*100 (Eq. 4.8)

ρ_n = 0.57% OK

ρ_m =[a_m/(bh/2)]*100 (Eq. 4.9)

ρ_m = 0.27% OK

C_{FV} Factor: 1.0

Facing flexural resistance calculation (R_{FF})

R_{FF} [kN] = minimum of :

$1.6xC_{FX}(a_{vn}+a_{vm})[mm^2/m]xh[m]$ (Eq. 4.10)

$1.6xC_{FX}(a_{vn}+a_{vm})[mm^2/m]xh[m]$

R_{FF} = 150.59 kN

FS_{FF} = 1.60 OK

Punching shear resistance calculation

$R_{FP} = C_P V_F$ (Eq. 4.11)

C_P = Correction factor due to ground support.

C_P = 1.15. If ground support is considered.

C_P = 1.00. If no ground support is considered (usual).

C_P = 1.00

V_F [kN] = $330\sqrt{f'c}$[Mpa]xD'$_c$[m]h$_c$[m]

D'c = Effective diameter of conical failure surface at the center of the section.

D'c = L_{BP} + h (Eq. 4.12)

L_{BP} = Bearing plate side.

L_{BP} = $\boxed{0.20}$ m

D'c = 0.35

h_C = Effective depth of the conical surface.

$h_C = h$ (Eq. 4.13)

V_F = 249.42 kN

$R_{FP} = C_P V_F$ = 249.42 kN

$FS_{HT} = R_{FP}/T_o$ (Eq. 4.14)

FS_{HT} = 2.64 OK

4.1.4.3. Final design. The final design consists of 12.00 m length nails of No.8 (25.40 mm (1in) diameter) steel rebar Grade 60, Fy = 420 MPa, spaced at 2.00 m in both directions. Facing have a thickness of 0.16 m, reinforced with a welded wire mesh schedule 3/3, Grade 70, Fy = 500 MPa, with 2 No. 5 (15.88 mm (5/8") diameter) Grade 60, 1.20 m length waler bars in each direction. Two rows of 6.00 m length, 75 mm (3") horizontal drains were installed alternated with weep holes Figures 4.11 and 4.12 shows a 3D model of the final solution. The wall is supported over a 0.50m width strip footing.

Preventing any possible movement a joint between shotcrete facing over interior soldier pile wall and the soil nailing area was left in place, in the same way a construction joint between the exterior soil nail walls attached to the soldier pile wall was left in place.

4.1.5. Construction. Soil nail walls were successfully constructed on time, on budget, and without safety issues (landslides or personal accidents). So, from the point of a strength limit state the structure performed successfully.

4.1.5.1. Equipment and methodology used.

4.1.5.1.1. Excavation and slope grading. Slope grading was performed by hand labor, for this purpose a safety line was installed at the top of the slope. All the personnel used safety equipment for suspended lines. Komatsu WB93S backhoe was used to load material from grading.

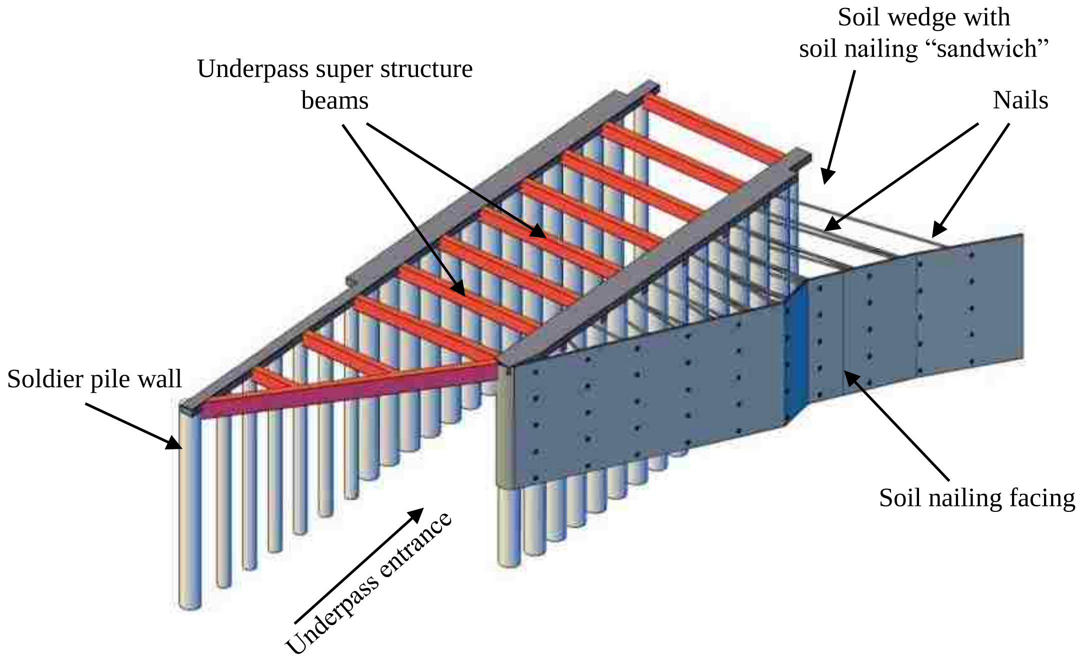


Figure 4.11. 3D model of underpass and soil nail wall external view

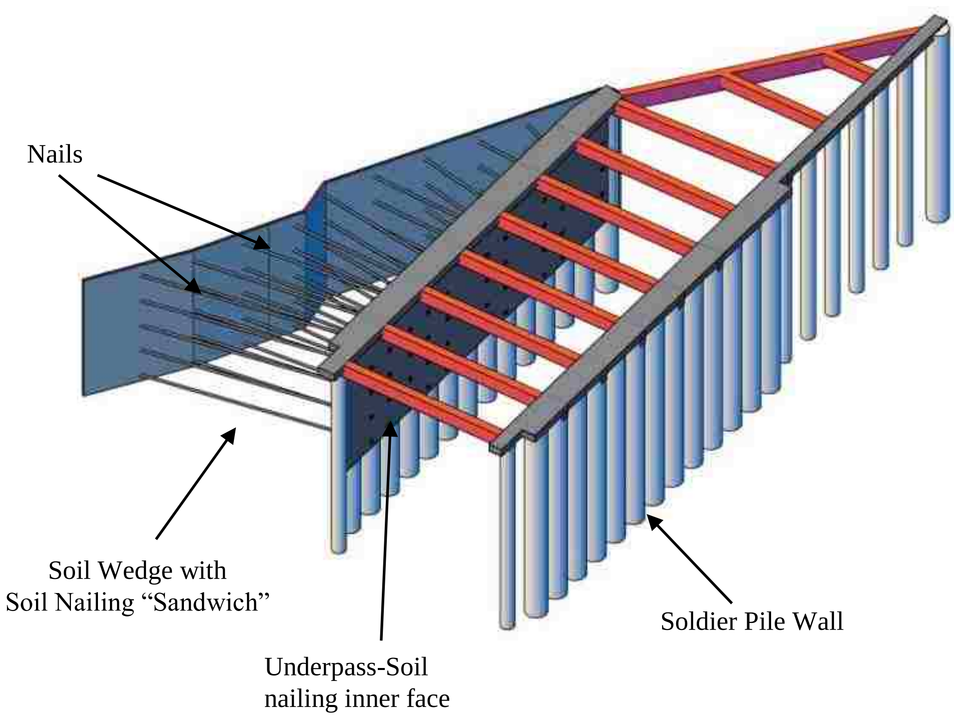


Figure 4.12. 3D model of underpass and soil nail wall reverse view

4.1.5.1.2. **Drilling soil nailing holes.** Due to the slope height, 16.70 m, different types of drilling equipment were required. For heights upon 2.90 m a Klemm 806 track mounted drill rig was used. For heights above 2.90 m an Atlas Copco DHR-45 pneumatic drill rig mounted on a Manitou MT 1435 telescopic elevator was used.

4.1.5.1.3. **Grouting.** Grouting was performed thru a sacrificial pipe attached to the reinforcement bar. A Cosma pneumatic single action piston grout pump was used.

4.1.5.1.4. **Shotcreting.** A Shotcreting basket was installed in the telescopic elevator from there concrete was sprayed. A BSA-100 Putzmeister concrete pump and an Ingersoll Rand 375 cfm / 100psi were used.

4.1.5.1.5. **Instrumentation.** Inclinometers were installed at two different locations of the slope, one with 23.0 m length and other with 11.0 m length. Pipe of 70 mm diameter was used.

4.1.5.2. Construction Sequence. Construction sequences is presented in the Figure 4.13.

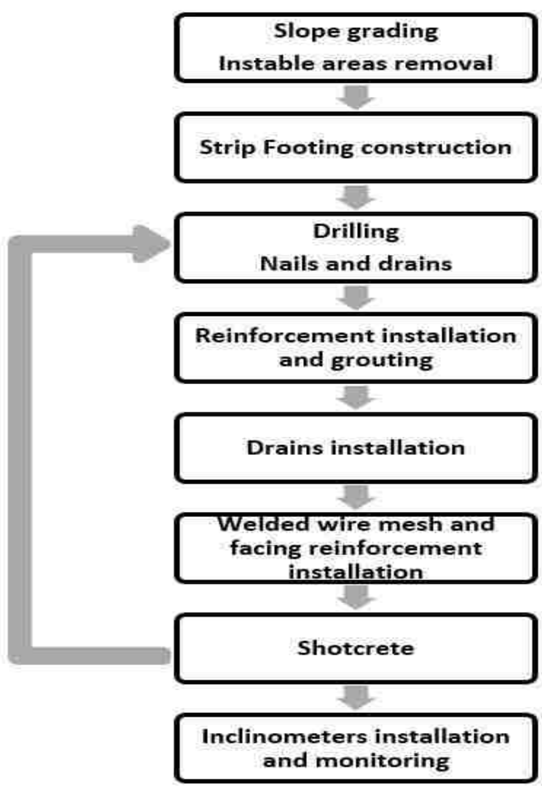


Figure 4.13. Construction flow chart

4.1.6. Instrumentation and Performance.

4.1.6.1. Instrumentation. Inclinometers were installed at three different locations in order to monitoring slope and soil nailing walls movements, Figure 4.14 shows an inclinometer installation. The Table 4.3 shows inclinometers characteristics. Readings were performed with a digital inclinometer and Figure 4.15 shows technicians conducting the inclinometer readings. Inclinometer system characteristics are shown in the Table 4.4. Digipro software was used to analyze the information. The monitoring program includes the reading of the inclinometer during one year period, in order to have a more complete data of the performance of this wall, Table 4.5 shows the monitoring schedule. Additional readings were conducted in 2015 to examine the long-term performance of earth structures.

Figure 4.14. Inclinometer installation

Table 4.3. Inclinometer system characteristics

Manufacturer:	Durham Geo Slope Indicator, DGSI
Probe	
Model:	Digitilt Inclinometer
Serial Number:	50302510
Sensor Type:	Analog force-balanced servo-accelerometers x 2
Wheel Base:	500 mm
Calibrated Range:	±30° ±30°
System Resolution:	0.01 mm
System Accuracy:	±6 mm / 25m
Precision:	±0.01%
Operative Temperature:	-20 to +50 °C
Material:	Stainless Steel
Readout / Datalogger box	
Model:	Digitilt Datamate
Serial Number:	50310900
Digital display	
Control Cable	
Serial Number:	50601050
Length:	50.00 m with marks each 0.50 m

Table 4.4. Inclinometers characteristics

Inclinometer No.1	Depth (m)	Pipe Diameter (mm)	Completion Date
1	22.00	70	01/12/2011
02A	22.00	70	03/20/2011
03A	21.00	70	03/16/2011

Table 4.5. Monitoring schedule

Reading No.	Inclinometer 1 Reading Date	Inclinometer 02A Reading Date	Inclinometer 3 Reading Date
1	01/20/2011	04/08/2011	04/08/2011
2	01/28/2011	04/11/2011	04/11/2011
3	02/03/2011	04/19/2011	04/19/2011
4	02/12/2011	05/02/2011	05/02/2011
5	02/26/2011	05/09/2011	05/09/2011
6	03/11/2011	06/10/2011	06/10/2011

Table 4.5 Monitoring schedule (cont.)

7	03/21/2011	08/23/2011	08/23/2011
8	04/08/2011	12/14/2011 b)	12/14/2011 b)
9	04/08/2011a)	05/14/2015	
10	04/11/2011	08/14/2015	
11	04/19/2011		
12	05/02/2011		
13	06/10/2011		
14	08/23/2011		
15	08/23/2011a)		
16	12/14/2011b)		
17	05/14/2015		
18	08/14/2015		

a) Verification reading, b) End of initial monitoring

Figure 4.15. Inclinometer reading

4.1.6.2. Performance. In the short-term, after one year of service, performance of the soil nail wall was very good showing almost no movement, at inclinometer #1. Figures 4.16 and 4.17 show the complete measurements history. The slope the inclinometer pipe was vandalized presenting a damage and pipe obstruction. The inclinometer cap was replaced and the pipe was cleaned. Long-term performance measurements on inclinometer #2A shows some peaks but a constant behavior that matches with no movement of soil mass and could represent the damage due to vandalism.

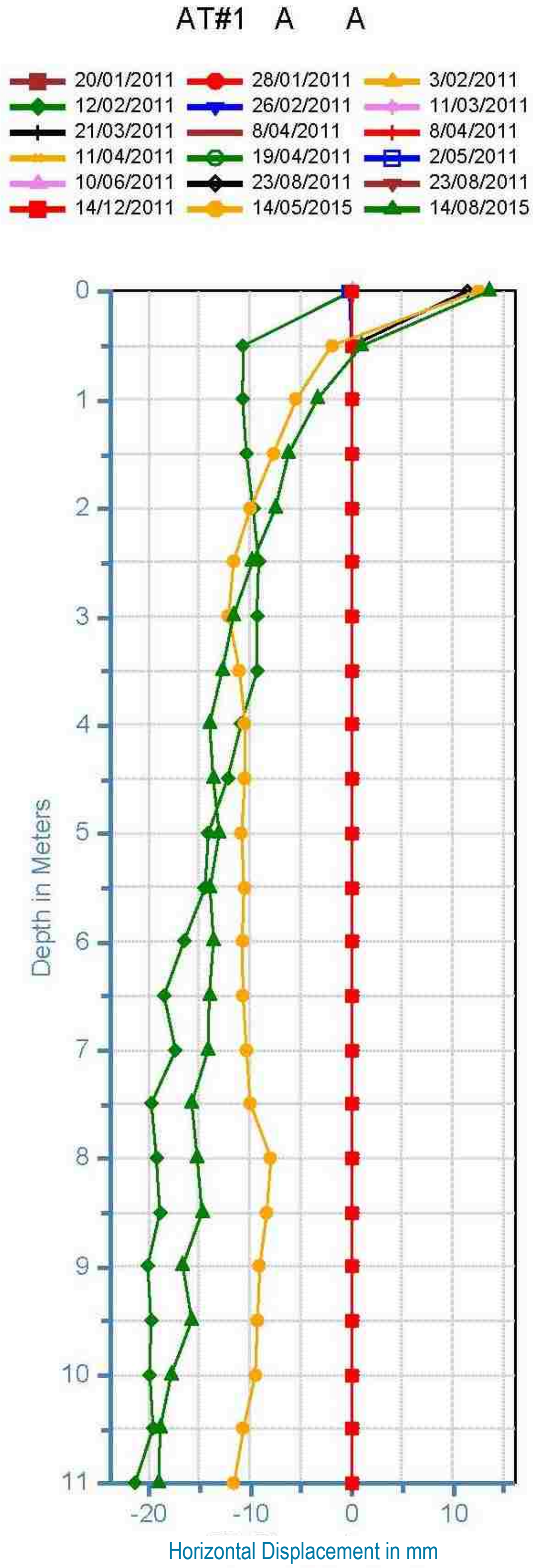
The inclinometer located near the underpass structure presented a short-term performance with very small displacements. Long-term monitoring shows an average maximum displacement of 20 mm that coincides with the wall base. Despite the stiffness difference between the soldier pile wall and the soil nailing wall, no major differential deformations, cracks or fissures were observed in the soil nail walls.

After visual inspection, the wall presents no apparent signs of movement, cracks, fissures or displacements. The real test for this earth structure system will be at the next extreme event, such as earthquake or storm related event. Thus, monitoring (inclinometers and general inspection) after any of these extreme events will be very important.

4.1.7. Lessons Learned. The objective of a case study is to present a detailed description of a constructed project and also share experience and knowledge. Even the most successful projects have lessons to be learned, throughout a life cycle of a project different lessons can be learned and opportunities for improvement can be discovered.

Identifying and documenting lessons learned provides a mechanism that communicate acquired knowledge more effectively and ensure that beneficial information is factored into planning, work processes, and activities for other similar projects.

Analyze lessons learned provides an opportunity to discuss successes during the project, unintended outcomes, and recommendations for similar future projects. It also allows the discussion of things that might have been done differently, the root causes of problems that occurred, and ways to avoid those problems.



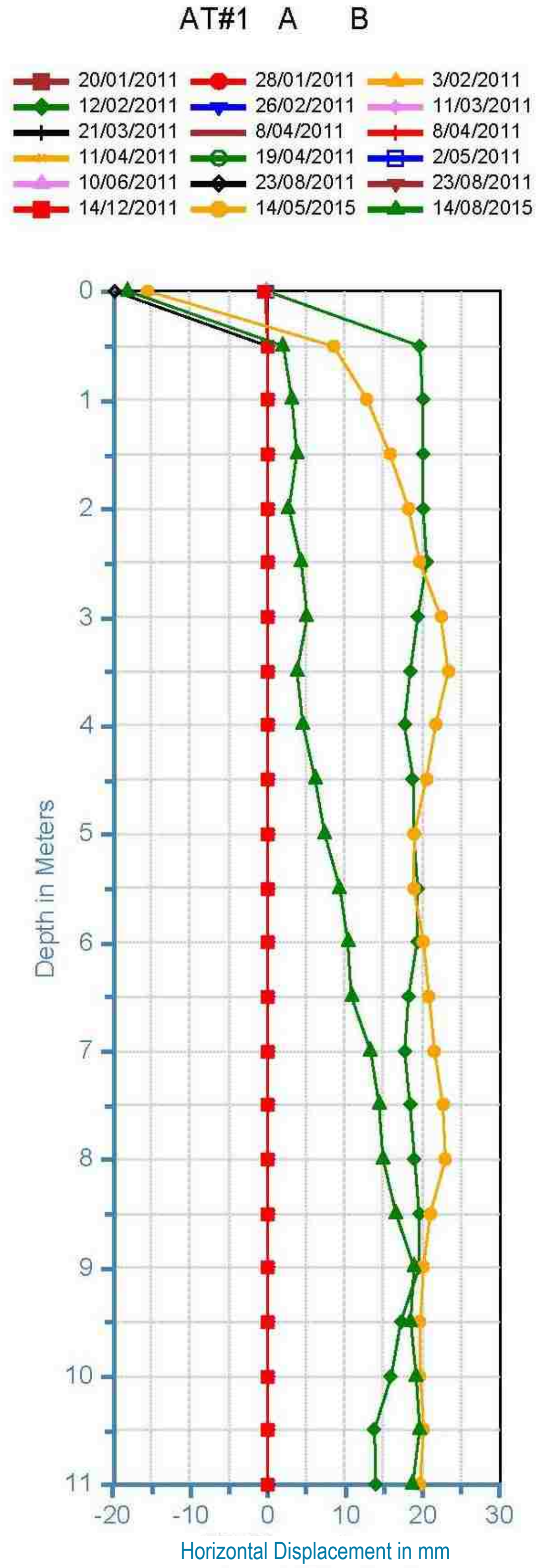
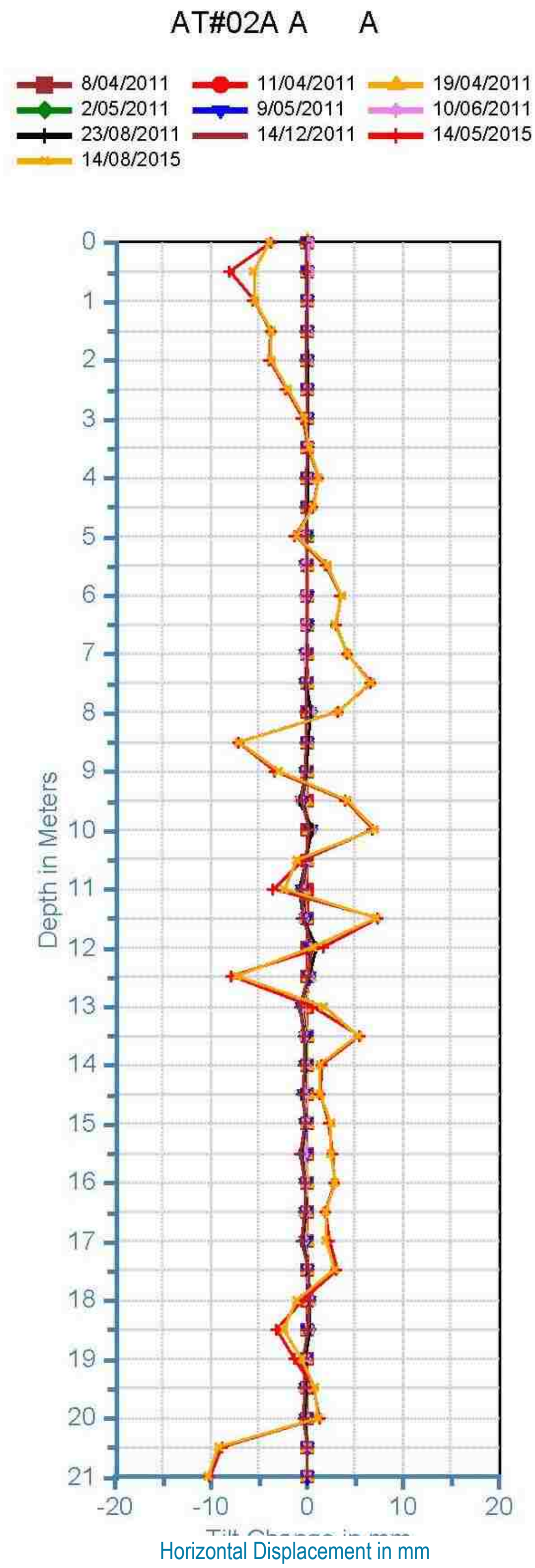


Figure 4.16. Inclinometer #1 displacements vs. depth



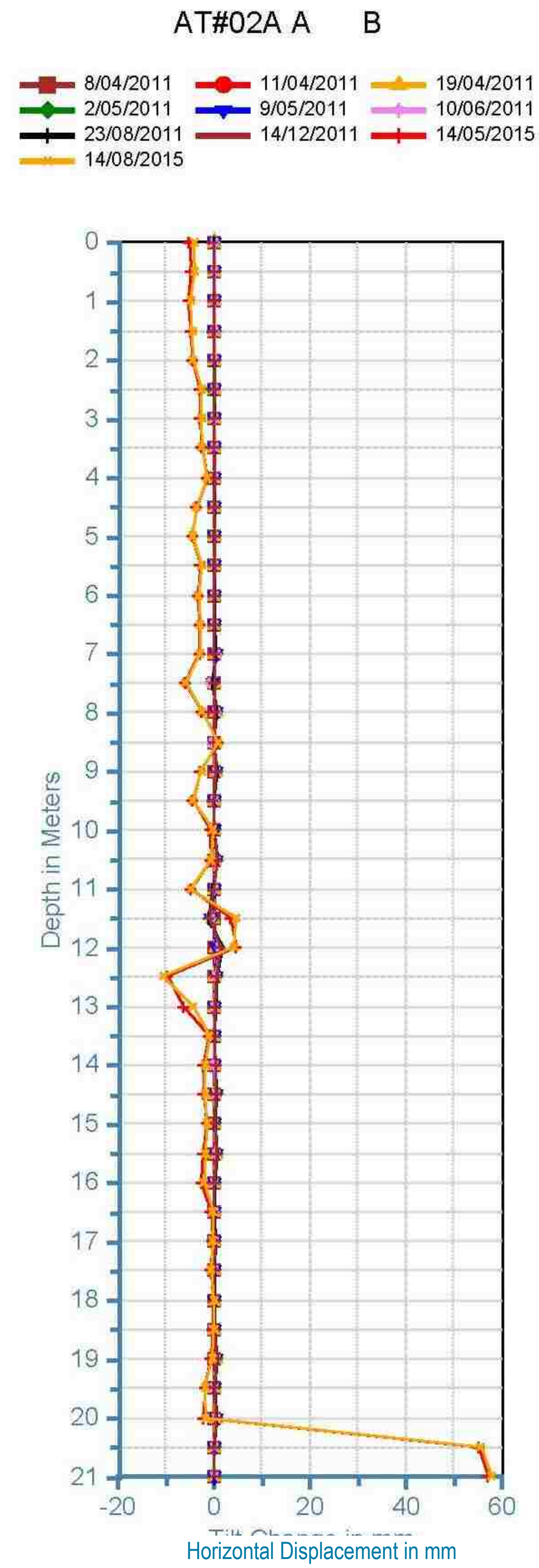


Figure 4.17. Inclinometer #02A displacements vs. depth

The major benefit of compile the lessons learned is retain and document both successful, best practices, and unsuccessful project activities for future reference. This allows new projects to repeat successful activities and to avoid those that were not successful.

The lessons learned are presented asking if this was the right solution, identifying if were there improvements that could have been made and closing with what were the success factors. This represent a walk through the most important aspects of the case study.

4.1.7.1. Was this the right solution? The soil nail wall increased the slope's factor of safety as well as protected the slope surface form erosion. It was cost effective in comparison with the anchored wall solution. To date the inclinometer shows no significant movements and whole area shows no signs of instability. The soil wedge area solution, soil nailing "sandwich", has performed in a satisfactory manner. Despite the stiffness difference between soil mass and underpass structure there are no signs of critical movements or major cracks or fissures. Analyzing the data and the overall performance, the solutions shows that are reliable and have a good long term performance.

4.1.7.2. Were there improvements that could have been made? Analyzing the project in retrospective, several improvements can be done. Improvement suggestions are presented below for each stage of the project.

4.1.7.2.1. <u>Geotechnical investigation.</u> For geotechnical survey, laboratory testing should include triaxial tests, Consolidated-Undrained. This could improve accuracy and reliability of soil strength parameters leading to reduce conservative assumptions in the simplified soil profile. It is important to mention that this suggestion could be difficult to implement due to the fact that in Guatemala only one laboratory has the equipment to perform triaxial tests. Also, the time to perform each test was not compatible with the emergency situation of the project. The tests could be performed as a complement to refine the final design.

A seismic geophysical survey could be included in the field investigation, providing an idea of the whole soil mass variability. This also could identify unknown or unperceivable failures. Geophysical survey is a fast test that provides results at an affordable cost that is very suitable for emergency situations.

4.1.7.2.2. Design. Finite element model could be used in order to predict deformations with greater precision. Also finite element analysis provides stress-strain predictions, so reinforcement optimization could be performed. A 3-D model also could be used particularly for the wedge area, this could lead to determine possible areas of stress concentration that requires specific attention.

The solution adopted is permanent, so every part of the solution has to be weather resistant. The bearing plate detail initially did not include any corrosion protection, finally an in situ concrete cap was used. The bearing plate cap detail could be included from the beginning in the project's drawings and planning.

An important improvement to optimize the analyses, would be model the final performance of the whole earth structure system. Initially the road had a lateral drainage ditch that was demolished during strip footing construction. This ditch was to be rebuilt over the strip footing. Late during construction, the drainage ditch was joined with the strip footing providing appropriate water relief at the top of the footing. This solution presented a cost reduction. The drainage ditch detail could be included from the beginning within project's drawings and planning.

In order to provide a baseline for the construction team and the ease construction process a suggested full construction sequence could be included in the project's drawings. This is a good practice because the design consider a particular construction sequence that became in a construction constriction due to its direct relation with the allowed movements. A change in the assumed construction sequence could result in undesired movements or even in a failure.

Additionally, some landscaping could be included as part of the project. This can include add pigment to the concrete, install an additional facing material as stone or brick, and some vegetation for surface and some vegetation for surface protection and erosion control.

4.1.7.2.3. Construction. One of the most important improvements to be made in future projects is to change the construction sequence, construction sequence was bottom-up instead top-down. A top-down process provides the advantage of being covered against eventual local landslides, which is a great safety feature for the construction team.

Also top-down construction eliminates the necessity to clean the constructed area from debries or any dumped material of works performed in the upper parts.

As part of quality control assurance a load testing and coring program should be included in the project specifications. Load testing allows verification of the pullout resistance estimated during design, and also validates the correct performance of the constructed nails. A coring program verifies the thickness of the concrete facing and its resistance.

4.1.7.2.4. Instrumentation. Inclinometers could be installed before the start of construction stage in order to have a complete record of the soil mass displacement. This requires coordination between the design and construction teams in order to avoid damaging the inclinometer casing or pipe.

Strain gauges and load cells could be included to double check or correlate with the inclinometers. An increase in the load of load cells could indicate or validate the existence of a failure or movement. Also it could provide information for the improvement of the design of future projects.

Survey monuments referred to an external benchmarks could provide additional information about the system (underpass structure-soil nailing wall) performance. Also, the inclinometer could be installed within a soldier pile wall in order to asses deformation in the wall and therefore estimate the stresses in the piles.

One of the most important improvements for instrumentation is provide a protected and secured manhole for the inclinometer or any other instrumentation. Any public or private infrastructure could be subject of vandalism, so additional protection have to be considered.

4.1.7.3. Success factors of the project. As improvements are addressed also success factors must be highlighted. The analyses of the different options with an open mind and from the perspective of advantages, disadvantages and its estimated cost converges is a validated solution. The best solution is the one that provides satisfactory safety at the lowest cost within the reasonable time.

Consider the interaction between the different elements of the system, as soil wedge and underpass structure, is critical as is directly related to its performance. An example of this is the difference of stiffness that inevitable conduce to fissure or undesired differential movements.

The use of a telescopic elevator was a time saver compared to the use of scaffolding. Including instrumentation was an important part of the project to provide information about solution performance, as well as to improve future designs.

The use of a design-build contract is not relevant from the technical point of view but it eases the design process and boost the pursuit for improvement in each stage of the project.

4.2. PERFORMANCE OF GROUTING INTENSITY METHOD, GIN, FOR A CUTOFF CURTAIN FOR SANTA TERESA DAM IN GUATEMALA

4.2.1. Introduction. Seepage control can be one of the most challenging conditions during the design and construction of an embankment. Piping and erosion are the most common problems related with seepage but also pore pressure increases can result in an undesirable performance. The use of grouting for seepage control and ground consolidation can be a very effective solution, but in many cases the implementation poses a challenge. The use of grouting has almost been considered a "black art" of geotechnical engineering, due the relative unfamiliarity of the relationships between key performance parameters such as the mixes, grouting rates, pressures, monitoring and verification process, and its consequential cost uncertainty. The Grouting Intensity Number, GIN, method developed in the early 1990s by Giovanni Lombardi and Don U. Deere presents a solution for controlling grouting and achieving a satisfactory performance level.

Despite the popularization of the method through the 1990s, it was not until 2010 that the method was first used in Guatemala: the grouting program for the Santa Teresa dam was specified using the GIN method. The Santa Teresa dam is made of concrete and is part of a relatively small hydro electrical project located in northern part of Guatemala, specifically in Alta Verapaz on the Polochic River. The project consists of a 29.4 m high, 83 m long dam, an intake, a 600 m tunnel, a 4 km conduction channel, and a 16 MW powerhouse Figures 4.18 and 4.19 shows project location. It's the second highest dam in Guatemala, Figure 4.20 shows an actual image of the project. The dam's foundation support is composed of metamorphic material and limestone. Due to project´s nature, soil type and water height, seepage is a critical issue.

The grouting work performed consisted of the installation of a cut-off curtain and vertical drains installation. The majority of the program was completed from inspection galleries located in the base of the dam, but also some surface drilling was performed. The aim of this case study is present the complete process and sequence for the design and construction of a cut off grouting curtain using de GIN method in Santa Teresa dam. Also present the lessons learned during this process.

Figure 4.18. Project location (Googlearth, 2016)

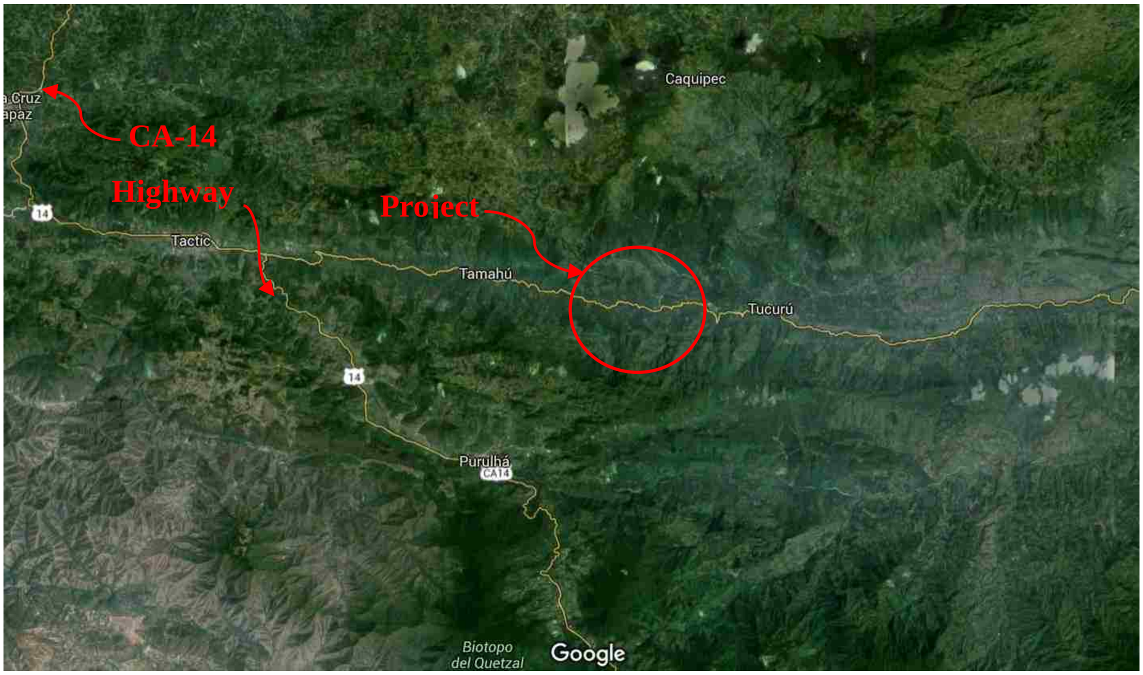
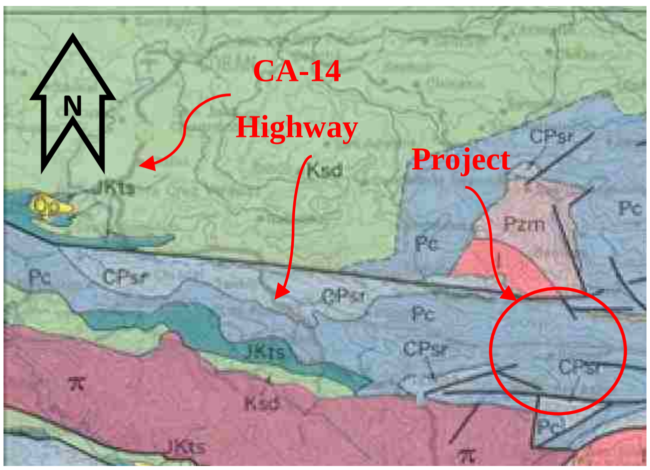


Figure 4.19. Project location (Googlemaps, 2016)

Figure 4.20. Santa Teresa Dam (Corporacionmultinveriones, 2016)

4.2.2. Site Characterization. An extensive geotechnical, geological and geophysical site investigations and characterizations were performed. The investigation program for the dam consisted of: boreholes, rock cores, test pits, standard penetration tests (SPT), geophysical refraction lines, cross hole tests, geological mapping, rock mass ratio (RMR) classification, Lefranc and Lugeon water tests, point load tests, and unconfined compression tests. In the Figures 4.21 is shown the general geological setting of the project area, meanwhile Figure 4.22 shows the dam area prior construction.



Where:
CPsr: Carboniferous Permian
I: Undivided Plutonic Rocks
JKts: Jurassic Cretaceous
Ksd: Cretaceous. Neocomian-Campanian carbonates.
Pc: Permian
π:Ultrabasic rocks of unknown age
Qa: Quaternary.

Figure 4.21. Area geological map (Instituto Geografico Nacional, 1970)

Figure 4.22. Dam area prior construction works

4.2.2.1. Boreholes. Several boreholes were performed in order to assess different parts of the project. Boreholes were drilled using rotary core drilling system in HQ diameter, hole (outside) diameter: 96 mm; core (inside) diameter: 63.5 mm. The total of 12 boreholes were drilled and were located as follows:

Three on the stroke of regional Polochic fault (F boreholes)

Three on the axis of the dam, on both banks of the river and optional dam axis site downstream in the left side (D boreholes).

Two, in the mouth of the tunnel entrance and (T boreholes)

A short drilling at the mouth of the tunnel (T-2 A borehole)

Three, in the alternative powerhouse (PH boreholes)

One on the middle part of the pressure pipe (PST-1 well boreholes)

A summary of the boreholes performed in the dam area is presented in the Table 4.6, location of the different exploration is shown in the Figure 4.23. The Figure 4.24 shows core boxes for borehole D-3 for a depth between 8.30 m and 18.00 m.

Table 4.6. Summary of boreholes performed in the dam axis

Borehole	Completion Date	Depth (m)	Inclination	Lugeon Tests	Lefranc Tests	% of Recovery
D – 1	03/14/03	50.00	30° NE35	14	0	84.3%
D – 2	04/08/03	50.00	30° NE88	5	0	99.0%
D – 3	06/23/04	30.25	30° NE35	6	2	97.6%

4.2.2.2. Geophysical investigation. The geophysical exploration program was conducted using the seismic refraction method. Several geophysical lines were programmed in various sites of the project, in order to investigate the general stratigraphy and structure of the ground. A seismic equipment manufactured by Geometrics, model Smartseis, 12 channels, battery operated, using as a source of energy or "trigger" a sledge hammer of 12 pounds that hits on a metal plate was used. Field information obtained is recorded in digital format and stored on the hard disk of the unit.

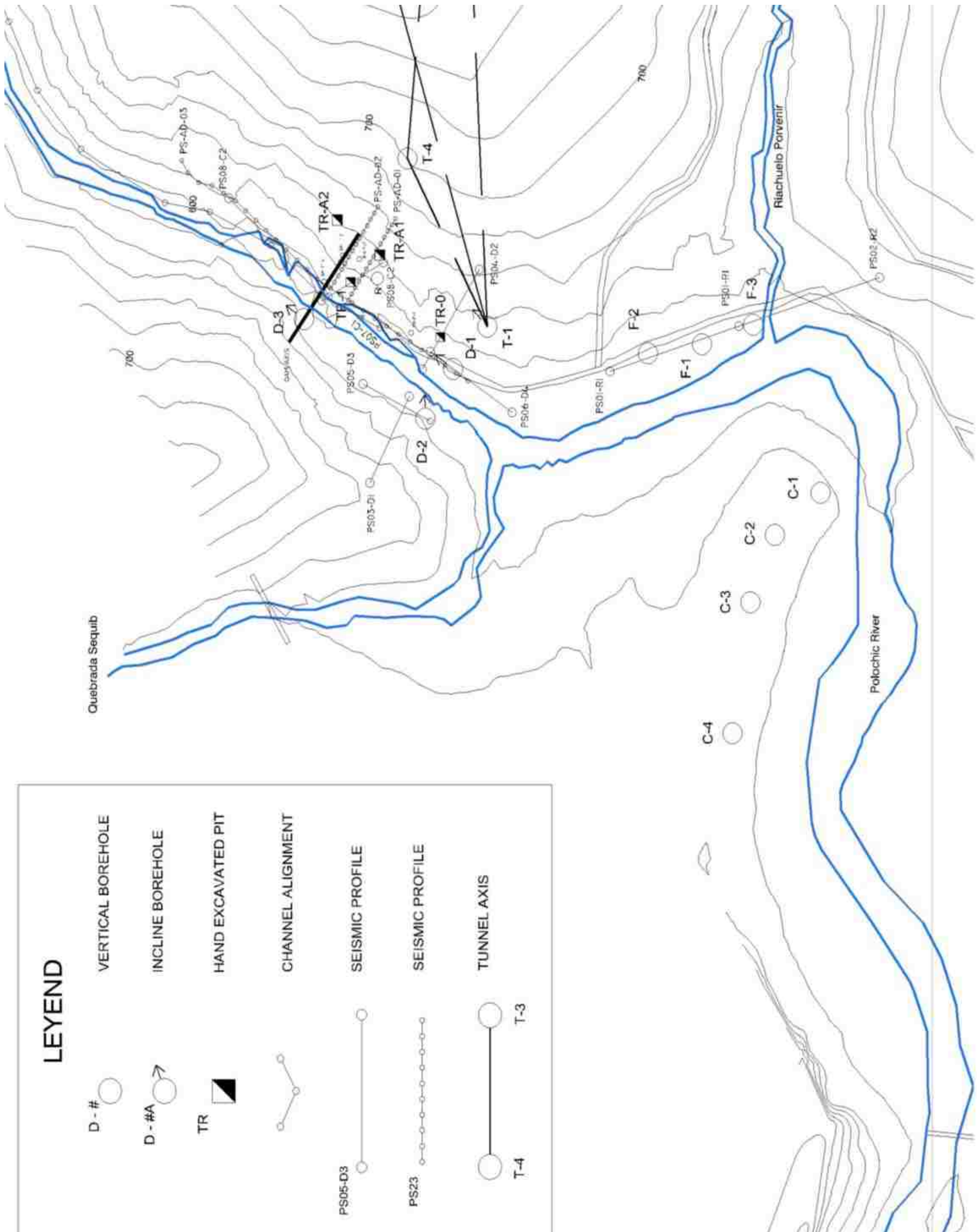


Figure 4.23. Explorations location (Alvarado, 2005)

Figure 4.24. Core boxes (Alvarado, 2005)

The interpretation was performed using the SIP (Seismic Interpretation Program). For the interpretation dimension topographic information and coordinate the starting and ending points of each line of research with the distances and elevations of each measuring point is needed to make corrections in the program interpretation of results. The exploration program, included the performance of 34 seismic profiles for the whole project.

Particularly for the dam axis area a long profile, consisting of two seismic lines that connect both sides the river were performed, profiles PS03-D-1 and D-2-PS04. A long profile, perpendicular to the previous, passing over the survey D-1 on the right abutment of the dam were also performed, profiles PS06-D-4, PS07-C-1. A standard profile, on the left abutment perpendicular to the dam axis profile was performed, PS05-D-3. A complementary exploration was performed including three seismic profiles identified as follows:

A transverse profile, PS Ad 01, 55.00 m long slope on the right side of the Polochic River, this profile is presented in the Figure 4.25.

A transverse profile, PS Ad 02, 55.00 m long, almost parallel to the PS 01. These two profiles crossing the dam axis profile.

A longitudinal profile, PS Ad 03, 110.00 m long located along the river.

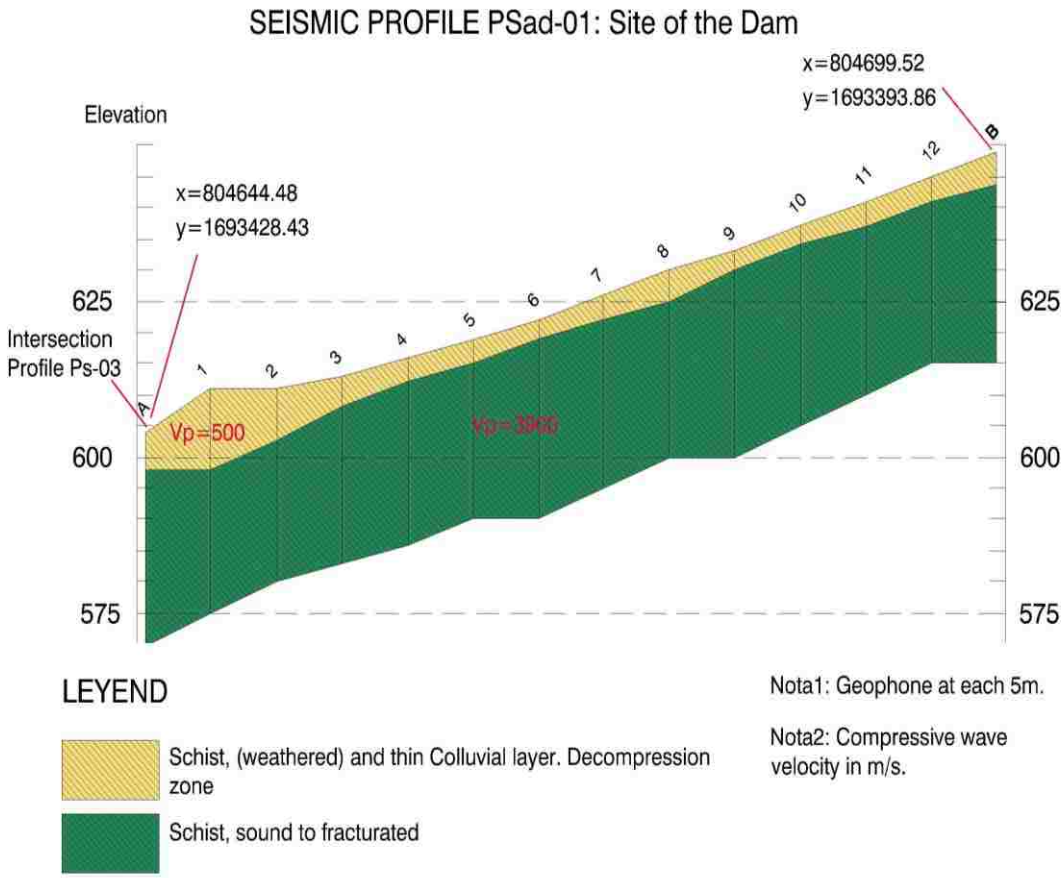


Figure 4.25. Geological-geophysical profile (Alvarado, 2005)

In order to evaluate shear wave velocities of the ground a cross hole test was performed. For the purpose of this test three boreholes were drilled on the right side of the Polochic River, upstream of the dam possible axis, approximately at elevation 625. Drill holes have a 114 mm diameter in order to allow the installation of 76.2 mm (3 inches) PVC pipe. Drilling was subsequently grouted to ensure that the tube walls having good adherence to the ground. The distance from the probe E2 (Transmitter 2) to survey R (receiver) is 5 m, distance from poll to poll E1 R (Transmitter 1) was 10 meters, Figure 4.26 shows the shear wave velocity, Vs, and compression waves velocity, Vp, versus depth.

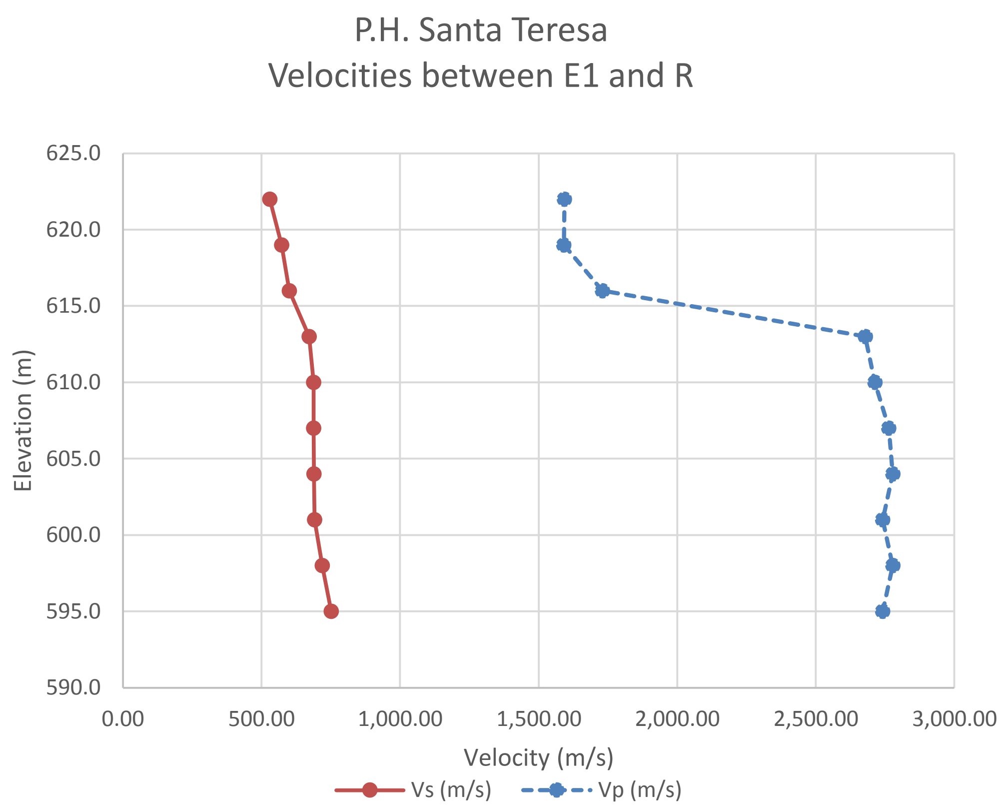


Figure 4.26. Graphic of shear and compression wave velocity (Alvarado, 2004)

4.2.2.3. Permeability tests. The exploration program included the testing of permeability within the boreholes in descending sections, every three to five meters. In the part of highly weathered soil or rock it was programmed to perform Lefranc permeability tests, by constant level upon tested section absorption. The Lefranc tests performed using a pneumatic packer testing a 30 cm section by lowering the casing to the bottom of the boreholes.

Pressurized permeability tests, Lugeon test, was executed in descending manner leaving an injection chamber of 3.00 m or 5.00 m long, isolated by double packer, in which case the lower packer was inflated inside terrain and had a length of 1.00 meters. Lugeon permeability tests were carried out using a reciprocating pump which develops a maximum flow of 136 liters a minute and can increase pressure more than 2,000 kPa. The full program pressures, for each section tested, was 150 kPa to 1,000 kPa, completing the test in five, seven or nine pressure steps, depending on whether the test was above or below 18 meters deep.

Permeability test were performed in the 3 boreholes located in the dam axis, D-1, D-2, D-3, a summary of the results of permeability test performed for D1 and D-3 are presented in the Table 4.7 and Table 4.8 respectively.

Table 4.7. Summary of the Luegon's permeability tests results in borehole D-1

Depth (m)	**Effective Pressure (kPa)**	**Lugeon Units**	**Permeability Coefficient (cm/s)**	**Depth (m)**	**Effective Pressure (kPa)**	**Lugeon Units**	**Permeability Coefficient (cm/s)**
3.50-6.50	170.6	193.8	2.05E-3	9.50-12.50	207.9	156.3	1.65E-03
13.00-16.00	214.8	0.3	2.80E-06	16.00-19.00	205.9	0.7	7.75E-06
19.10-22.10	176.5	1.77	1.88E-03	22.50-25.00	172.6	188.3	1.99E-03
25.00-28.00	219.6	165.6	1.75E-03	28.10-31.10	219.6	156.2	165E-03
31.15-34.15	150.0	125.5	1.33E-03	34.20-37.20	189.2	143.3	1.52E-03
37.25-40.25	198.1	119.8	1.27E-03	40.30-43.30	221.6	5.1	5.41E-05
43.35-46.35	457.6	0.8	8.84E-06	47.00-50.00	229.46	2.6	2.79E-05

(Adapted from Alvarado et al, 2005)

Table 4.8. Summary of the Luegon's permeability tests results in borehole D-3

Depth	Effective Pressure (kPa)	Lugeon Units	Permeability Coefficient (cm/s)	Depth	Effective Pressure (kPa)	Lugeon Units	Permeability Coefficient (cm/s)
12.00-15.00	785.8	5.8	6.24E-5	15.00-18.00	588.6	5.7	6.05E-5
18.00-21.00	786.7	20.5	2.22E-6	21.00-24.10	1,073.21	4.8	5.16E-5
24.10-27.10	1,073.21	6.4	6.79E-5	27.10-30.25	1,061.4	21.9	2.34E-4

(Adapted from Alvarado et al, 2005)

4.2.2.4. Local geologic context. The site topographically corresponds to a narrowing of the river that matches the outcrop quite foliated phyllites layers and layers of massive conglomerates and meta grauwacas that continuously outcrop on the left side, in the river bed and in the lower part on the right bank of the river.

Structurally, the layers of limestone, outcropping upstream apparently layers overlie phyllite and meta conglomerates; however, it is a stratigraphic series invested by a regional strike-slip fault that is located adjacent to the North block it. The metamorphic rock shows with a high degree of fracturing, as demonstrated by D-3 drilling executed on the left side of the river, this fracturing continues at depth and always coincides with the direction of the foliation of phyllites. There are a marked slippage between layers creating crushing zones and rock alteration, which causes open fractures, sometimes filled with detrital material, as in the case of drilling D-3 between 2.80 m and 10.90 m deep spanned almost parallel to one these structures, recovering sandy silt with many fragments of metamorphic rock as gravel and sand. The stratification of both limestone and meta conglomerates, phyllite, meta-greywacke and meta arkosas has a strong tendency to NW-SE orientation and the inclination of the layers is generally between 70° and 80° to the SW.

The contact between the limestone layer and the meta conglomerates is due to a reverse fault, which has produced dynamic metamorphism in the two units. In both drilling and surface outcrops can be seen intercalations of dark green phyllite with thin layers of limestone and limestone tend to be more siliceous always keeping a tendency to foliation as presented layers metamorphic. On the right side and the left side of the river it is a slip contact (fault) between layers, even with folds and deformations of drag between layers. The contact of the fault, formed by intensely crushed rock varies between 30 and 50 cm thick. In the contact of limestone with meta conglomerates there is great amount of limestone clasts, that can have 15 or 20 cm long and 3-5 cm thick, are observed. They are presented in flattened shapes, elongated and are guided by the very trend of foliation. However, layers of calcareous meta conglomerates are extend well into this metamorphic formation.

On the site of the dam axis, this metamorphic formation was investigated by borehole D-3, located on the left side of de rive at elevation 610.00, about 9 meters above the river level. D-3 borehole was executed inclined 30° 35° E N direction, reaching the depth of 30 meters. Borehole began directly meta grauwaca and meta conglomerate, foliation inclined 60°. Between 2.80 and 10.90 m depth the borehole entered in a series of open fractures that resulted in the recovery off sandy clay loam with lots of fragments of phyllite and meta grauwaca. From that depth and to the bottom of the borehole meta conglomerate, phyllites and meta grauwaca, with cericite over the fractures surface were found.

The determination of five rock cores taken between 7.45 and 29.82 m depth showed variable values of specific gravity, between 2.699 and 2.723, and values of 15,558.7 and 28,517.67 kPa on the unconfined compressive tests. The sample with higher values always refer to those having the highest concentration of limestone clasts; however, low values relate to samples with higher content of fine-grained meta grauwaca and phyllite. As mentioned above in this same borehole six Lugeon permeability tests were performed, from the depth of 12.00 meters. The values obtained are clearly related to the degree of rock fracturing and vary between 6 and 20 units Lugeon. This relatively high values indicate the need to inject the rock to waterproof it and consolidate it.

4.2.3. Grouting Method Determination. According to Jorge Hosttas, geologist of the project, from the results of the Lugeon tests, geological profile, and the expected loading conditions for the dam, it was determined by the consultant firm, Coyne et Bellier, that seepage must be controlled and a grouting cut-off curtain was proposed.

For the implementation of the cut-off curtain, two grouting options were considered by the owner technical team: conventional grouting method and the GIN method. The assessment of differences between of both methods is presented in the Table 4.9.

Table 4.9. Grouting method evaluation

Item Description	Conventional Grouting	GIN
Man power		Parameter monitoring technician / consultant
Equipment		Equal or advanced technology
Grout Mix	Different Densities Density variation in ascendant manner Limited by independent values of pressure or volume Probably greater consumption	Single Mix Limited by related values of pressure and volumen (GIN)
Additives	Related to grouted quantity	Probably higher cost per mix unit of volume
Control		Easier control (one mix)
Uncertainty	Equal or greater	
Supervision	Equal or greater	
Time		Equal or lesser

Conventional grouting requires less trained personnel and simpler equipment as major advantages. In the other side, GIN requires especial trained personnel and monitoring equipment but provides greater control that is particularly important for the geological setting of the project. The GIN method limits the volume of grouted material by combination of pressure and volume. Is important to mention that Limitation of grouted volume is the main feature that triggered the selection of GIN method.

4.2.4. Grouting Determination. Grouting determination requires the analysis of several conditions. The following analysis were performed in order to determine grouting specifications.

4.2.4.1. Structural analysis of rock. According to the geological report, there are four structures that dominate the behavior of the whole mass:

a) General layering of the rock. The strata or layers of rock are bound N 60° to 70° W dipping 76° to 80° to the SW. Strata cross the dam axis at an angle of 15° and inclination is upstream.

b) Rock layers are, on average, from 0.50 to 1.50 cm thick, the area is considered as "thick stratification".

c) There are two main families of joints that can be seen on both sides of the river: N 10 ° E dipping 70 ° NW (especially on the left bank) and N 70 ° E dipping 80 ° NW. On the slopes you can observe them as open joints of 1-3 cm and lead to the formation of unstable triangular prisms.

d) There are other families of less noticeable fractures and which usually are closed fractures that may extend 1.00 to 2.00 meters.

e) Most joints show the surfaces of their flat faces, little rough, in wide corrugations.

f) Inclination of the metamorphic rock is favorable for the foundation of the dam, both from the point of view of their stability and their imperviousness. Irregular layers of phyllites turn out to be the weakest in the whole rocks.

f) The families of joints whose general geometrical arrangement angles form 40° to 60° with the direction of the river, is not conducive to long lines in the direction of the river.

It is not excluded that there is a possible leak of groundwater describing a complex route either following a fracture with intersecting fractures another system. It might be more careful with the river parallel to the direction of joints, whose origin seems to be more recent than the two joints systems described and also give evidence of being natural drainage of the elevations that define the course of the river current

4.2.4.2. Complementary information analysis. Site investigation included the performance of 4 boreholes with continuous sampling and the testing of permeability, Lugeon type were made, systematically every three meters deep.

a) Index sample recovery: The recovery achieved in these four holes was 100%, except for some isolated sections.
b) RQD index: Contrary to the results of the recoveries of the RQD samples ranged from 45% to 54%, with several sections where the rate was "0".
c) In isolated cases the rock to have been affected by failures tends to be crushed.
d) Permeability tests Lugeon: The metamorphic rock has low permeability. The behavior of the absorption curve, during the tests performed, tells that little absorption at low pressure and moderate absorption at high pressure will occur, apparently washing fractures. It could be, in any case, but little fracturing extended open.

Taking into account the following factors:

- Overview of the metamorphic formation laying.
- Structures derived from the families of fractures and eventual failures in the area of the dam.
- State of the surfaces of open fractures.
- Index recovery and RQD Rock.
- Results of the permeability tests Lugeon.

It was estimated that the "void volume" in this type of rock is particularly "low" unless it comes to intercepting an open fracture or failure.

Also taken the area as a whole, it was assumed that these are three sectors with different behavior: a) the left bank covering Block 1 of the dam, where the two systems mentioned fractures dominate the structure of the rock .; b) the central part of the dam covering Blocks 2, 3, and 4 where the rock sample and better conservation; c) the right bank partly covering Block 5 and 6 where they meet again open fractures and there is a certain tendency to fractures parallel to the river.

During the first grouting campaign, for rock consolidation, an initial value for the void index of 15% was used to fix a volume. For the determination of the best GIN curve the same void index was used. Accepting this value of 15%, means that every cubic meter of rock have 150 liter of void that will be injected or filled with grout. This ratio is also interpreted as a consumption of 150 liter per linear meter of drilling, assuming an influence area of one square meter per linear meter of drilling.

Stage length is limited for the grouting pressure, grouting penetration and by borehole packer length in order guaranty borehole sealing. The total length of a simple borehole packer system used by local contractors is about 1.00 m, therefore normally and, in function of the ground condition, a multiple of this length is selected to define stage length. The grid of grouting points is determined by the ground type and condition. For the site a 3.00 m separation single line of grouting points was determined based on the estimation of rock soundness and continuity. The grid consists of primary vertical drill holes of 17.00 to 20.00 m depth, set to reach elevation 580, spaced each 6.00 m with secondary vertical drill holes of 12.00 to 15.00 m depth, set to reach elevation 585, spaced each 6.00 m forming a final grid of 3.00 m. The abutment on the right was approached differently due to the slope steepness and orientation of the rock planes using a single injection point with different inclinations. The length of the inclined drill holes was up to 20 m, with an inclination variation of 15 degrees between each drill hole. Based on these conditions a consumption of 450 liters for each 3.00 meters stretch was estimated. The spacing between injection holes pre supposed that injection should run and fill gaps in a surrounding distance of 1.50 m. This theoretically create a rock volume of 27,000 liters with a theoretical vacuum of 4,050 liters.

Pressure is limited by ground conditions and by the overburden pressure in order to avoid ground uplift or damage. Due to this limitation grouting pressure typically is increased with depth, so penetration is also increased. In the Table 4.10 a summary of the estimated pressure at the middle of each stretch is presented.

Table 4.10. Pressure at the middle of each grouting stretch

Stretch Depth (m)	**Middle Depth (m)**	**Overburden Pressure (kPa)**	**Height of the Water Column (m)**	**Hydrostatic Pressure (kPa)**	**Total Pressure (kPa)**
0.00 – 3.00	1.50	39.00	31.5	309.02	348.02
3.00 – 6.00	4.50	117.0	34.5	338.44	455.44
6.00 – 9.00	7.50	195.00	37.5	367.88	562.88
9.00-12.00	10.50	273.00	40.5	397.31	670.30
12.00-16.00	14.00	364.00	44.5	436.54	800.54

Notes: Rock specific weight: 26 kN/m^3, considerations for height of the water column: Dam height + underground water pressure.
(Adapted from Hosttas, 2011)

Pressures and estimated consumptions were determinate based on Lombardi's Table 3.8. The final solution is presented in the Figure 4.27. Selected grouting and GIN characteristics are presented in the Table 4.11:

Table 4.11. Grouting and GIN characteristics

Stage	**Stage Length (m)**	**Maximum Pressure (kPa)**	**Maximum Volume (liter)**	**GIN Intensity**
0 – 6.00 m	3.00	400	3,000	Moderate
6.00 – 9.00 m	3.00	600	4,500	High
9.00 – 20.00 m	3.00 or 4.00	800	4,500	Very High

(Adapted from Hosttas, 2011)

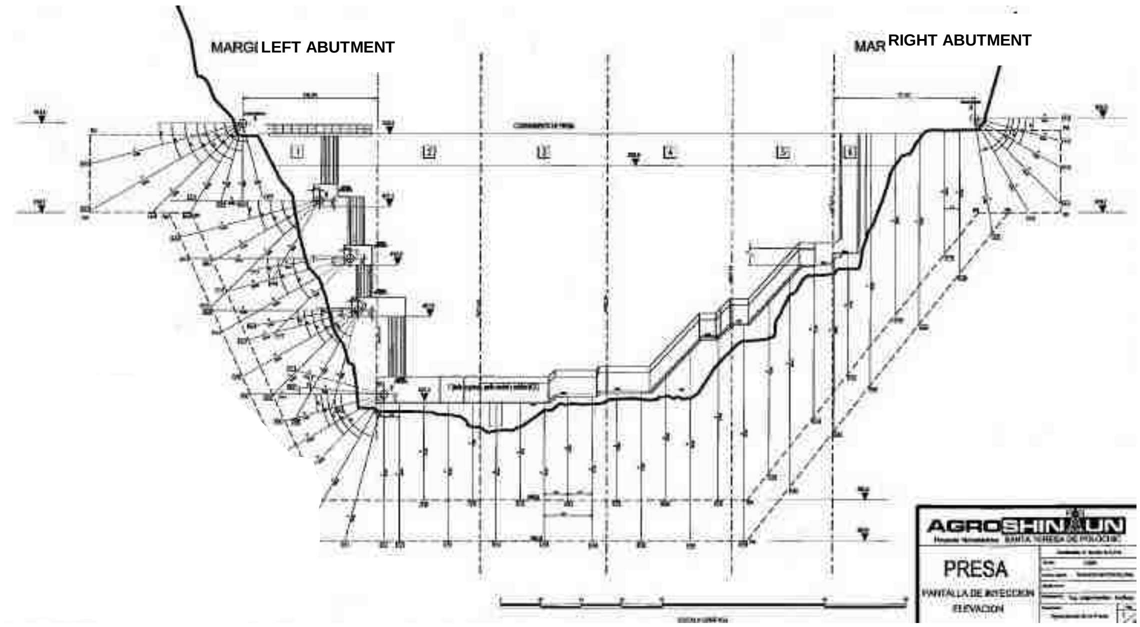


Figure 4.27. Cut-off curtain plan

4.2.4.3. Grout determination. In order to ease construction process a single grouting mix was designed. The initial mix design starts with a water:cement ratio by weight of 0.67 to 0.8:1. A series of tests were conducted to determine the best dosage of grout mixture to obtain a mixture having the desired characteristics possessed both its composition and dosage. The dosage of the grout was as follows:

• 1 sack of cement (42 kg)

• 35 liters of water

• 1.5 kg of bentonite

• 350 cubic milliliters of high-range water-reducing admixture (Rheobuild)

This grout had a viscosity of 32 seconds and 3% decantation. Grout has a theoretical density of 15.6 N/cm³. The grout density that was measured in the field was 14.12 N/cm³. The cement used contained pozzolan and has a strength of 28 N/mm^2 after 28 days in standard cement mortars.

4.2.4.3.1. <u>Bentonite.</u> Wyoming sodium bentonite manufactured by Baroid was used. This additive is used in a proportion of 1.32% or 0.55 kg per sack of cement. Bentonite was added to the mix without previous moisturizing. The mix was performed in a high speed mixing tank.

4.2.4.3.2. <u>Additives.</u> A plasticizer high-range water-reducing admixture was added to the mix, Rheobuild. This additive do not delay setting and provides high initial resistance without loss of final resistance. It is manufactured by The Chemical Company BASF.

4.2.5. GIN Implementation. The contract was awarded based on unit prices due to consumption uncertainty, this is a standard practice in this kind job. The ground grouting process was performed in two stages. The initial stage consisted in a rock consolidation grouting campaign. This stage was completed previous to the dam construction. The cut-off curtain was performed at the end of the dam construction from the dam surface and service galleries within the dam. The cut-off curtain construction process can be outlined as follows:

4.2.5.1. Sleeve installation. In order to reduce drilling and possible damage to the concrete of the dam a 10 cm diameter PVC sleeves were installed in the base of the dam during the initial construction stage. Figure 4.28 show PVC sleeve installed during construction.

Figure 4.28. PVC sleeves intalled during construction

4.2.5.2. Grouting station set-up. Grouting station were located in strategic points to reduce pumping distance as well as to ease grouting works. During the set-up of the grouting station a foreign technician from the grouting monitoring manufacturer, Jean Lutz, came to the project to perform the initial equipment calibration. This calibration included pressure, volume, grouting rate, stoppage parameters. Figure 4.29 show one of the two grouting stations.

Figure 4.29. PVC sleeves intalled during construction

4.2.5.3. Drilling. Due to access and site conditions, different types of drilling equipment were used. For drilling within the service galleries a Longyear LM-55 electric drill equipment, no combustion, were used. For the drilling from the dam surface an Ingetrol Explorer MD3 hydraulic equipment were used. Rotary core drilling system was used in order to avoid damage the concrete of the dam. Core drilling also allows to verify ground stratigraphy. Boreholes were drilled in NQ diameter, hole (outside) diameter: 75.7 mm; core (inside) diameter: 47.6 mm. Figure 4.30 shows drilling process with the Ingetrol Explorer MD3 equipment.

Figure 4.30. Drilling from the surface using wire line system

4.2.5.4. Grouting. The first step for grouting is mix preparing. The mix was prepared by two Cemix grout fabrication station manufactured by Atlas Copco. The grout station was equipped with a high speed mixer of 100 liters capacity a high turbulence agitator of 200 liters capacity and an endless screw type grouting pump capable to deliver 120 liters / min at pressures between 800 and 5,500 kPa. The equipment is driven by electric motors and the electro system is controlled by automatic valves. An automatic recording equipment Lutin model NX-B53 manufactured by Jean Lutz was used to control the grouting pump. This unit was calibrated for recording pressure, volume and flow, being programmed to use three curves GIN system parameters. This type of recorder prints the results on paper and simultaneously recorded on a card that is used to download the information to a computer and graph the results of each step of injection. Grouting was performed in 3.00 m stretch using a single packer. A split-spacing method was used. Grouting sequence is first downstream line, upstream line and at the end the center line. Figure 4.31 shows grouting works within the service gallery and Figure 4.32 shows Lutin NX-B53 grouting control equipment.

Figure 4.31. Grouting from acess gallery using wire line system

Figure 4.32. PVC sleeves intalled during construction

4.2.5.5. Instrumentation. After the cutoff curtain completion eight piezometers were installed. The main purpose of the piezometers was to monitor the effectiveness of the grout curtain and the pressures beneath the dam.

4.2.6. GIN Performance. The drilling work for the cut-off curtain began on November 17, 2010, grouting having begun until 24 March, 2011. The work began in the GI-4 gallery corresponding to the perpendicular to the axis of the river deeper gallery. The grouting process has to be flexible to adapt to ground conditions, therefore if high consumptions are detected or GIN stop occurs in adjacent boreholes additional grouting points are recommend. As well as whether permeability test show the need of permeability reduction and therefore additional grouting.

4.2.6.1. Cut-off curtain final geometry and construction sequence. During the construction of the dam 10 cm diameter PVC sleeve were installed for the line within the service gallery in the upstream side. The sleeves were installed vertical from the gallery floor to the level of the rock. These sleeves are spaced on average every three meters. The curtain on the right abutment corresponds to arrange of 6 drill holes of different inclination. The left abutment, for structural reasons, it was decided to drill and inject the curtain from the NE end of the main gallery and from three galleries that stand in the elevations 607, 614 and 621, by arranges who have between 5 and 7 holes of different angles. Superficially the curtain in this margin ending in a range of seven drill holes.

The works were performed with a spacing of 9 meters, for example, it began with drilling F-5, F-8 and F-11, proceeded inject wait for a period of 12 hours and restart the works in the drill holes F-6, F-9 and F-12. In the specific case of the incline arranges the grouting sequence was alternated, for example, the G-1 G-3 and G-5, once injected (the sequence from bottom to top) was waited 12 hours before drilling G-2, G-4 and G-6 surveys (where exist).

4.2.6.2. Central section of the dam. The central section of the dam corresponds to the work done from the gallery GI - 1 on 497.2 elevation and rises toward the right abutment up to elevation 614. The drill holes were made through a PVC pipe embedded in concrete to the level of the rock as mentioned above.

The section between the central part of the dam and the right abutment can be divided into 2 main sections:

a) the section between the F-5 and F-20 drilling, which shows that the absorption of cement are low to very low , between 2 to 3 liters of grout per meter (equivalent to 12 - 19 N of cement). The preliminary investigation noted very low permeability or impermeable sections. Clearly, the fracturing of the rock and are not laterally extended closed fractures. The geological surface mapping showed a very good quality rock with closed fractures, more than 1.00 meter apart, with possible fractures that show some oxides on their surfaces.

b) The second stage is between the elevations 615.4 and 631.40 that is outside the body of the dam on the right abutment. The section between the drill holes F-21 to F-26, covering the stretch of excavations on mean high right margin and where consumption is observed from 4 to 80 liters of grout per linear meter. The stretch was characterized not only by an increased number of open fractures of some millimeters to centimeters, corresponding to the fracture systems forming large triangular wedges into the rock (N 10° E 70° NW dipping and dipping N 70° E 80° NW). Surface was observed in these fractures with several meters long before becoming closed fractures.

Throughout the whole stage two drill holes are particularly interesting: the F-17 in the stretch from 12.00 to 15.00 meters deep took 2,394.4 liters (798 liters per linear meter or 5,180 N of cement per linear meter) and, close to it drilling F-19 in the sections 14 and 17 meters deep, 11.00 to 14.00 meters, 2.00 to 5.00 meters and from 0.00 to 2.00 meters by GIN sealed sections with 794 liters consumption linear meter, 469 liters, 883 liters and 258 liters per linear meter respectively. The only way to explain this anomalous behavior within the group, would accept that open fractures system N45 ° E with dips between 40 ° and 60 ° (seen inclined in the direction of the slope to the river) were open. These fractures were observed at different levels as continuous irregular open fractures even a few centimeters wide, undulating. As noted in the preliminary studies these fractures appear to be caused by the distension of the rock on the slopes, caused by the rapid excavation of the river canyon.

4.2.6.3. Exterior arrangement of the right abutment. The cut-off curtain on the right abutment has and arrangement of 6 drill holes that are inclined between 5° (vertical) and 90°. The drill holes have a variable length that ranges from 9.00 to 15.00 meters. Each section showed different absorptions at different depths ranging up to one liter to 140 liters per linear meter of grout, depending on the degree of fracturing and opening, these sections sealed "pressure". An exception is constituted by two sections, G-1 drill hole at the depth of 6.00 to 9.00 meters, drilling G-4 9.00 to 12.00 meters deep, which absorbed 471 and 676.9 liters per meter and sealed by GIN. That’s lead to believe that open fractures "N 45 ° E" crossed the system.

A different behavior was taken into drill hole G-6, between 0.00 and 4.00 meters and drill hole F-26 between 0.00 and 3.00 meters deep, stretches that cannot be sealed due to the presence of rock very fractured and a soil layer. When trying to grout these sections they communicated with the nearby perforations and through fractures floor, making it impossible seal it properly.

4.2.6.4. Left abutment grouting. Originally the drill holes for the cut-off curtain on the left abutment had been placed as a vertical deep drill holes and an arrangement from the top of the dam. The final configuration of abutment, almost vertical due to cut for dam foundation, led to change the process to a series of arrangement to run parallel to the river galleries located at different elevations. Including part of the GI-4 gallery that was isolated from the main section, at the gallery have diverted the river cut, the other galleries are in the elevations 607.60, 614.00 and 621.20. From the lower gallery they have four arrangements that ran underground and surface a fan made from the level 630.06.

4.2.6.4.1. Surface arrangement; arrangement A. It consists of seven drill holes ranging from vertical to a horizontal, the deepest reaches 21.00 meters and the shortest (vertical) reaches 9.00 meters. All the holes were grouted from the bottom up in sections of 3.00 meters.

A fairly consistent absorption result, per linear meter, was observed tending to be low in the range 2 to 30 liters and invariably sealed having reached the pressure specified for the depth considered. Exceptions happen in the drill hole A-1, in the sections between 6.00 and 9.00 meters, it took 1,000 liters per meter and stopped GIN system "volume"; the next section of this same stretch was stopped by GIN when it came to the absorption of 454.6 liters meter at a pressure of 340 kPa.

The drill hole A-6, which reached depth of 9.00 meters for its three grouting sections stopped by GIN: the stretch from 9.00 to 6.00 meters absorbed 560 liters of grout with a final pressure of 260 kPa; the stretch 6.00 to 3.00 meters took 469 liters to a final pressure of 320 kPa and finally the stretch of 0.00-3.00 meters took 629.6 liters final pressure of 260 kPa.

The drill hole A-7 had in the intermediate stretch of 3.00 to 6.00 meters took 1,000 liters and ended "volume" at a pressure of 20 kPa.

From a geological point of view this side of the river was always shown as affected by strong fracture systems, in particular, determined ways triangular wedges or limited by fractures more or less open hillside and it complicated by another system parallel to the direction of the river, open and tilt in the direction of the slope fractures. Together these fracture systems create areas of increased fracturing and even crushed rock; it is possible that the exceptional consumptions occurred due to grout crossing open fractures to very fracture areas.

4.2.6.4.2. Gallery GI-1; arrangement B. From gallery GI-1, located at elevation 621.20 a range of 5 boreholes were drilled from the horizontal to an inclination of 60° (30° from vertical), from 6.00 to 15.00 meters deep. It should be mentioned that the depths are indicated from the rock or natural ground; since the total depth of these boreholes from the surface of the gallery is greater for having been PVC pipe as guides to cross the layer of concrete between the gallery and rock.

The drill holes in this arrange are also characterized by low absorption, an average of 4 to 6 liters of grout with isolated sections that reached maximum 16 liters or 66 liters per meter, having sealed by "pressure". Two holes showed sections with have high absorption, they are: B-1, the length of 9-12 meters deep GIN ended with 294.3 liters per meter at a pressure of 500 kPa and; B-5 drilling that took 517.7 liters with final pressure of 290 kPa. The geological description of this level is the same as that described above for arrange "A".

4.2.6.4.3. Gallery GI-2; arrangement C. The GI-2 gallery is located at elevation 614.0, seven holes was performed ranging from 10 ° to 60 ° (taken from the horizontal) with de depth between 9.00 and 12.00 meters.

This range of grouting is also characterized by a majority of sections where absorption of grout was, with sections with a consumption of 2.00 liter per meter to sections with a consumption of 30 liter per meter and even 94 liters per meter; however they all sealed by "pressure". During frilling a high absorption of water occurred, but finally drill holes were sealed by "pressure". The drill hole C-5 (9.00 m deep) showed variable absorptions between 224 and 329 liters per meter grout.

The drill hole C-1 in the sections between 11.00 and 14.00 meters and between 9.00 and 11.00 meters sealed by GIN, the first with 540.4 liters and pressure of 270 kPa and the second with 267 liters meter with final pressure of 590 kPa.

4.2.6.4.4. Galley GI-3; arrangement D. The GI-3 gallery, located on the left abutment is on the elevation 607.6, from here eight boreholes with and inclination between 10° and 90° were drilled. The concrete part, starting at the drill holes, made through PVC pipes that were left embedded in concrete. The deepest drilling rock reached 14.00 meters and the shortest to 9.00 meters deep. All sections of grouting were 3.00 meters long and the grouting process started from the bottom to the top.

The middle and upper part of this drilling was characterized by little absorption of grout per linear meter, from 2 to 16 liters per meter. The deeper parts, with higher pressure, presented a higher absorption per meter about 41 liters per meter. Only in the drill holes D-1 and D-2 showed deep sections where the grouting was completed by GIN, in the case of drilling D1 sections between 11.00 and 14.00 meters and between 8.00 and 1.001 meters had absorptions 540 and 287 liters per meter with final pressures of 270 and 590 kPa respectively. Coincident with depth between 8.00 and 11.00 meters, drill hole D-2 grouting stopped by GIN with 455 liters of grout and a pressure of 320 kPa.

Except for the two drill holes mentioned above, grouting results generally show that the rock quality improves markedly in depth and fracture systems become more closed.

4.2.6.4.5. Galley GI-4; arrangement E. This arrangement was made from the deepest gallery (level 597.20) and includes the largest number of drill holes, three holes up comprising inclinations between 10° and 15° and seven holes ranging from 5° to 90°. The maximum depth of these boreholes reached 18.00 meters.

Most sections had low absorption, with a grouted volume varying between 4 and 10 liters meter; however some isolated stretch in which there were absorptions between 200 and 500 liters per meter, these sections sealed by "pressure". Exceptionally, a single stretch, corresponding to the drill hole E-6 from 10.00 to 13.00 meter depth ended by GIN with an absorption volume of 265.5 liters at a pressure of 560 kPa.

Similar to what is indicated for the section of the arrangement "D" results of grouting made broadly show that the quality of the rock is remarkably good with greater depth and fracture systems become more closed.

4.2.6.5. Grouting expansion program. During project execution in January 23, 2011 a supervision visit from the consultant's geologist, Silvio Ianos, recommend some modifications to the grouting program. After analyzing the results of the grouting campaign in each of the sections of the dam it was recommended strengthening the work performed adding another grouting line nearby and parallel to the sections where takeovers of grout were markedly higher.

The specific recommendations are bullet listed below:

- In the center section of the dam, two boreholes must be drilled F-19, called F-18A and F-19A, taking them deeper than the first boreholes. The result of these did not reflect what was obtained in drilling F - 19, but on the contrary had very low absorption between 1.5 and 4 liters of grout per linear meter, with one section that reached 20 liters meter.
- On the right abutment of the dam make two drill holes parallel to F-26, numbered holes F-25A and F-26A. Likewise, these two holes must be drilled two meters deeper more than original boreholes. The result was that the absorption in most of the sections was very low: between 1 and 5 liters per meter. The exceptions are the sections between 6.00 and 9.00 meters, in the first case took 90.8 liters of grout per meter and the second sealed by GIN with 326 liters per linear meter. Clearly these three holes passed through a gap area and high permeability has finally been waterproofed.
- Also on the right abutment in the arrangement "G" comprising six inclined perforations was requested to strengthen the G-4 drilling at a depth of 9.00 to 12.00 meters were sealed by GIN, absorbing 676.9 liters meter. 3A-G and 4A-G wells on both sides of first drilled. However the results showed very low absorption with isolated sections of 10 to 14 liters per linear meter. In this same arrangement G-5A borehole was drilled in order to reinforce waterproofing well G-6, the result obtained in drilling F-5A showed very low absorptions between 0.4 and 4.4 liter linear meter.

4.2.6.6. Grouting program summary. Estimating the grout quantities is one of the biggest challenges of a grouting project. The work quantities that were initially estimated are bullet listed below:

- 260 boreholes (between 5 to 30 m depth),
- 2,516 linear meters of drilling,
- 51.8 Ton of cement (1,234 bags) grouted (21 kg/lm),

- 18 Lugeon permeability tests, and
- 8 piezometers.

The work quantities that were actually performed are bullet listed below:

- 301 boreholes,
- 3,019.37 linear meters of drilling,
- 103.1 Ton of cement (2,456 bags) grouted (33.9 kg/lm),
- 21 Lugeon permeability tests, and
- 8 piezometers.

Boreholes were increased in a 15.8% and consequently the drilling amount increased up to 20%. Almost the double, additional 98%, of the volume of cement considered initially was grouted, but related to the drilling quantity the average consumption increased 61.9%. The additional Lugeon test were required for the particular areas that were considered in the expansion program.

The Table 4.12 summarize grout absorptions in the different dam locations.

Table 4.12. Average grout absorption and GIN stop

Dam Section	Borehole (Depth)	Average Grout Absorption (liter/m)	Observations
Central Section D_{max}: 30 m	F-5 to F-20	2 to 3	Very low
	F-21 to F-26	4 to 80	Very low
	F-19 (0 – 2 m)	129	GIN stop
	F-19 (2 – 5 m)	294	GIN stop
	F-19 (11 – 14 m)	156	GIN stop
	F-19 (14 – 17 m)	264	GIN stop

Table 4.12 Average grout absorption and GIN stop (cont.)

Dam Section	Borehole (Depth)	Average Grout Absorption (liter/m)	Observations
Right Abutment Arrangement D_{max}: 12 m	G-1 to G-6	1 to 140	Very low to Low
	G-1 (6 – 9 m)	157	GIN stop
	G-4 (9 – 12 m)	225	GIN stop
Left Abutment Arrangement D_{max}: 21 m	A-1 to A-7	2 to 30	Very low
	A-6 (0 – 3 m)	209	GIN stop
	A-6 (3 – 6 m)	156	GIN stop
	A-6 (6 – 9 m)	186	GIN stop
D_{max}: 15 m	B-1 to B-5	4 to 66	Very low
	B-1 (9 – 12 m)	294	GIN stop
D_{max}: 14 m	C-1 to C-7	2 to 94	Very low
	C-1 (9 – 11 m)	267	GIN stop
	C-1 (11 – 14 m)	270	GIN stop
D_{max}: 14 m	D-1 to D-8	2 to 41	Very low
	D-2 (8 – 11 m)	151	GIN stop
D_{max}: 18 m	E-1 to E10	4 to 10	Very low
	E-6	88	GIN stop

(Adapted from Hosttas, 2011)

The main reason for grout over consumption was the rock condition, even though not seen logical due to general low consumption of the rock mass. As mentioned above the rock condition is fairly good but in some particular stretches the consumptions increased exponentially, even whether this stretches are less than the 2% of the total length they doubled the grouting volume. In this 20 stretches, 2% of the total stretches, the total consumption was 10,162 liters that is the 71% of the total volume grouted. Other factor that increased the initial quantity estimation was the final configuration of the left abutment of the dam. The almost vertical form of the abutment forced to use inclined borehole arrangement that increased the number of boreholes and therefore the volume of grouting. The additional boreholes performed for the expansion grouting campaign, 10 drill holes and 144.00 meters, represents the 28.6% of total additional drilling and only 5.7% of the original drilling estimation. The GIN number stopped the 70% of the high consume stretches, as mentioned above the consumption of this stretches increased the 61.9% the overall average consumption, so GIN stoppage implied a reduction in the overall consumption. After the completion of the curtain wall and since its operation start date in July 2011 no noticeable problems have appeared. In general, and using the words of Mynor Celis, Operations Manager of the owner of Santa Teresa dam, the cut off curtain performed satisfactory with no inconvenience reported.

4.2.7. Lessons Learned. The objective of a case study is to present a detailed description of a constructed project and also share experience and knowledge. Even the most successful projects have lessons to be learned, throughout a life cycle of a project different lessons can be learned and opportunities for improvement can be discovered.

Identifying and documenting lessons learned provides a mechanism that communicate acquired knowledge more effectively and ensure that beneficial information is factored into planning, work processes, and activities for other similar projects.

Analyze lessons learned provides an opportunity to discuss successes during the project, unintended outcomes, and recommendations for similar future projects. It also allows the discussion of things that might have been done differently, the root causes of problems that occurred, and ways to avoid those problems.

The major benefit of compile the lessons learned is retain and document both successful, best practices, and unsuccessful project activities for future reference. This allows new projects to repeat successful activities and to avoid those that were not successful.

The lessons learned are presented asking if this was the right solution, identifying if were there improvements that could have been made and closing with what were the success factors. This represent a walk through the most important aspects of the case study.

4.2.7.1. Was this the right solution? Grouting programs are always challenging due to the uncertainties related to the ground condition, taking into account that even with a heavy ground investigation only a very small part of the ground is really tested. The decision of use GIN number method instead traditional grouting procedure, limited by pressure or volume and different mixes, was the right approach basically due to the fact that the 70% of the high consumption stretches were stopped by GIN. These high consumption stretches represents the 71% of the over consumption so if a traditional parameters, volume or pressure, were be adopted probably the over consumption had been greater. Other important feature of the solution is reliability, even when addition drill holes were required, a low absorption was observed in almost all the stretches. Also the use GIN method simplified the execution and quality control of the grouting program by use of a single mix instead of increasing density mixes.

4.2.7.2. Were there improvements that could have been made? Analyzing the project in retrospective, several improvements can be done. Improvement suggestions are separated for each stage of the project.

4.2.7.2.1. Geological geotechnical investigation. As shown above an extensive geological geotechnical investigation campaign was performed, some improvements could be recommend particularly for the dam site. After analyzing the overconsumption, it could be recommended to perform 4 to 5 additional boreholes reducing the distance between exploration points in the dam axis. Alternative boreholes also could be performed upstream and downstream from the footprint of the dam with purpose of estimate the absorption of the ground near to the dam axis. Also permeability test should be carryout within this additional boreholes. The starting depth of permeability test could be reduced, by means start in an upper level, in order to perform a better estimation of the ground permeability. Taking advantage of the boreholes piezometers could be installed in order to evaluate water level variation. Finally, a rock dilatometer test could be used to assess rock deformability.

4.2.7.2.2. **Design.** In general terms the design parameters provided a satisfactory performance despite the overconsumption. Even with the additional drill holes that was performed the selected parameter full filled their aim of form a cut off curtain. The main feature from the design that could be improved is the quantity estimation. The estimation of grout consumption could be increase in a 60% by means instead use an estimated consumption of approximately 0.5 bags of cement per linear meter of drilling use an estimation of approximately 0.8 bag of cement per linear meter of drilling. This estimated consumption can be clearly associated with the local geology, meta conglomerates, becoming in one of the most interesting lessons from this case study.

4.2.7.2.3. **Construction.** The construction of the cut off curtain was successful in terms of time and quality. Some improvement could be done to ease the work as the installation of PVC sleeves in the dam abutments. The sleeves saved drilling time and therefore money in the lower part of the dam, this practice could be repeat in the abutments for the grouting arrangements.

An alternative for assess grouting consumption is perform a grouting trial area that could be completed previous to construction stage. Also grouting methodology could be validated in this trial area.

A clause in contract could be included establishing a variation in the cost, reduction, in case that the overconsumption exceed certain amount, in example 50%. This could be very useful from the owner perspective.

4.2.7.2.4. **Instrumentation.** Grouting process was fully instrument with the Lutin equipment, keeping a record about consumptions, pressures, GIN stoppage and grouting duration. In order to ease readings of the piezometer, vibrating wire piezometers could be installed instead porous tip piezometers. Also as a dam is a long life structure and due to its function and size is a critical structure its monitoring could be eased by automatizing the readings.

4.2.7.3. **Success factors of the project.** As improvements are addressed also success factors have to be highlighted. Analyze the different options with mind openness and from the perspective of advantages, disadvantages and its estimated cost converges in a validated solution. The best solution is the one that provides satisfactory safety at the lowest cost in the right time.

The GIN method for grouting provides a measurable procedure to design and install grout curtains. The systematic filling of the larger to smaller fissures during each successive grouting stage by tracking and limiting (first) the grout volume, then the combination of volume and pressure, and finally the grout pressure results in confidence that the grouting procedure will be effective.

The use of a single mix eased grouting process, reducing construction decisions, and supervision necessity. Grout mix workability and performance were very good, as well as the overall performance.

The construction of service galleries contributed to ease the access and performance of the work, but also contributed to dam construction process avoiding stops for grouting allowing perform grouting works with the dam construction completed.

The use of portable equipment eased drilling works, particularly from the service galleries. Also the establishment of two separate grouting stations helped to accelerate works.

4.3. SEISMIC GROUND IMPROVEMENT: STONE COLUMNS PERFORMANCE FOR A POWER PLANT IN THE SOUTHERN ALLUVIAL PLAINS OF GUATEMALA

4.3.1. Introduction. As part of the opportunity of energy market grown in Guatemala many projects for power plants were launched. Each of this projects have particular advantages as location, services nearby, also disadvantages as ground conditions or permits. In the particular case of the Genosa Power Plant site has several advantages as highway and transmission lines vicinity, is located near to Puerto Quetzal, the principal Seagate of Guatemala, Figures 4.33 and 4.34 shows its exact location. Also other similar projects are located in the near area, thermic power plants, this suggested that permits could be obtainable.

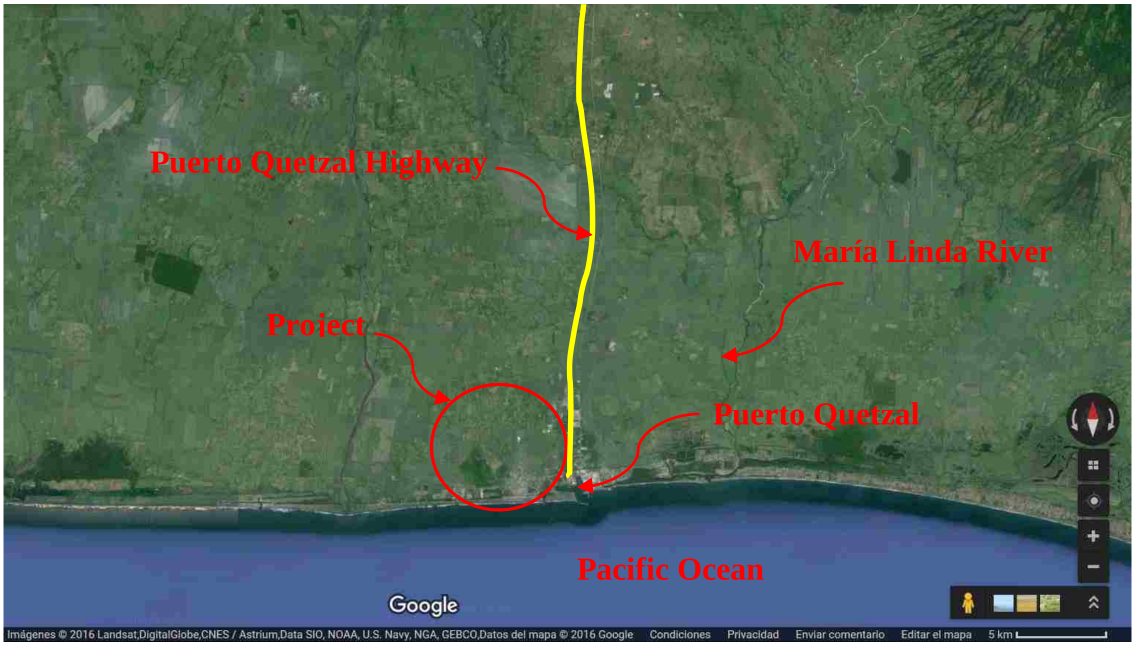


Figure 4.33. Location of Genosa Power Plant

Figure 4.34. Location of Genosa Power Plant

The power plant consists of a main structure with three engines, auxiliary structures as cooling tower, electrical substation and a storage tanks area, Figure 4.35 shows an actual view of the power plant.

Figure 4.35. Panoramic view of Genosa Power Plant

The main disadvantages for this project was the ground conditions. The area is historically related to settlement problems of different structures as warehouses or liquid gas storage tanks. Also, the site is located very close to Pacific´s subduction zone, been a very high intensity seismic zone. Geotechnical investigation shown the existence of peat lenses that are directly related with the historical settlement behavior. In addition ground water level is almost at the surface level and the ground is a sequence of sand layer with different densities increasing the risk of liquefaction during a seismic event, Figure 4.36 and 4.37 shown liquefaction during 1976 earthquake. These conditions prepared the scenario for an especial foundation solution in order to take advantage of the site benefits. The special foundation should to avoid or reduce settlements but also to mitigate liquefaction risk. Traditional piling approach was analyzed but after an initial assessment of the cost of a piling solution another solution was required. A ground improvement solution arose as an alternative finally converging in a vibroreplacement solution (stone columns).

Figure 4.36. Sand blows located near to Motagua River during 1976 earthquake (Plafker, 1977)

Figure 4.37. Flow liquefaction in a bank of Motagua River during 1976 earthquake (Plafker, 1977)

The aim of this case study is present the complete process and sequence for the design and construction of a ground improvement using de Stone Columns in the southern alluvial plains of Guatemala. As well as present the lessons learned during this process.

4.3.2. Site Characterization. An extensive geotechnical site investigation was performed. The investigation program consisted of: boreholes, undisturbed samples, standard penetration tests (SPT), cone penetration tests (CPT), shear wave velocity measurements, laboratory testing including: consolidation tests, sieve analysis, direct shear test. In the Figure 4.38 is shown the general geological setting of the project area, meanwhile Figure 4.39 shows the area prior construction.

4.3.2.1. Local geologic context. The shallow regional geology consists of Quaternary alluvium (Qa) within the Maria Linda watershed. These soils, consisting of interbedded, underconsolidated and poorly drained layers of sands and silts, and to a lesser extent gravels and clays, having erratic depositional patterns typical of coastal plains. The Quaternary alluvium (Qa), about 1,000 m thick, is underlain by rocks of the Tertiary and rock and sands of the Cretaceous, followed by Ophiolitic Basement.

Figure 4.38. Area geological map (Instituto Geografico Nacional, 1970)

Figure 4.39. Project area prior construction works

4.3.2.2. CPT and boreholes. Site investigation started with the performance of Cone Penetration Tests (CPT). A total of 9 cone penetration tests (8 CPTu and 1 SCPTu) were performed in order to assess different parts of the project, Figure 4.40 shows the location of the explorations. The soundings were halted when refusal conditions were met. The cone penetrometer tests were carried out using an integrated electronic seismic piezocone. The piezocone used for the soundings completed on the land was a compression model cone penetrometer with a 15 cm^2 tip and a 225 cm^2 friction sleeve. The cones are designed with an equal end area friction sleeve and a tip end area ratio of 0.80. The piezocone dimensions and the operating procedure were in accordance with ASTM Standard D-5778-07.

Pore pressure filter elements, made of porous plastic, were saturated under a vacuum using glycerin as the saturating fluid. The pore pressure element was six millimeters thick and was located immediately behind the tip for all soundings. The cone was advanced using a 14 Ton capacity Ramset mounted to a Klemm 807 drill rig. The following data were recorded every five centimeters as the cone was advanced into the ground:

- Tip Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (U)

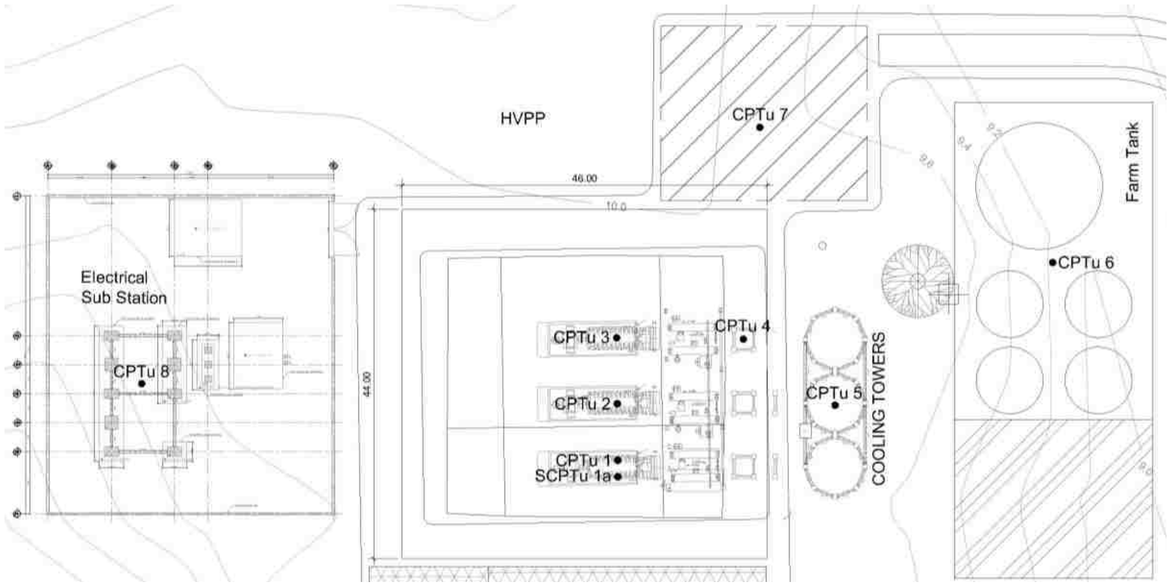


Figure 4.40. CPTs location

Before each sounding a complete set of analog baseline readings are taken with an integrated multi-meter and compared with the digitized value on the computer screen. This provides a check on the analog to digital conversion board. Evaluation of the analog baselines is key to consistent readings. The baseline data should be stable and should not wander excessively during the course of a sounding. Baseline data can be used to apply corrections to the cone data where necessary. For this project, the baseline shift from sounding to sounding was small, typically less than 0.1% of full scale, and no data corrections were applied.

When cone penetration is stopped, the piezocone essentially becomes a piezometer. While stopped, pore water pressures are automatically recorded at five-second intervals and the readings are stored in a dissipation file. Dissipation data can then be plotted onto a dissipation curve consisting of pore water pressure (U) verses time (t). The shapes of dissipation curves are very useful in evaluating soil type, drainage and in situ static water level. A flat curve that stabilizes quickly (i.e. less than 30 seconds) is typical of a freely draining sand. In this case, the final measured pore water pressure is the static in situ water pressure.

Soils that generate excess dynamic pore water pressure during penetration will dissipate this excess pressure when penetration stops. The shape of the dissipation curve and the time of dissipation can be used to estimate c_h, the coefficient of consolidation that can in turn be used to calculate K_h, the horizontal permeability.

The data from the soundings was plotted using the computer program ScreenZW. classification as part of the plot. The soil classification is based on the classification chart presented in the Figure 4.41. The plot of the CPT performed in the engine No. 1 is shown in the Figure 4.42. ScreenZW was developed by ConeTec Inc. and it incorporates soil behavior type (SBT).

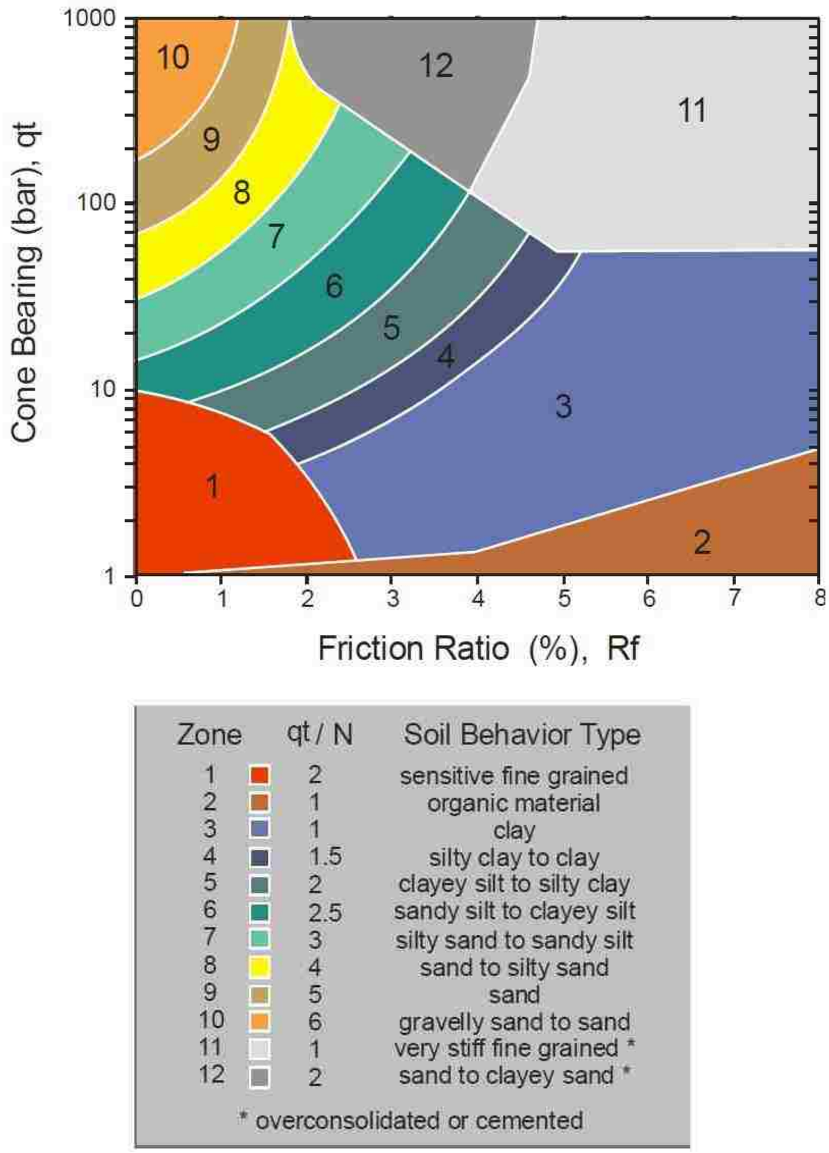


Figure 4.41. Non-normalized behavior type classification chart

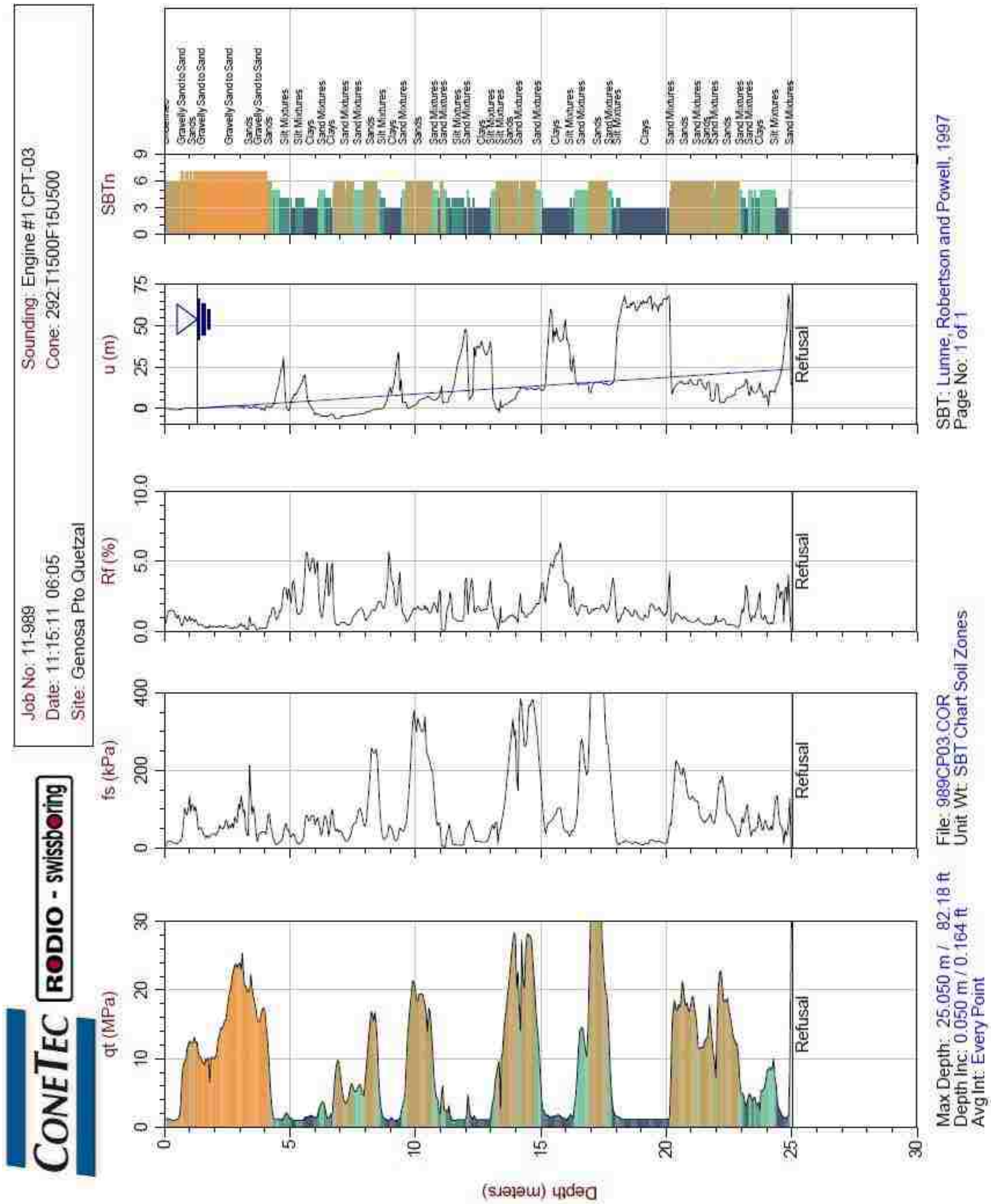


Figure 4.42. CPT plot for engine No. 1

The Figure 4.43 show the performance of the CPTs and Table 4.13 shows a summary of the works performed.

Figure 4.43. CPT performance

Table 4.13. Summary of cone penetration tests

CPT Type and Number	Location / Structure	Completion Date	Depth (m)	Ground Water Table (m)	Comments
CPTu1	Engine # 3	03/14/03	2.8	1.3	Refusal
SCPTu1a	Engine # 3		13.3	1.3	Refusal / Seismic
CPTu2	Engine # 2		24.7	1.3	Refusal
CPTu3	Engine # 1		25.1	1.3	Refusal
CPTu4	Stack # 1		23.3	1.3	Refusal

Table 4.13 Summary of cone penetration tests (cont.)

CPTu5	Cooling Tower # 3		24.3	1.3	Refusal
CPTu6	Farm Tank	04/08/03	16.1	0.5	Refusal
CPTu7	Warehouse	06/23/04	25.1	1.3	Refusal
CPTu8	Transformer	06/23/04	25.6	1.0	Refusal

In order to complement the information from CPTs one borehole was drilled using rotary core drilling system in HQ diameter, hole (outside) diameter: 96 mm; core (inside) diameter: 63.5 mm. The borehole was drilled using a Mobile Drill B-57 drill rig, Figure 4.44 shows borehole drilling. From this borehole undisturbed samples from soft soil, were extracted using a Shelby tube, outside diameter: 50.8 mm; length 762 mm.

Figure 4.44. Borehole drilling using rotary core system

4.3.2.3. Shear wave measurement. The shear wave measurements were taken in sounding Engine #3 – CPT-01a at 1.00 m intervals. During seismic testing, the seismic signals were recorded using a geophone mounted in the cone and an up-hole integrated digital oscilloscope. A sledge hammer hit against a beam was used for the seismic source. Normal reaction for the beam was provided by the dead weight of the rig placed upon the beam. A schematic of the shear wave testing configuration is shown in Figure 4.45.

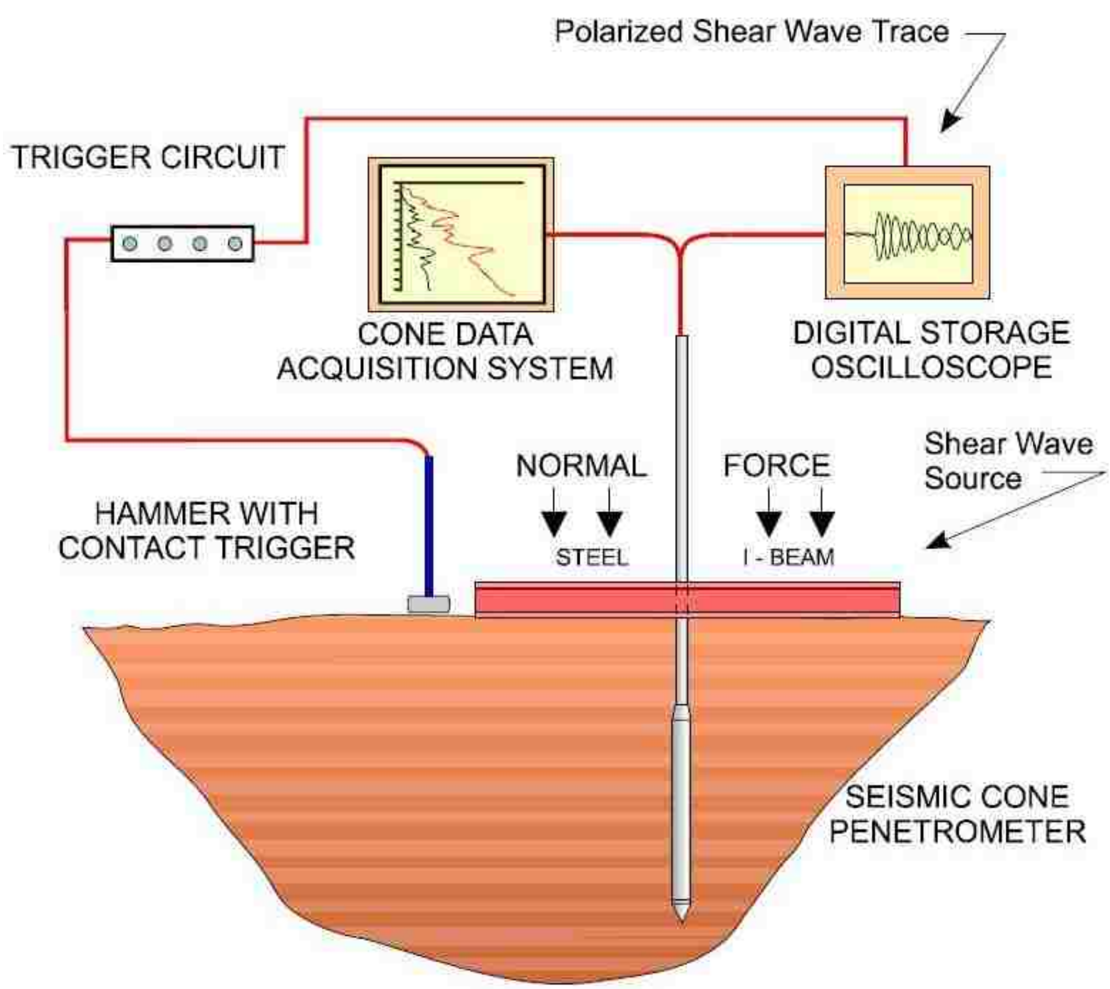


Figure 4.45. Schematic of shear wave testing configuration presented by Conetec

The resulting shear wave velocity calculations are plotted in the Figure 4.46. From this profile the site was classified as Type E, soft soil, according to the Guatemalan Society of Structural and Seismic Engineering, AGIES, in its publication Structural Safety Code, NSE, volume 2, Table 4.4.

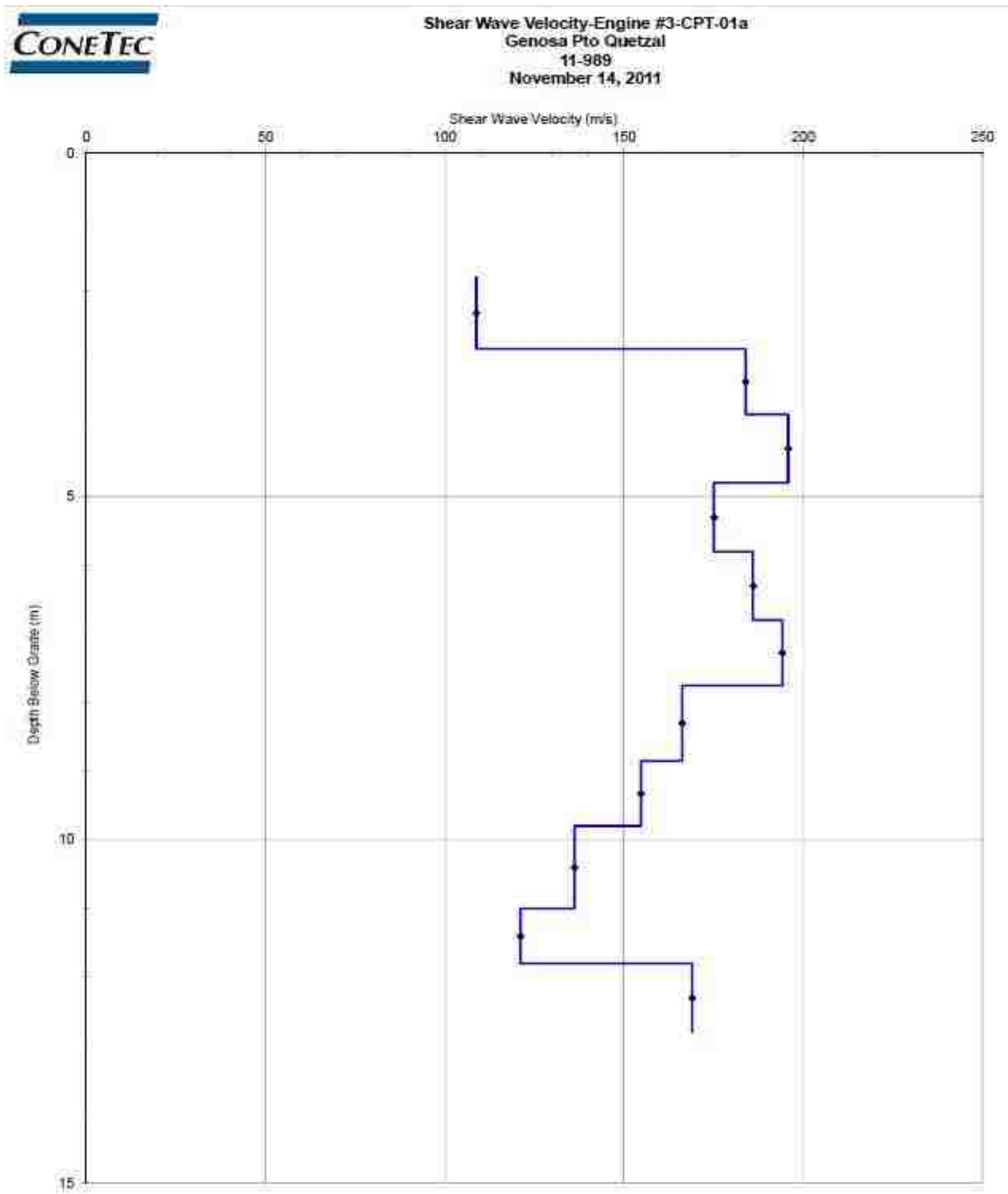


Figure 4.46. Shear wave velocity profile

4.3.2.4. Laboratory program. Laboratory testing program consisted of identification tests as moisture content, grain-size and wet density also consolidation tests were performed. The strength parameters were determined using the direct shear test for intact soil samples recovered. Complete laboratory test results are included in the appendices. The summary of the laboratory testing program is presented in the Tables 4.14, 4.15 and 4.16.

Table 4.14. Laboratory test program results summary, direct shear

Sample number	Depth (m)	Natural moisture content (%)	Wet density (kN/m^3)	Cohesion (kPa)	Internal Friction Angle
S-1	4.95-5.60	53.4	16.8	42.16	20°30’
S-2	8.00-8.75	69.0	14.0	23.96	26°00’

Table 4.15. Laboratory test program results summary, grain size distribution

Sample number	Depth (m)	Grain Size Distribution Sieve Size in mm											
		75	50.8	38.1	25.4	19.1	12.7	9.52	4.76	2.00	0.42	0.18	0.074
2-A	1.50-1.95							100	99	97.8	78.2	62.3	39.1
3-A	3.00-3.45				100	84.9	80.8	79.0	76.8	71.4	33.5	13.4	4.2
5-A	6.00-6.05					100	97.2	96.4	96.1	95.0	93.7	92.5	90.5
6-A	7.20-7.65					100	86.9	79.6	78.2	77.6	76.3	54.5	22.6
9-A	10.75-10.80			100	69.1	69.1	69.1	69.1	68.3	63.1	59.6	58.4	54.5
11-A	12.25-12.70							100	99.3	99.1	98.5	96.4	81.1
12-A	15.25-15.70							100	99.1	98.9	98.6	94.3	40.0

Table 4.16. Laboratory test program results summary, consolidation test

Sample number	Depth (m)	Wet Unit Weight (kN/m³)	Moisture Content (%)	Consolidation Test (According ASTM D2435)			
				Pre-consolidation Pressure (kPa)	Voids Ratio e_o	Specific Gravity	Compression Index c_c
S-1	4.95-5.60	15.47	66.2	578.79	1.81	2.67	0.42
S-2	8.00-8.75	15.04	76.7	529.74	2.12	2.71	0.79

4.3.2.5. Simplified soil profile. As result of site characterization program a simplified soil profile was determined.

Elev. 10.00

1 – Clayey Sandy Gravel

4.00 m q_t = 5 to 20 MPa

R_f = 0.5 to 1% Elev. 6.00

2 – Sandy layer with Silty and Clayey Lenses

16.00

q_t = 1 to 5 MPa

R_f = 1 to 5% Elev. -10.00

3 – Silty Sand and Sandy Layers with Silty and Clayey Lenses

5.00 m

q_t = 0.5 to 4 MPa

R_f = 0.5 to 1% Elev. -15.00

Where: q_t is CPT tip resistance, R_f is the friction ratio.

4.3.3. Stabilization Method Selection. Different options were evaluated, from a deep foundation solution to different densification methods. As mention above to problems have to be solved or mitigated, liquefaction risk and settlement problems. Below is the analysis of the different options:

Deep foundation alternative using bored piles.

- Bored piles, caissons, were selected. Concrete driven piles are discarded due to possible length (20.00 m) probably requires joints that are not locally manufactured. Steel driven piles are not in stock in the probable diameter despite that the driven equipment (hammer and cranes) is available in the area.
- Piles are structural elements that brings reliability.
- Piles will transfer loads to deeper and denser ground layer.
- Piles are regularly performed and can be relatively easily tested.
- Equipment is available in the area.
- Piles do not mitigate liquefaction risk.
- In this ground conditions negative friction phenomenon have to be considered. The negative skin friction have to be considered in a stretch from 4.00 to 20.00 m depth, due to the overlaying of very soft layer between sand layers.
- Negative friction increases pile section and depth and therefore increases cost.

Ground improvement alternatives

The initial assessment included:

- Ground Pre Loading. Normally preload requires certain amount of time and in order to reduce time or load dimensions drainage aid is require as example: vertical drains.
- Grouting. Treatment area is relative small, but ground layering indicates that consumptions could be very high and/or unpredictable even with a containing treatment.
- Dynamic compaction. The depth of the treatable ground is about 20.00 limiting the effect of the compaction. Also there are no good experiences with this kind of treatment in the area in a other project.
- Vibroflotation. Vibro compaction was discarded due to the presence of layer of fine grained material.

Vibroreplacement (Stone Columns):

- Mitigates liquefaction, allows ground drainage.
- Compact the ground in the column perimeter.
- It can be performed in relative fast manner.

- Equipment is available in Central America.
- Requires well trained personnel.
- Has a relative limitation in the bearing load improvement.
- It was performed only once in Guatemala.

The more feasible options were Bored Piles and Vibroreplacement by means Stone Columns. The bored piles was discarded because they do not mitigate liquefaction by themselves and additional works have to be performed as vertical drains.

After analyze the options including safety, technical viability, costs, construction and time vibroreplacement solution, stone columns, was selected. The best solution is the one that provides satisfactory safety at the lowest cost in the right time.

4.3.4. Solution Calculation and Design. The contract mode was design and build, the contractor as subsidiary of a French geotechnical contractor, Soletanche-Bachy, ask for advice to his head office. The design was performed by the Vibroflotation Group, part of Soletanche-Bachy, a global expert in ground improvement design and execution.

4.3.4.1. Compactability determination. As presented in the simplified soil profile the ground to be improved consists of 25.00 m of loose to dense sand and soft to firm silty to clayey material. The first step for design of any vibroflotation technique is determine if the soil vibrocompactable or need stone backfill. For this purpose Brown (1977) developed a chart based mostly in the soils grain size distribution. This chart is presented in the Figure 4.47 and has the following zones.

Zone A: The soils of this zone are very well compactable.

Zone B: The soils in this zone are suited for Vibro Compaction. They have a fines content of less than 8 to 10 %.

Zone C: Compactable. Stone backfill is needed if the fines content is higher than 10%.

Zone D: Stone columns are a solution for a foundation in these soils. There is a resulting increase in bearing capacity and reduction on total and differential settlements.

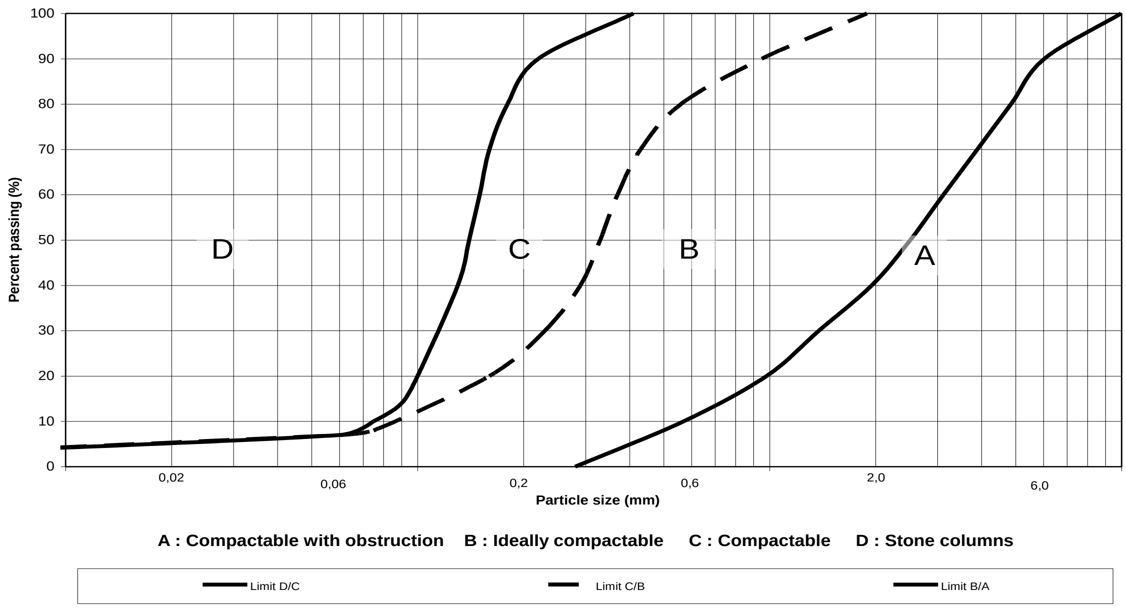


Figure 4.47. Vibrocompactability chart (modified from Brown, 1977)

The bulk of the soil lies in zone D of the vibrocompactability chart. Therefore, the "weaker" layer, silty and clayey material, leads to the need of stones to be improved. But, the combined action of vibration and stone feeding will enhance the ground improvement efficiency in sandier material to reach a higher range of resistances (especially in the upper part where adequate bearing capacity is required).

4.3.4.2. Liquefaction assessment. As mentioned earlier the project is located very close to the subduction zone of the Pacific Ocean. According to the Guatemalan Society of Structural and Seismic Engineering, AGIES, in its publication Structural Safety Code, NSE, volume 2, Table 4.5 the Moment Magnitude, Mo, is equal or greater than 7.0. Also from this publication the estimated Peak Ground Acceleration, PGA, for the project area and for an ordinary structure is 0.39g.

Liquefaction assessment was performed determining the Cyclic Stress Ratio (CSR) And Cyclic Resistance Ratio (CRR) using the criteria presented by Robertson and Wride in 1998 that is presented in the Figure 4.48. According to grain size analysis sand layers has a fines content, FC, less than 5%.

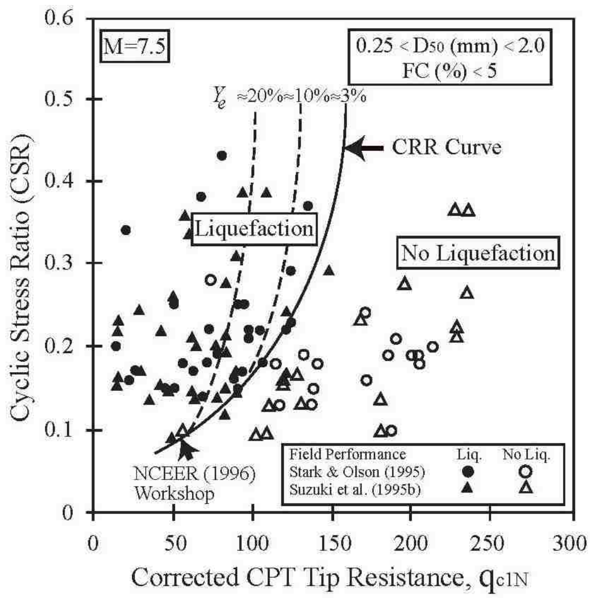


Figure 4.48. Curve recommended for calculation of CRR from CPT data (Robertson and Wride, 1998)

The final analysis determined that the upper sand layers, between 5.00 to 10.00 m, are susceptible to liquefaction. In some cases there are sand layer as deep as 16.00 m that are also susceptible to liquefaction.

4.3.4.3. Mitigation of liquefaction risk. The depth of stone columns technique was set down to 20.00 m in order to reduce settlements and mitigate the risk of liquefaction at depth under the more sensitive structures (engines & tanks). Underneath 20.00 m, liquefaction is not a significant problem because of the higher confinement and the smaller influence at surface. It is common practice worldwide that the cut-off level of any works dealing with the mitigation of the risk of liquefaction can be 10.00 to 20.00m. Under the less sensitive structures (warehouse, transformers and cooling towers), a treatment of 13.00m deep, down to a dense sandy layer was suggested but not performed.

Based on the experience of Vibroflotation Group an inclusion factor was set in 10% in order to improve ground conditions. This inclusion factor is directly related to the achievable column diameter with the equipment available. The 10% inclusion factor translates into a 2.70m square grid spacing with an average stone column diameter of 0.95m. Indeed, this system is self-regulating and therefore, bigger stone columns will be constructed in softer ground and smaller ones in denser sands (to be also improved due to vibrations).

4.3.4.4. Settlement calculations. As part of the analysis both settlement without and with ground improvement were calculated.

4.3.4.4.1. Settlement with no ground improvement. In order to estimate the settlement of the original ground under the loads of the structures, the main following assumptions are taken into account herein:

Over a depth of 25.00m for a general profile and in average it can be draw from the CPTs. The cumulative thickness of silty material is 10.00 m while there are 15.00 m of sandy material already in a dense state.

There is a 4.00 m very dense sandy gravel layer in the first meters that can play the role of a load distribution layer.

The installation of an additional working platform to raise the general elevation of the project by 1.00 m will increase the total thickness of the stiff upper layer to a total of 5.00 m of very good ground conditions for bearing capacity and an excellent distribution of the loads at depth.

Distributed load on the original ground below the upper dense sandy layer (engines & tanks) can thus calculated as the sum of the different loads:

1.00m additional fill.

20 kPa distributed load on the deeper layer (under 5.00 m depth) of the heavier structures like engine building.

40 kPa (considering 20 columns of the structure of 150 Ton and 3 engines of 530 Ton over a total area of 1,100m²).

Total of 60 kPa as a distributed load at the top of the deeper layer (under 5m depth).

Under the structures thanks to the spreading layer, oedometric conditions can be assumed.

Average tip resistance q_c in silty layers: 1.4 MPa

Ratio E_{oedo} / q_c = 4.0 as per Menard, L. (1998); Cassan, M. (1988) (Eq. 4.15)

E_{oedo} = 5,600 kPa

Average tip resistance q_c in sandy layers: 15 MPa

Ratio E_{oedo} / q_c = 3.3

E_{oedo} = 50,000kPa

Total settlement:

W_{total} = 60 kPa x (10.00m / 5,600kPa + 15.00m / 50,000 kPa) = 12.0 cm

4.3.4.4.2. <u>Settlement with ground improvement.</u> In order to estimate the settlement of the improved ground under the loads of the structures, the main following assumptions are taken into account herein:

Ground improvement treatment down to 20.00 m depth.

The 5.00m remaining untreated meters are sandy material and bring negligible settlements.

Ground improvement factor calculation is based on the homogenisation method proposed by Heinz Priebe, 1995. The homogenization method is linked to the inclusion factor, the in-situ elastic modulus, $E_{in\text{-}situ}$, and stone column elastic modulus, $E_{stone\ column}$. The combination of these parameters in proportion to the inclusion factor (IF) leads to a new improved and equivalent elastic modulus, $E_{improved}$:

$E_{improved} = [IF * E_{stone\ column} / E_{in\text{-}situ\ soil} + (1 - IF)] * E_{in\text{-}situ\ soil}$ (Eq. 4.16)

Is important to mention that the modulus ratio between stone columns and soil shall be taken between 6 and 10.

One assume an average ratio $E_{stone\ column} / E_{in\text{-}situ\ soil}$ of 8.

Therefore, with a 10% inclusion factor the ground improvement factor is 1.7.

In addition, a 50% improvement of the surrounding ground is taken into account because of the sandy and silty natures of the material.

The overall and global ground improvement factor that can be assessed is thus: 1.7x1.50 = 2.55.

Therefore, forecasted settlements under the heavier structure is around: Settlements with no ground improvement / ground improvement factor = 12.0 / 2.55 = 4.7 cm rounded to 5.0 cm.

The whole structure will settlement in a very short period of time thanks to the draining effect of the stone columns and thus the larger part of the settlements will take place during construction.

4.3.4.4.3. <u>Calculation for smaller load and shallower treatment depth.</u> In the less sensitive areas (cooling towers, warehouse and switchyard) a shallower treatment was suggested regarding the settlement issue with the following assumptions:

Distributed load on the original ground below the upper dense sandy layer can then calculated as the sum of the different loads:

1m additional fill: 20 kpa.

Distributed load on the deeper layer (under 5.00 m depth) of the ligher structures: 20 kPa.

Total of 40 kPa as a distributed load on the deeper layer (under 5.00 m depth).

Average tip resistance q_C in silty layers: 1.4MPa

Ratio E_{oedo} / q_c = 4 as per Menard, L. (1998); Cassan, M. (1988)

E_{oedo} = 5 600kPa

Average tip resistance q_C in sandy layers: 15MPa

Ratio E_{oedo} / q_c = 3.3

E_{oedo} = 50 000kPa

Same ground improvement factor but considering that above 13.00 m depth there are 6.00 m of silt and 7.00 m of sand in average and below 13m depth there are 4.00 m of silt and 8m of sand.

Total settlement after ground improvement:

W = 40 kPa x [(6.00m/5,600 kPa + 7.00m/50.000 kPa) / 2.55 + (4.00m/5,600 kPa + 8.00m /50,000kPa)]

W = 2cm + 3.5cm ≈ 6cm.

4.3.4.5. Load bearing capacity. A 200 kPa load bearing capacity was proposed with a minimum safety factor of 3. The two approaches herein show that the natural ground already provides this bearing capacity.

4.3.4.5.1. <u>Undrained cohesion approach (material with cohesion).</u> As already stated in the assumptions for the settlements calculations, the average tip resistance is 1.4 MPa. If we assume a single layer composed of cohesive material, assuming that the undrained cohesion (C_U) is constant. Therefore the N_C factor is equal to a minimum of 5 (as □□+ 2 = 5.14, simplified to 5). A factor of safety, F.S., of 3 is then applied to calculate the allowable bearing capacity q_{all}.

$$q_{ult} = C_u \cdot N_c = \frac{\overline{q}_c}{10} \cdot Nc = \frac{1400}{10} \cdot 5 = 700kPa \qquad \text{(Eq. 4.17)}$$

$$q_{all} = \frac{q_{ult}}{F.S.} = \frac{700}{3} = 230kPa \geq 200kPa \quad \text{(Eq. 4.18)}$$

This is obviously too pessimistic an assumption and most of the foundations are shallow foundations like isolated footings where the bulk of the stresses are concentrated in the upper meters (around 1.50 m x width of the footings).

4.3.4.5.2. Pressuremeter method. The pressumeter method appears to be the most suitable approach for footings of a maximum width of 3.00 m. This footing width will required to reach an average cone resistance in the upper 4.5.00 m to ensure the bearing pressure of 200kPa.

These first meters are dense sandy material and therefore the following rules can be applied:

Ratio between cone penetration test resistance (q_c) and pressuremeter resistance (p_l) in sandy to clayey silt material:

$$q_c / p_l = 10$$

Average tip resistance to reach after ground improvement of 8MPa:

$q_c = 8MPa$ which leads to $p_l = 0.8MPa$

$$q_{adm} = \frac{p_l x1,2}{3} = \frac{0.8x1.2}{3} = 0.320MPa = 320kPa > 200kPa$$

In the upper 5.00 meters of the geotechnical profile composed of sandy material, a tip resistance of 8 MPa is to be reached to ensure a sufficient bearing capacity of 200kPa. A particular attention shall be paid at this after ground improvement by checking the resistance thanks to Post-CPTs down to 5.00 to 6.00 m (1.00 m below the first sandy layer).

4.3.4.6. Trial area. As part of the design a trial area was defined be carried out prior to the start of the production stage. This trial enabled to establish which amperage of the electric motor of the vibroflot must be reached during the compaction procedure in order to achieve the necessary gravel consumption in the various types of layers to obtain the required compaction and/or the required gravel inclusion factor.

During the compaction process, the vibroflot presses the gravel material horizontally against the in-situ soil. The resistance of the subsoil to the vibroflot can be measured by the intensity of the electric current required by the motor of the vibroflot. For given amperage of the motor, the stones are pushed farther against the softer soil layers than against the firmer layers. The trial area consisted of 9 treatment points implemented on a 2.70 m square grid spacing to assess these different parameters.

In order to verify the ground improvement efficiency in the upper 5.00 to 6.00 m, the sandy layer that provides the required bearing capacity, a Pre-CPT was set out at the exact location of the trial area before improvement and three Post-CPTs after the works. Figure 4.49 shows trial area proposed setting.

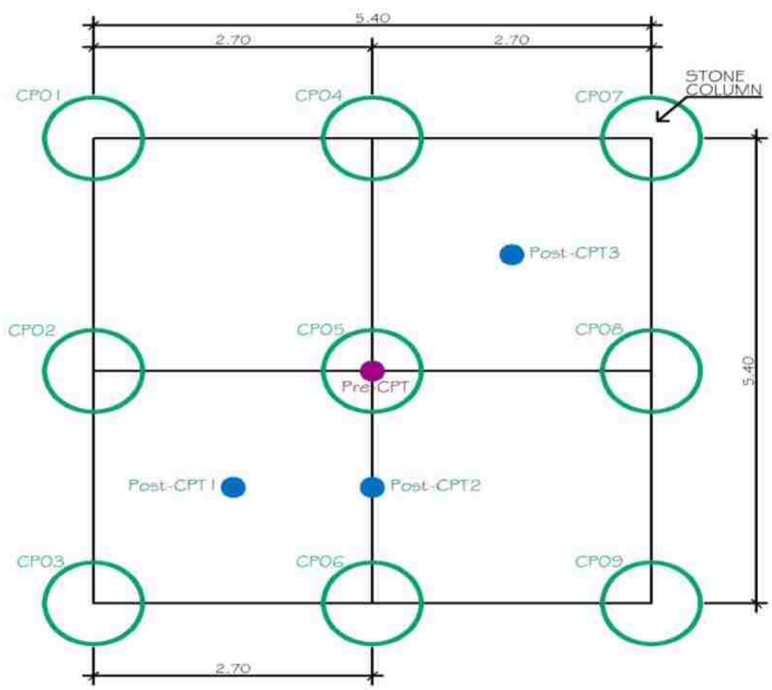


Figure 4.49. Trial area setting

CPTs location was determined based on:

Post-CPTs at the centre of grid spacing (“weakest-point”), Post-CPT 1 and 3.

Post-CPT at mid-distance between two compaction points (“mid-point”) Post-CPT2.

The depth of Pre and Post-CPT was limited to 1.00 m below the upper sandy gravel layer estimated to be 5.00 m thickness in average. Therefore, the total penetration depth is 6.00 m from the working platform elevation.

4.3.5. Construction. After analyze the ground conditions, the ground water level and mostly based in equipment availability, the wet top feed method was selected.

4.3.5.1. Equipment. The system is composed by five items, from which the most important equipment for the process is the Vibroprobe, which provides the vibration to the in-situ soils. Other items include service cranes from which the vibroprobe issuspended, generators to provide the electric power to the probes, air compressors to provide the air pressure, high pressure pumps to provide necessary water at high pressure to the vibroprobes, and service pumps to supply the water from water sources.

The essential parts of the Vibroprobe are shown on the Figure 4.50. It is essentially a long slender steel tube with two parts: the vibrator and the follow-up tubes. The vibrator, the heart of the Vibratory Probe, consists of a hollow cylindrical body with 300-400mm diameter connected by means of a special elastic coupling to the follow-up tubes of a slightly smaller outside diameter. The characteristics of the V23 vibroprobe are summarized in Table 4.17.

Table 4.17. Vibratory probe characteristics

Vibroflot Denomination	V23
Length (m)	3.57
Diameter (mm)	350
Weight (kg)	2,200
Motor (kW)	130
Speed (min^{-1})	1,800
Amplitude. (mm)	23
Dynamic Force (kN)	300

The vibroflotation technique was carried out using the equipment bullet listed herein:

- A vibroprobe Vibroflotation V23.
- A 80 tones crane with a 35.00 m long boom, Link Belt 138. The crane verification format is presented in the Figure 4.51.
- A vibroprobe with a Vibroflotation V23 vibrator and follow-up tubes (total length: 28m) to reach a maximum penetration depth of 25.00 m.
- A 300 kVA (440V) electric generator, Caterpillar.
- A high pressure jetting pump, Landini, ($100m^3/h$ under 1.2MPa).
- An air compressor, Ingersoll Rand, (21,000 l/min)
- A digital parameter recorder Jean Lutz LT3n.
- A wheel loader John Deere 544H with a 2.30 m3 bucket capacity.

Figure 4.50. V23, Vibratroy probe schematic

GUATEMALA - PUERTO QUETZAL GENOSA

Equipement list and characteristics for vibrocompaction - Single probe

MAXIMUM REQUIRED DESIGN PENETRATION DEPTH [m]	20

No.	Type	Equipment	Unit length [m]	Unit weight [kg]	Length [m]	Weight [ton]
1	V23	Vibroflot	3.30	2 200	3.30	2.20
1	VR1-30	Cable coupling	1.10	380	1.10	0.38
1	RK2b	Lifting head	1.30	430	1.30	0.43
4	VR 5.75 (50mm thickness)	Follow up tube	5.75	1 800	23.00	7.20

PENETRATION AND BOOM LENGTH PARAMETERS	
Complete length (a) [m]	28.70
Complete length below lifting head [m]	27.40
Useless height of the last follow-up tube (b) [m]	2.00
Maximum allowable penetration depth [m]	**25.40**
Standard boom or hammer head type boom	standard boom
Safety additional height over the lifting head (c) [m]	6.00
RECOMMENDED MINIMUM CRANE BOOM LENGTH - 70° ANGLE [m]	35

WEIGHT CHARACTERISTICS		
Complete weight of the single unit [ton]		10.2
Weight per linear meters [ton/m]		0.4
Maximum designed penetration depth [m] and corresponding weight the friction is based on [ton]	20.00	7.1
Maximum friction along the vibroprobe - Parts in the ground [%]	50	
MAXIMUM WEIGHT CONSIDERING FRICTION ALONG THE VIBROPROBE LENGTH IN THE GROUND [ton]		13.8

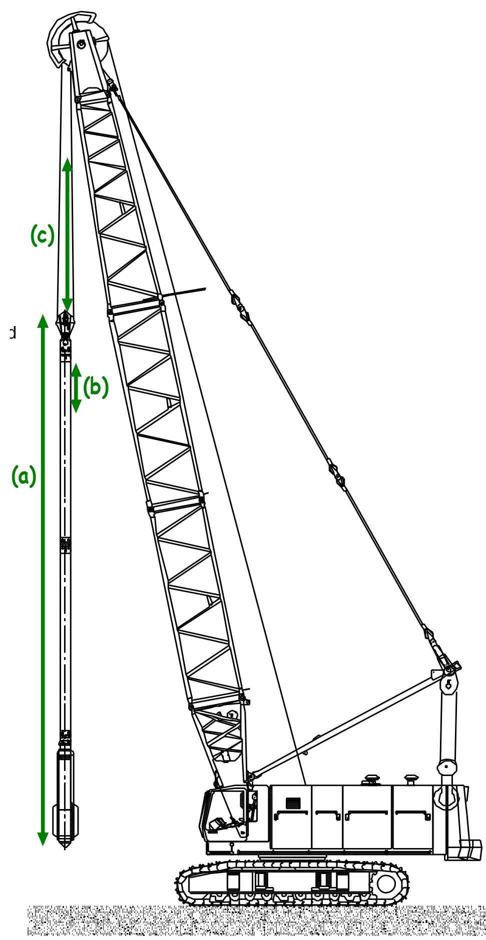


Figure 4.51. Crane verification format

4.3.5.2. Installation methodology – stones columns: wet top feed method.

The first step is prepare the working platform. This is very important in order to avoid accidents and allow a proper drainage of the area. The platform and accesses was compacted, drained and levelled in order to accommodate a crawler crane as well as the traffic of loaders required for stone delivery. The spoils and water generated by the stone column installation will be managed to maintain a clean working platform at all times. The working platform elevation was set at +0.50 m, at least 1.50 m above ground water elevation. The Figure 4.52 shows the work platform prior work start.

Figure 4.52. Working platform construction

The working methodology can be described in 3 main steps. Suspended from a crane or other supporting device, the penetration unit is positioned above the selected treatment point.

1 - The vibroflot is penetrated to the required depth under the combined effects of its own weight, vibration and the jetting action of air, water or both. Once the vibroflot has reached the required depth, a free and clean annular space around the vibroflot was created by a succession (usually one or two) of "washing" operations consisting in lifting the vibroprobe close to the ground surface and lowering it quickly back into the ground to

the maximum depth. When the vibroflot has reached again the full depth, the amount of water discharged from the tip of the vibroflot was adjusted so that the water level in the hole stays at about 1.50 m above ground water table or at working platform level. This ensures the stability of the hole. The figure 4.53 present de vibroflot penetration using water and air jet.

Penetration is stopped at the design depth or upon refusal on very dense sandy layers or very stiff clayey layers or big elements. Refusal was defined in the following way: the intensity in the electric motor of the vibroflot reaches values in excess of 200/250 A and/or penetration becomes slower than 0.50 m per minute, whichever comes first.

Figure 4.53. Vibroflot penetration

2 - The vibroflot is then lifted by 0.50 to 1.00 m, and coarse gravel or crushed stone is tipped into the hole. The vibroflot is then either re-penetrated to within a short distance of the previously penetrated depth or held at the current depth until the amperage is reached, which is sufficient to produce the specified average column diameter or the required ground compaction. Radial forces produced by the vibrator force the added material horizontally out against the in situ soil, thereby compacting it. The process is repeated until the required volume of stone has been inserted or the required degree of compaction has been achieved. The Figure 4.54 show gravel filling and compaction process.

Figure 4.54. Gravel filling and compaction

3 - The filling / compaction cycle is then repeated in upward increments up to the working platform level or up to the upper level of the soft silty ground layers or lenses. During this operation, additional gravel or stone is added to the hole but without overfilling to avoid bridging of the gravel. In this manner dense granular material

interlocks with the surrounding ground and densifies it. The Figure 4.55 show final stage of the compaction process. Works were performed between 15th of January and 4th of February of 2012.

Figure 4.55. Final stage of the compaction process

4.3.5.3. Material supply. Gravel or crushed stone consisted of elements within the range 20-60 mm of crushed stone with no more than 2% of material out of range. The rate of gravel supply is critical to ensure continuity of works for the rig deployed on site. In order to guarantee material supply and ease construction process gravel was stored in two different sites of the project.

4.3.5.4. As-built trial area. A trial area was carried out prior to the start of the production stage. The working procedure for the installation of stone columns is described above. The trial area location was chosen in order to implement the first stone columns to define the working method in ground conditions which are representative of the bulk of the works. The trial area was therefore set out at the location of Pre-CPT No.4 surrounding stack No.1.

The geotechnical description that can be drawn from the CPT data is the following: 20 meters of alternating beds of medium dense to very dense sandy layers and soft to firm silty to clayey layers. The general plan view of the trial area in relation to the future buildings and structures to be constructed is shown on Figure 4.56.

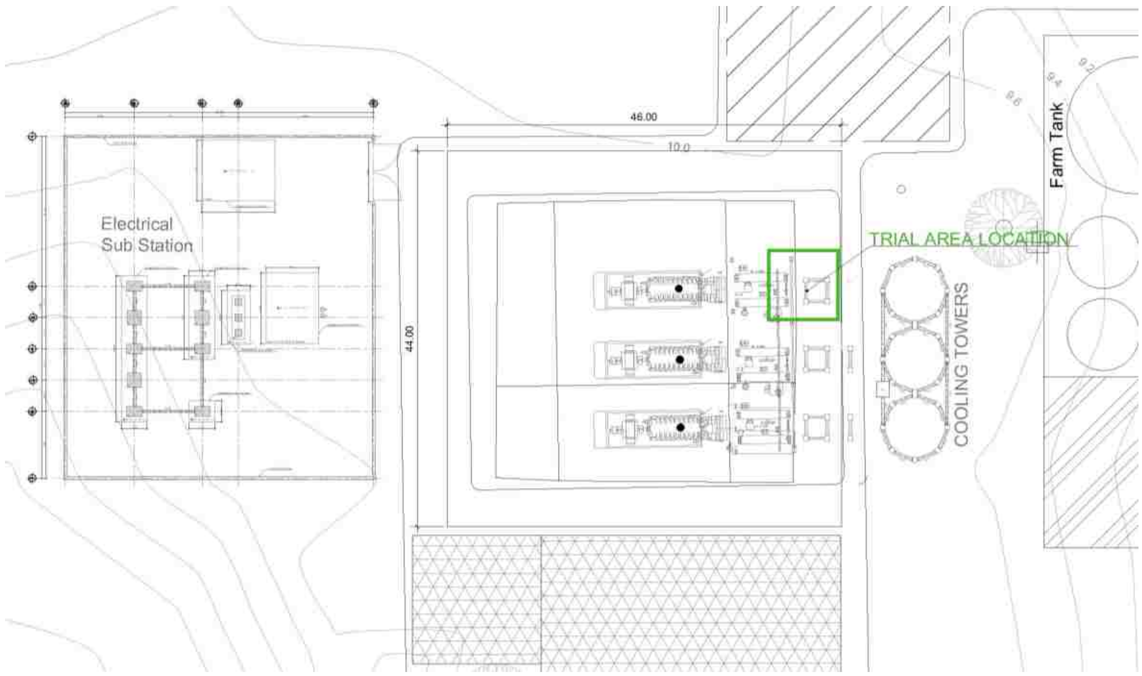


Figure 4.56. Trial area location

One recalls the main parameters of the design: 10% inclusion factor corresponding to a 2.70m square grid spacing with an average stone column diameter of 0.95m implemented on site as shown on the Figure 4.57. Grey dots, circled in red show the stone columns pattern in relation to the engines and the structures while the trial area is composed of the grey dots circled in green. The exact location of Pre-CPT no.4 is indicated as PreCPT TA and displayed as a red dot in between two compaction points in the southern part of the trial area.

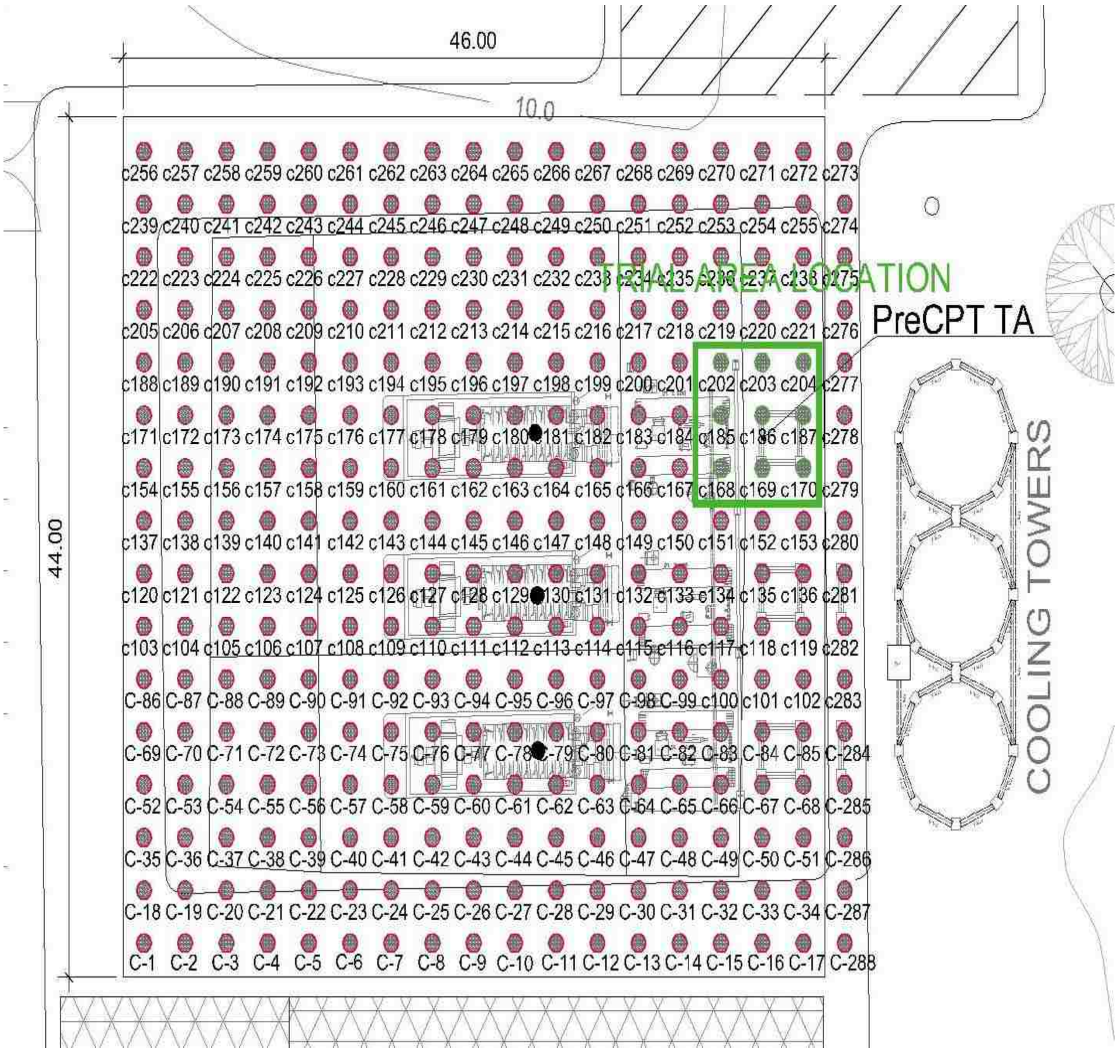


Figure 4.57. Ground improvement treatment

The stone columns were carried out considering this sequence: C170 to C168, C185 to C187 followed by C204 to C202 on the 20th and 21st of January of 2,011. Post-CPT were carried out on the 26th of January after a rest period of 4 to 5 days to ensure dissipation of excess pore pressure. Spoils were removed and the former working platform was scrapped (30cm) in order to show up the top of the stone columns, Figure 4.58 shows trial area after striping.

Figure 4.58. Trial area after striping

Initially, only 3 Post-CPTs were planned to be carried out as mentioned in the design section (Pre-CPTs 01 to 03) but due to early refusals found at shallow depth (1.00 to 2.00 m) for them, it was decided to immediately increase the number of tests to check that the very hard ground conditions of the first meters were homogeneous over the whole trial area, Figure 4.59 show Post CPT performance. Consequently, 2 additional post-CPTs were implemented, Post-CPT 04 and 05 at the weakest point locations as shown on Figure 4.60 and Figure 4.61 presents Post-CPT 04 profile.

Figure 4.59. Post-CPTs performance

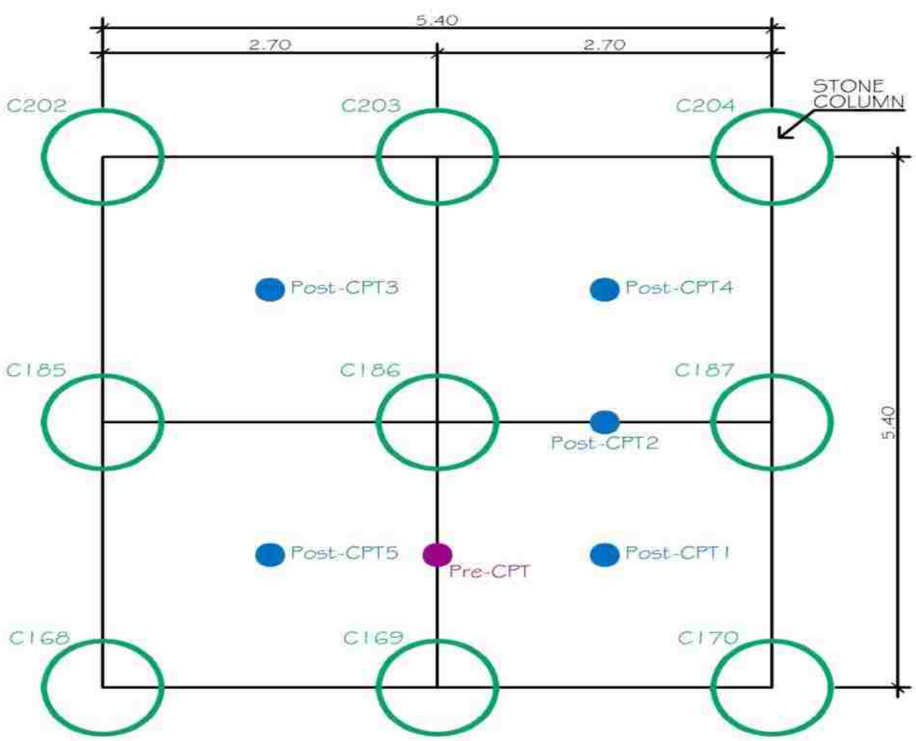


Figure 4.60. Trial area setting

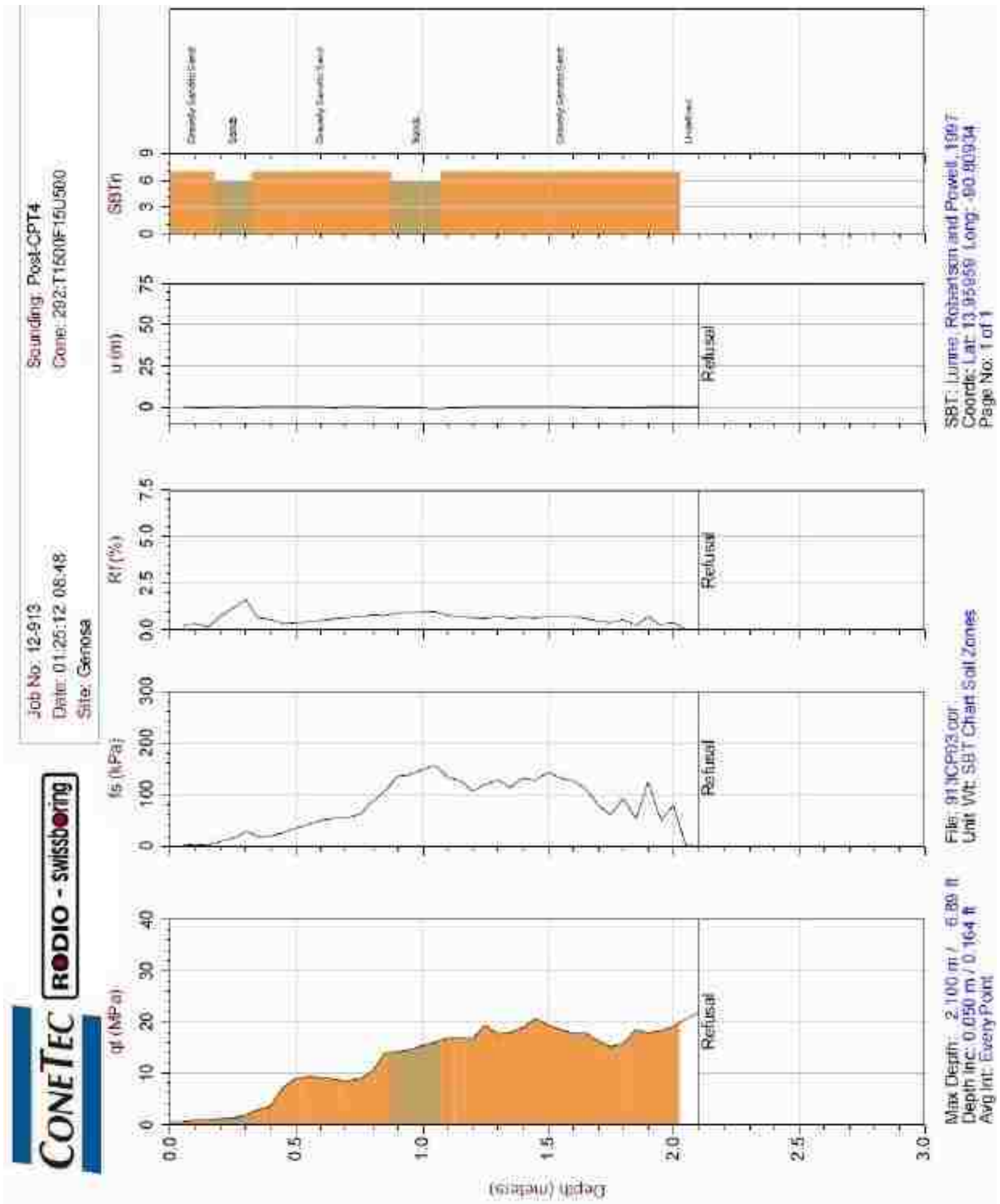


Figure 4.61. Profile of post-CPT 04

The following conclusion were drawn from the trial area:

The working methods which were defined during the trial area ensured a good quality of the stone column construction.

All Post-CPTs carried out within the trial area found refusal between 1.00 and 2.00m depth due to the very dense sandy material in the upper part of the profile regardless their location (weakest or mid-points).

Efficiency of ground improvement is thus checked thanks to in-situ means.

Tip resistances range between 10 and 20MPa and are therefore in excess of the criterion of 8MPa requested to satisfy the bearing capacity of 200kPa.

4.3.5.5. Quality control. During the operations, series of quality control processes are being undertaken to control the installation of the treatment points.

Setting-out: The main grid points of the treatment layout drawings were set out using total station and then checked/confirmed by the Engineer. Each treatment point was then be set out from the main grid points by means of measuring tape and staked on the ground by a peg/stick.

Penetrated Depth: The depth of penetration was controlled and monitored by two means. Depth markers are welded every 0.50 m along the silo tube for visual control by the operation team and the supervisors. During the penetration and installation of stone columns, the depth of the tip of the penetration unit was automatically and instantaneously shown and recorded by a digital logger with printout records.

Amperage: A calibrated ampere meter was installed at the vibro-rig as a mean to control the amperage during the penetration and the stone column installation, Figure 4.62 shows Jean Lutz LT3N parameter record equipment and ampere meter. The amperage consumed during penetration and stone column installation is also logged and recorded automatically.

Volume of gravel: Each time a batch (loader bucket) of gravel is placed into the ground, it is digitally recorded by action of a push-button in the crane operator’s cabin. The number of gravel loads delivered to site shall also be recorded on a daily basis.

Stone column diameter: The average diameter of the stone column can be calculated using the actual total volume of gravel consumed and the actual column length. For every column, the digital continuous recording of the batch placement also allows to estimate the variation of column diameter with depth depending on the stiffness of the in-situ soil.

Figure 4.62. Jean Lutz LT3N parameter recorder and ampere meter

4.3.5.6. Reports and records. For the stone columns works, the Stone Column Daily Report was prepared based on the information recorded with the Jean Lutz LT3N parameter record equipment, Figure 4.63 show the Quality Control printouts obtained.

4.3.5.7. Construction anomaly. After the works were finished, during scraping and conformation of the power plant final platform two holes, similar to sand blows were found. The holes have a depth between 60 to 70 cm and a diameter 80 cm. The figure 4.64 presents images of both holes.

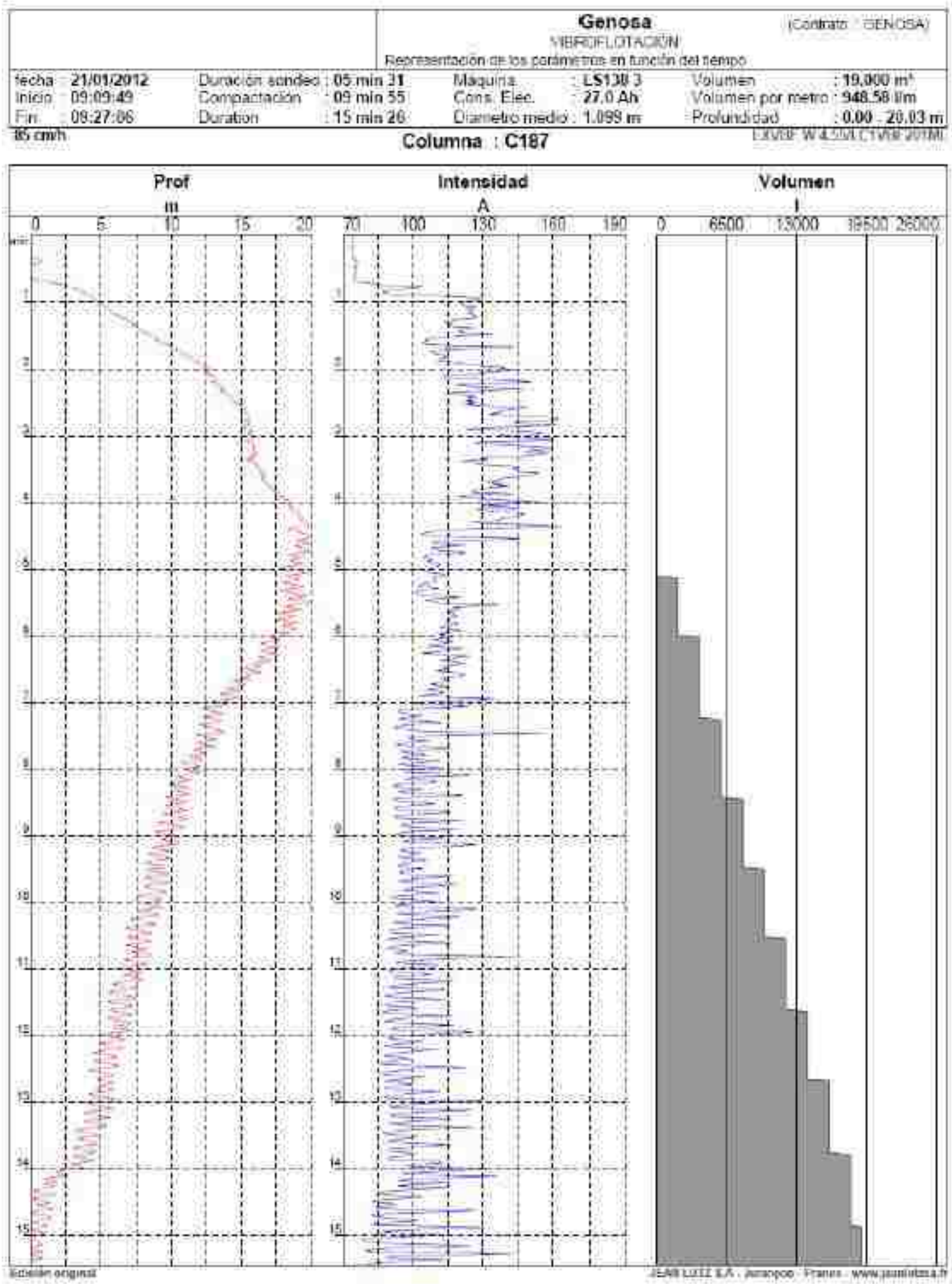


Figure 4.63. Quality Control Printout

Figure 4.64. Construction Annomalies

4.3.6. Monitoring and Performance. In order to verify and monitoring the long term performance of the solution 8 survey points were marked around the motors warehouse and an external reference was established using a survey monument. The points are shown in the Figure 4.65, dates of the readings are presented in Table 4.18.

Table 4.18. Readings program

Reading Number	Reading Date	Comment
0	10/10/2012	Reference reading
1	12/04/2012	After Earthquake reading
2	12/05/2013	
3	12/08/2014	
4	06/01/2015	

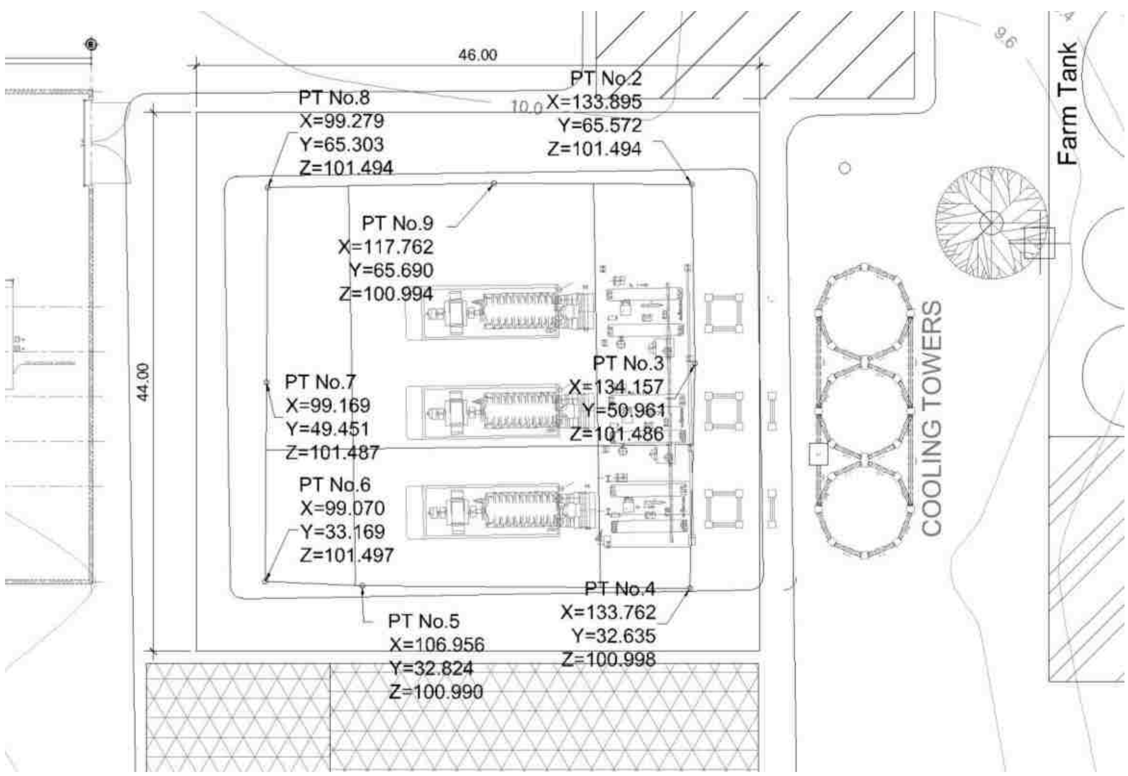


Figure 4.65. Survey Points Location

Reading were performed using Topcon DL 102 Electronic Digital Level and a Nikon DTM 322 Total Station, Table 4.19 and 4.20 presents instruments characteristics, Figure 4.66 shows the survey monument and surveying works.

Figure 4.66. Survey monument and surveying works

Table 4.19. Digital level characteristics

Manufactured by	Topcon
Model	DL-102C
Telescope	
Magnification	30×
Objective Aperture	45mm
Field of View	1°20′
Resolving power	3″
Compensator	
Working Range	±15′
Setting Accuracy	±0.5″
Height Measurement	
Accuracy (Standard deviation for 1km double-run levelling)	
Electronic Reading	1.0mm w/Fiberglass staff
Optical Reading	1.5mm
Least Count	1mm / 0.1mm

Table 4.20. Total station characteristics

Manufactured by	Trimble
Model	M1
Telescope	
Magnification	30×
Objective Aperture	45mm
Field of View	1°20′
Resolving power	3″
Compensator	
Setting Accuracy	±1″
Distance Measurement	
Accuracy (with single prism)	6.25 cm / 3,000 m
Measuring Interval (Precise mode)	1.8 sec.
Measuring Interval (Normal mode)	0.8 sec.
Least Count (Precise mode)	1 mm
Least Count (Normal mode)	10 mm
Angle Measurement	
Accuracy Horizontal Angles	2”
Accuracy Vertical Angles	5”

The maximum settlement recorded is 14 mm, Point 4,that is the 28% of the maximum settlement estimated, 50 mm. The corner points showed the greater settlements with a trend of settlement to SE. The middle points of the building showed the smaller settlements, Points 5 and 7. A maximum settlement difference of 12 mm between Point 4 and Point 5 is probably absorbed by the structure without perceptible signs of movement. The Figures 4.67 shows the graphic of the total settlement of each point and Figure 4.68 shows the graphic of the incremental settlement, both graphic shows a trend that no major settlements occurred after a the third reading, about a year after Power Plant operation beginning. Is important to mention that on 7th of November of 2012 an Earthquake of 7.4 of Moment of Magnitude occurred, the epicenter was located in the Pacific coast of Guatemala at about 70 km from the site. During this event liquefaction was observed in Champerico a seafront community as well as in San Pedro, San Marcos the largest town near to the epicenter.

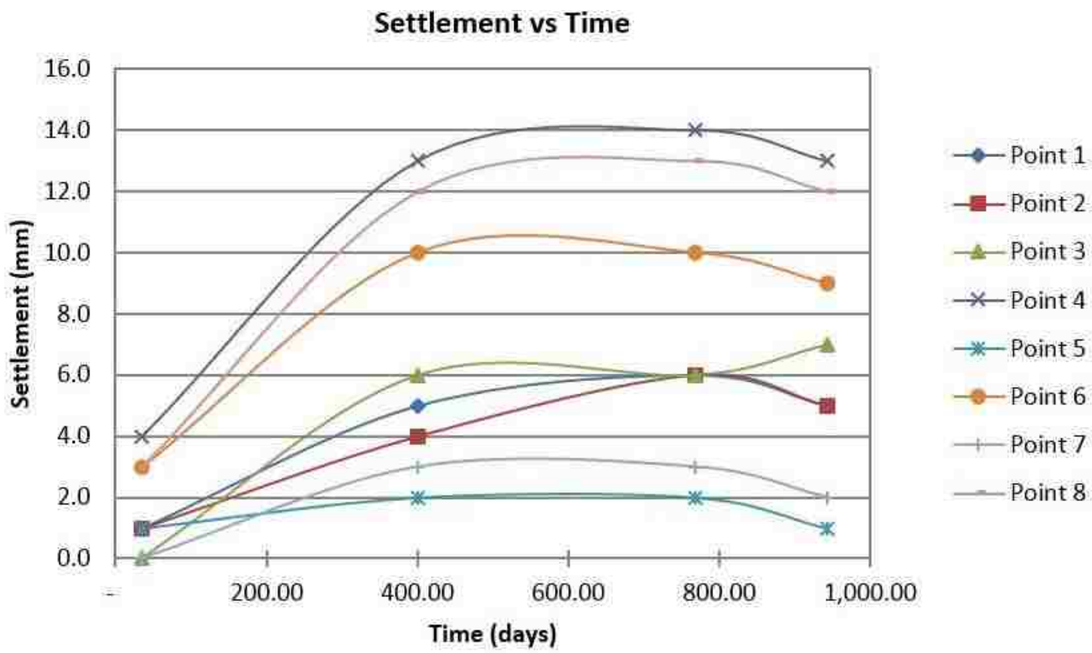


Figure 4.67. Graphic of settlement vs time

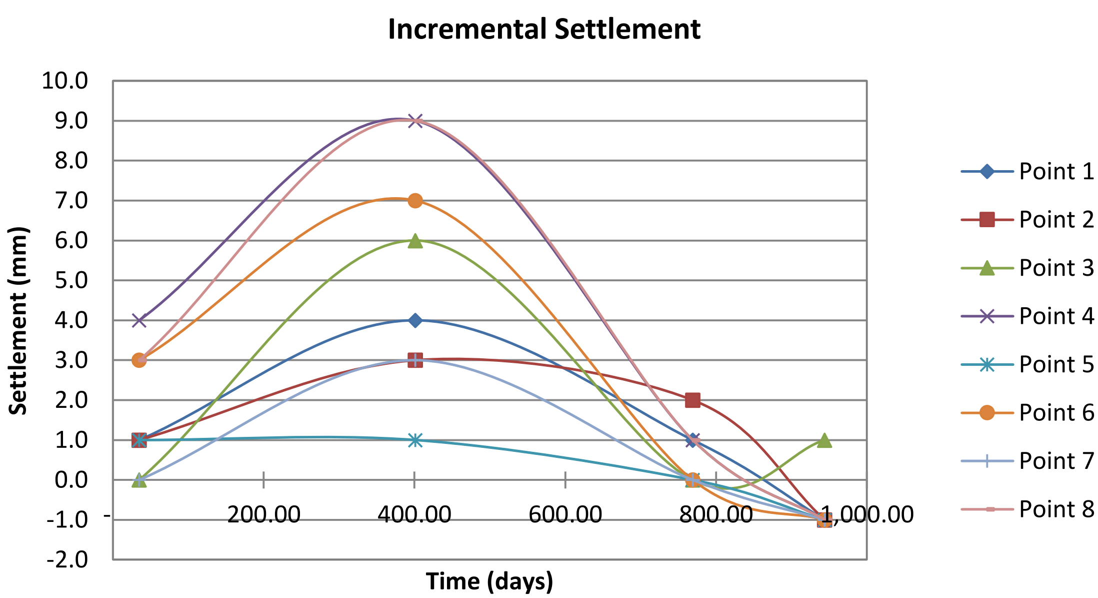


Figure 4.68. Graphic of incremental settlement

Despite that some settlements were recorded after the earthquake the largest settlements occurred after the initial operation period. In general the no perceptible settlements or movements are observed in the structure as well as no cracks or fissures.

The motors are very sensitive to tilt, particularly their axis, motors presents no sign of movement of misalignment.

4.3.7. Lessons Learned. The objective of a case study is to present a detailed description of a constructed project and also share experience and knowledge. Even the most successful projects have lessons to be learned, throughout a life cycle of a project different lessons can be learned and opportunities for improvement can be discovered.

Identifying and documenting lessons learned provides a mechanism that communicate acquired knowledge more effectively and ensure that beneficial information is factored into planning, work processes, and activities for other similar projects.

Analyze lessons learned provides an opportunity to discuss successes during the project, unintended outcomes, and recommendations for similar future projects. It also allows the discussion of things that might have been done differently, the root causes of problems that occurred, and ways to avoid those problems.

The major benefit of compile the lessons learned is retain and document both successful, best practices, and unsuccessful project activities for future reference. This allows new projects to repeat successful activities and to avoid those that were not successful.

The lessons learned are presented asking if this was the right solution, identifying if were there improvements that could have been made and closing with what were the success factors. This represent a walk through the most important aspects of the case study.

4.3.7.1. Was this the right solution? The critical issues for the project were liquefaction risk and ground settlement. Vibroreplacement, Stone Columns, bring solution to both issues, particularly to liquefaction risk. The performance of the solution in reference with other similar projects in the area is remarkable good with no perceptible signs of movement. The recorded settlements apparently stopped and reached only the 28% of the maximum settlement estimated initially. From this some savings could be done, this is addressed herein. Also and important matter for the project was the performance time, completing the whole area treatment in 20 days.

4.3.7.2. Were there improvements that could have been made? Analyzing the project in retrospective, several improvements can be done. Improvement suggestions are separated for each stage of the project.

4.3.7.2.1. <u>Geotechnical investigation.</u> As shown above an extensive geotechnical investigation campaign was performed. The most remarkable improvement could be perform additional boreholes and perform it earlier. The geotechnical campaign included a single borehole that was performed in order to obtain intact samples to estimate settlement. It was performed relative late, CPT survey was performed first. The additional boreholes could allow obtain intact samples at shallower and deeper locations, improving the information for the design. Also triaxial tests could be performed instead direct shear tests. Finally, a piezometer could be installed in the borehole performed allowing monitoring ground water level.

4.3.7.2.2. <u>Design.</u> Design improvement can be separated in two areas, ground improvement design and load bearing determination. The relative small settlements probably indicates that inclusion factor was too large or that the contribution of the upper sandy gravel layer was underestimated. This can be observed since the verification tests, Post-CPTs, which presented refusal in very first meters. Probably the interbedding effect of the sandy layer allowed or improved the drainage during the pore pressure excess caused by compaction, this could also increase treatment effectiveness. This interbedding effect also could be took into account for settlement calculations, performing a more detailed calculation. For future projects in the area, assuming a similar geological setting, and after analyze the improvement reached the columns could be shorter reaching 15.00 to 16.00 m depth. This depth reduction represents a 25% saving. The bearing capacity could be performed in a more detailed manner, by means taking into account the contribution of the upper sandy gravel layer and load influence factor. Probably bearing capacity was not critical for the motor foundation based on the fact that the motors requires a minimum foundation dimensions. But bearing load improvement could be fully used in the auxiliary structures. A good combination between a more detailed calculation of settlements and the experience obtained in this project will lead to a sensible improvement for future projects.

4.3.7.2.3. <u>Construction.</u> The construction stage was achieved in very satisfactory manner. The main improvement that could be recommend is referred to the gravel supply. Despite that from the begin gravel supply was identified as critical and in almost all the project was very efficient in some stretches the project ran out of material, this could also attributed to the high production rates. The verification using CPT could be improved performing pre drills in order to pass hard upper layer and then assess underlying ground. Other improvement that could be implement is the addition to the vibro-rig of a cabin or safe stage for the pump (water jet) operator. This could be replaced by an integrated control of the vibropobe and the pump.

4.3.7.3. Instrumentation. Instrumentation program could be improved in two ways, instrumentation selection and measurements frequency. Instrumentation selection could include a settlement plate installed prior poring the foundation of the structures. Other alternative could be install a magnetic settlement system consisting of magnetic rings installed within the ground and the foundation. Also the location of the instrumentation could be improved be means install interior points near or over motor foundations. The alternative instrumentation installation requires coordination between the geotechnical contractor and the general contractor, this could be eased by the project manager or supervisor and have to be addressed prior works start. This alternative and additional instrumentation will provide additional information about solution performance particularly for the most sensitive area, motors area, which could be used as early alert. The monitoring program include yearly measurements, in order to assess in a most accurate way the performance of the structure measurement frequency should be shorter. The reading frequency could be incremental from monthly during the first 3 months then each 2 months for four months then two more measurements each six months ending in yearly readings.

4.3.7.4. Success factors of the project. As improvements are addressed also success factors have to be highlighted. The initial success factor is knowledge about site geology and geologic hazards. Based in this knowledge CPT was selected as part of the geotechnical investigation program. The CPT possess the characteristic of reduce human uncertainty during its performance contrary to Standard Penetration Test, SPT, that is the most common alternative available in the area. The CPT exploration produced high quality information that was easily used for liquefaction assessment as well as for ground improvement design.

Have the advice and support of a ground improvement expert as Vibroflotation Group eased considerably the project progress during every stage, from the solution determination, passing through the design to ending in the construction. This expertise contributed to have and analyze alternatives for design and construction in a rapid manner, also helped in a solid planning that transformed in a successful execution.

The construction was carried out in a rapid manner mostly due to the experienced personnel and very strong planning. As mentioned above material supply was critical, this supply was very efficient during almost all the construction.

The trial area allowed design assumption verification but also provided an opportunity to validate construction procedures, equipment function ability and production rates.

5. CONCLUSIONS

This study covered the geotechnical state-of-the-art in Guatemala including its history, educational, professional, and contractor resources and capabilities. It also covers the practice and performance of actual projects and techniques and the learned lessons.

The history of Guatemalan modern geotechnical engineering practice is relatively short with a traceable time frame of about 60 years. The most relevant events in this brief history are the incorporation of geotechnical professionals trained abroad, major construction projects, foreign contractors, and natural disaster, particularly the 1976 earthquake.

The incorporation of geotechnical trained professionals represented an upgrade to the practice including new analysis techniques, design, and execution. The most relevant engineers were Ing. Armando Lopez, Dr. Rodolfo Semrau, and Ing. Federico Koose.

The construction projects brought opportunities for new techniques and execution, such as the hydro-power projects, which are the most relevant for geotechnical engineering and also large building projects. Construction projects also brought the need for specialized contractors, which Swissboring Overseas is by far the most relevant. Other international contractors allowed technology transfer to geotechnical practice.

Guatemala is particularly vulnerable to natural disasters due to its particular geotechnical and geologic setting, as well as its climate. The 1976 earthquake was by far the most relevant natural disaster, which highlighted the need to improve the construction and engineering practice. Other major events have been the tropical depression and hurricanes, such as hurricane Mitch, that induce ground instabilities upon the associated heavy rainfall. Major landslides and flooding have caused infrastucture damage and loss of life. These events have raised awareness of the risks associated with geotechnical issues and also resulted on improvements in procedures for subsurface investigation and geotechnical engineering practice.

The educational resources and capabilities are presented for undergraduate and graduate programs. The civil engineering programs include mandatory courses in soil mechanics theory, laboratory practice, and foundation engineering. The geological sciences are the logical complement to geotechnical engineering, however this program is located in, Coban, Alta Verapaz. Coban is a relative small town located about 4 hours from Guatemala City, which limits the student population and the interest in the career. Even though in the last decade a graduate MS program in geotechnics was launched at USAC, its enrollment and graduate completion rate have been low. Recent administrative disruptions have impacted the students' ability to graduate.

Laboratory resources can be divided in two types: (1) university laboratories, and (2) commercial laboratories. The average commercial soil laboratory has limited resources, the university laboratories are better equipped than the average commercial laboratory. The better equipped laboratories are at the Universidad de San Carlos de Guatemala (USAC), Oficina de Ingeniería de Guatemala (OIG), Dr. Rodolfo Semrau Laboratory, and Universidad del Valle de Guatemala (UVG).

Historically, there have been a Society of Guatemalan Geologists and the Colegio de Ingenieros (CIG), but nothing for geotechnical engineers. The Guatemalan Society of Soil Mechanics and Geotechnical Engineering (AMSIG) is the most recent geotechnical professional society and its membership includes the majority of geotechnical practitioners. This was a significant step forward for the geotechnical community, but it came with great challenge to bring opportunities for professional development, improvement to the practice, published resources, and its general ability to remain active.

Geotechnical construction solutions are directly related to the contractors capabilities. The contractors can be divided in two categories: (1) in-situ testing and field investigation and (2) geotechnical construction. The most common in-situ and geophysical investigation tests and techniques are available in Guatemala, but the individuals to conduct this work are limited. The contractor with more capabilities by far is Rodio-Swissboring with almost all the capabilities for in situ testing and soil investigation.

In summary, Guatemala continues to struggle in geotechnical engineering, with most of its contemporary capabilities available to major projects that can afford the specialized engineers and contractors. The majority of the projects are carried out with limited to no subsurface investigations and minimal geotechnical engineering input.

A case study presents a detailed description of an engineering project, including the most relevant information that lead to the performance of the solution adopted. Case studies are very important for the progress of the practice and an excellent opportunity to share experience and knowledge. Three case studies were selected for this study based on the overall topic of ground stabilization and the access to project data. It is important to mention that access to data was requested from the owners and engineers, and all three projects are more of a showcase of success stories and not a case history of a unique failure. There is a culture of covert the mistakes and limiting access to previous mistakes or failures, to avoid professional shame or blame. Therefore, the lessons learned are not from the point of view of forensic or learning from failures.

The soil nailing project constructed in the volcanic soils of Guatemala has a very good performance resulting in small long-term displacements (less than 25 mm). The application of a soil nailing "sandwich" in the soil wedge area had a very good performance with small long-term displacements (less than 25 mm), and served as an alternative to retaining structures with complicated and/or constrained geometry. Volcanic piroclastic deposits are reasonably stable when cut near vertical and in the long-term they may experience instabilities when saturated or disturbed. However with the added reinforcement using the soil nailing technique increases stability and it has gained significant popularity in Guatemala.

The grout consumption into metamorphic formations of the Santa Teresa dam area was estimated to be 0.80 bags (42 kg bag) of cement per linear meter of drilling. This was in contrast to the initial assumption of 0.50 bag per linear meter. The 2% of the grouted length consumed the 71% of the total grouting volume. The GIN number stopped 70% of the high consumption stretches, as mentioned above the consumption of this stretches increased the 61.9% of the overall average consumption, so GIN stoppage implied a reduction in the overall consumption. Since the completion of the curtain wall and the start of operations in July 2011, no noticeable problems have been identified. In general, and using the words of Mynor Celis, Operations Manager of the owner of the Santa Teresa dam, the cut-off curtain performed satisfactorily with no inconveniences reported. The GIN technique was introduced to Guatemala in this project and resulted in very efficient outcome for the dam. Without this technique the expenses in grout and time to develop the cut-off curtain would have been much higher.

The maximum settlement recorded for the structure supported on the area treated with stone columns was about 14 mm, or about 28% of the maximum settlement predicted, 50 mm. The edges of the structure resulted in the greater settlements and a trend of more settlement towards the SE corner. The center of the building experienced the lowest magnitude of settlement, with a maximum differential settlement of 12 mm, which can be absorbed by the structure without perceptible sign of distress. An incremental settlement behavior was observed with time, the major settlement occurred after the third reading, about one year after the Power Plant began operating. The stone column ground stabilization solution performed remarkably well during the 2012 earthquake (M=7.4). No perceptible displacement or damage were observed. Only minor settlements were recorded after the earthquake the largest settlements occurred after the initial operation period.

The participation of foreign expertise (international companies) is transcendental for geotechnical engineering practice in Guatemala. Based on the three case studies presented in this manuscript, only the soil nailing project (Case Study #1, Sec. 2.4.1.1) could be performed without assistance from foreign experts.

All of these case studies had some level of performance monitoring during and/or after construction. Even though the techniques and the monitoring programs were not complex, the philosophy of measuring the outcome of a geotechnical construction made these projects unique in Guatemala. It is not known how efficient the designs were, since none of them experienced measurable signs of distress. It is the opinion of the author that there is a significant conservative assumptions in the design to assure adequate performance. Unfortunately, conservative design is only accessible to owners that can afford the extra expense. Many other constructed facilities and infrastructure do not enjoy these resources and cannot err in the side of caution, resulting in failures and underperformance. It is the obligation of the geotechnical community in Guatemala to document and report on failures and instill a philosophy of learning from failures through case histories. Otherwise, the lessons learned are limited and the understanding or the local ground conditions continue to be obscured by undocumented knowledge.

6. RECOMMENDATIONS

The recommendations presented herein apply to different stakeholders. In order to try to implement as much as possible these recommendations the first action is distribute this work among all the possible stakeholders. The information compiled varies from historic anecdotes to the consumption grout values and deformations for a wall cut-off wall curtain. All are useful for different applications in geotechnical engineering.

The compilation of the modern history of Guatemalan geotechnical engineering is a starting point for a continuing effort to record progress. This effort could be continued by the membership of AMSIG as part of its aim and with the author's leadership. AMSIG's role in the improvement of the practice is definitively relevant. In the short-term two quick wins for the society could be completing the agreement with CIG and start with seminars for professional development. The suggested topic for these seminars are in situ testing, laboratory testing (a certification could be proposed), and bearing capacity determination. Other opportunities are to publish case histories in the PanAmerican Geotech conference and the ISSMGE conference, where the number of delegates, topics, and paper allocation is in our favor.

The geotechnics MS program at eh USAC is an excellent platform to improve the educational resources in Guatemala. The addition of a soil dynamics related course, such as geotechnical earthquake engineering, could be a reasonable addition to the MS program curriculum. An alternative to be explored is to reinforce the faculty with more experienced and specialized professionals and scholars.

The case studies provide a reference of successful engineering solutions. As mentioned before, to communicate these case studies is relevant, this duty could be performed by the author or by AMSIG. The case studies could be used as a model of "best practices" that can be achieved reasonably in Guatemala. The lessons learned present improved areas that could be implemented not just for similar projects, but also to different geotechnical related projects. The final recommendation is to encourage owners and contractors to continue monitoring the performance of finished projects with collaboration of other engineers.

The recent 2015-16 changes in Guatemalan socio-political environment have brought new policies regarding the opening of government records to the public. This may present an opportunity to publish more freely the geotechnical case studies that before were not available for publication in the past culture environment.

APPENDIX

GEOTECHNICAL CONTRACTORS DIRECTORY

GEOPHYSICAL SURVEY CONTRACTORS

Geociencia Aplicada

Geophysical survey capabilities: Seismic Refraction, Multichannel Analysis Surface Wave (MASW), Electric Resistivity, Ground Penetrating Radar (GPR).

Telephone: 2339-3389

E-mail: jpligorria@gmail.com

Geocon

Geophysical survey capabilities: Seismic Refraction.

Address: 5ª. Av. 5-55 Zona 14, Euro Plaza, Torre 2 Nivel 2, Guatemala.

Telephone: 2328-8000

E-mail: ventas@precon.com.gt

Webpage: www.geocon.com.gt

Geopetrol

Geophysical survey capabilities: Seismic Refraction, Electrical Resistivity.

Address: 7a. Av. 14-44, Zona 9, Edificio La Galeria, Oficina 27, Nivel 1, Guatemala.

Telephone: 2360-3033

Webpage: www.geopetrolsa.com

Ingeotecnia

Geophysical survey capabilities: Electrical Resistivity.

Address: 10ª. Av. 2-96 zona 8 Mixco Pinares de San Cristóbal, Mixco.

Telephone: 2478-8678

E-mail: ca@ingeotecnia.net

Webpage: www.ingeotecnia.net

GEOTECHNICAL CONTRACTORS

Geocimsa

Geotechnical construction capabilities: Soil Nailing, Anchors, Grouting, Hand excavated piers, Drilled Shafts up to 600 mm, Shotcrete, Micropiles, Rock and Soil Drilling.

Address: 2a. calle "A" 11-67, Zona 15, Col. Tecún Umán, Guatemala.

Telephone: 2218-4343

E-mail: info@geocimsa.com

Webpage: www.geocimsa.com

Geocon

Geotechnical construction capabilities: Soil Nailing, Anchors, Grouting, Driven Piles, Mechanically Stabilized Walls, Geosynthetics, Shotcrete, Rock and Soil Drilling.

Address: 5ª. Av. 5-55 Zona 14, Euro Plaza, Torre 2 Nivel 2, Guatemala.

Telephone: 2328-8000

E-mail: ventas@precon.com.gt

Webpage: www.geocon.com.gt

Pilotecmar

Geotechnical construction capabilities: Soil Nailing, Anchors, Grouting, Driven Piles, Drilled Shafts larger than 1800 mm, Sheet piles, Micropiles, Shotcrete, Rock and Soil Drilling.

Address: 20 calle 5-36, Zona 10, Guatemala.

Telephone: 2382-2500

E-mail: info@pilotecmar.com

Webpage: www.pilotecmar.com

Prodecsa

Geotechnical construction capabilities: Soil Nailing, Anchors, Grouting, Hand excavated piers, Micropiles, Shotcrete, Rock and Soil Drilling.

Address: 1ª. calle 22-34, Zona 15, Vista Hermosa 2, Guatemala.

Telephone: 2484-7966

E-mail: gcastaneda@prodecsa.com.gt

Rodio-Swissboring

Geotechnical construction capabilities: Soil Nailing, Anchors, Grouting, Driven Piles, Drilled Shafts larger than 1800 mm, Driven Piles, Sheet piles, Micropiles, Shotcrete, Jet Grouting, Stone Columns (Vibroreplacement), Vibrocompaction, Wick drains, Diaphragm Walls, Rock and Soil Drilling.

Address: 12 calle 1-24, Zona 10, Edificio Casa Veranda 1er. nivel, Oficina 101, Guatemala.

Telephone: 2201-6600

E-mail: info.gt@rodio-swissboring.com

Webpage: www.rodio-swissboring.com

Soiltec

Geotechnical construction capabilities: Soil Nailing, Anchors, Grouting, Driven Piles, Drilled Shafts up to 1500 mm, Driven Piles, Sheet piles, Rammed Aggregate Piers, Micropiles, Shotcrete, Rock and Soil Drilling.

Address: 4ta. calle 21-21, Zona 14, Guatemala.

Telephone: 2366-2251

E-mail:

Webpage: www.soiltec.com.gt

Terrasol (STI)

Geotechnical construction capabilities: Soil Nailing, Anchors, Grouting, Driven Piles, Drilled Shafts up to 1500 mm, Micropiles, Shotcrete, Rock and Soil Drilling.

Address: 3a. calle "A" 8-68, Zona 10, Guatemala Ciudad

Telephone: 2201-2400

E-mail: info@sti.com.gt

Webpage: www. sti.com.gt

SITE INVESTIGATION AND CHARACTERIZATION CONTRACTORS

Dr. Rodolfo Semrau

Site investigation and characterization capabilities: Test pits, undisturbed sampling, Vane Shear Test.

Address: 11 Av. 18-42 Zona 11, Colonia Bosques de Mariscal, Guatemala.

Telephone: 2472-7255

E-mail: dr.rodolfosemrau@gmail.com

Geocon

Site investigation and characterization capabilities: Rock Drilling, Standard Penetration Test, (SPT).

Address: 5ª. Av. 5-55 Zona 14, Euro Plaza, Torre 2 Nivel 2, Guatemala.

Telephone: 2328-8000

E-mail: ventas@precon.com.gt

Webpage: www.geocon.com.gt

Geotecnica

Site investigation and characterization capabilities: Rock Drilling, Standard Penetration Test, (SPT), Plate Load Test.

Address: Km. 29.5 Carr. al Pacífico CA-9 Sector B, Lote 20 Lotificación "El Ceibillo", Amatitlán.

Telephone: 2331-9143

E-mail: ventas@geotecnica.com.gt

Webpage: www.geotecnica.com.gt

Grupo Phi

Site investigation and characterization capabilities: Rock Drilling, Standard Penetration Test, (SPT).

Address: 48 calle 22-76, Zona 12, Colonia Prados de Monte María, Guatemala.

Telephone: 2477-3783

E-mail: info@grupo-phi.com

Webpage: www.grupo-phi.com

Ingeotecnia

Site investigation and characterization capabilities: Standard Penetration Test, (SPT), Plate Load Test.

Address: 10ª. Av. 2-96 zona 8 Mixco Pinares de San Cristóbal, Mixco.

Telephone: 2478-8678

E-mail: ca@ingeotecnia.net

Webpage: www.ingeotecnia.net

Pala

Site investigation and characterization capabilities: Rock Drilling, Standard Penetration Test, (SPT), Plate Load Test.

Address: 36 Av. "A" 13-50, Zona 5, Jardínes de la Asunción Sur, Guatemala.

Telephone: 2336-1050

E-mail: info@corporacionpala.com

Webpage: www.corporacionpala.com

Rodio-Swissboring

Site investigation and characterization capabilities: Rock Drilling, Standard Penetration Test, (SPT), Cone Penetration Test, (CPT), Permeability Test, Pressuremeter, Vane Shear Test, Point Load Test, Plate Load Test.

Address: 12 calle 1-24, Zona 10, Edificio Casa Veranda 1er. nivel, Oficina 101, Guatemala.

Telephone: 2201-6600

E-mail: info.gt@rodio-swissboring.com

Webpage: www.rodio-swissboring.com

Servicios Unficados de Ingeniería

Site investigation and characterization capabilities: Standard Penetration Test, (SPT), Plate Load Test.

Telephone: 5523-4590

E-mail: suisa2020@gmail.com

Webpage: www.suisa.com.gt

Suelos y Cimentaciones

Site investigation and characterization capabilities: Rock Drilling, Standard Penetration Test, (SPT), Plate Load Test.

Address: 20 Avenida A 8-26 Zona 11, Guatemala.

Telephone: 2460-0556

E-mail: suelos.cimentaciones@yahoo.com

Webpage: www.suelosycimentaciones.com

SOIL LABORATORIES

Dr. Rodolfo Semrau

Testing capabilities: Grain Size Distribution, Liquid / Plastic Limit, Triaxial Cell, Oedemeter / Consolidation, Unconfined Compression, California Bearing Ratio, (CBR), Compaction Test, (Proctor).

Address: 11 Av. 18-42 Zona 11, Colonia Bosques de Mariscal, Guatemala.

Telephone: 2472-7255

E-mail: dr.rodolfosemrau@gmail.com

Geo Estudios

Testing capabilities: Grain Size Distribution, Liquid / Plastic Limit, Direct Shear, Unconfined Compression, California Bearing Ratio, (CBR), Compaction Test, (Proctor).

Address: 8ª. calle 1-69, Zona 1, Guatemala.

Telephone: 2232-8297

E-mail: wilmadeleon@hotmail.com

Geocon

Testing capabilities: Grain Size Distribution, Liquid / Plastic Limit, Direct Shear, Unconfined Compression, California Bearing Ratio, (CBR), Compaction Test, (Proctor).

Address: 5ª. Av. 5-55 Zona 14, Euro Plaza, Torre 2 Nivel 2, Guatemala.

Telephone: 2328-8000

E-mail: ventas@precon.com.gt

Webpage: www.geocon.com.gt

Geotecnica

Testing capabilities: Grain Size Distribution, Liquid / Plastic Limit, Triaxial Cell, Unconfined Compression, California Bearing Ratio, (CBR), Compaction Test, (Proctor).

Address: Km. 29.5 Carr al Pacífico CA-9 Sector B, Lote 20, Amatitlán.

Telephone: 2331-9143

E-mail: ventas@geotecnica.com.gt

Webpage: www.geotecnica.com.gt

Grupo Phi

Testing capabilities: Grain Size Distribution, Liquid / Plastic Limit, Unconfined Compression, Compaction Test, (Proctor).

Address: 48 calle 22-76, Zona 12, Colonia Prados de Monte María, Guatemala.

Telephone: 2477-3783

E-mail: info@grupo-phi.com

Webpage: www.grupo-phi.com

Ingeotecnia

Testing capabilities: Grain Size Distribution, Liquid / Plastic Limit, Unconfined Compression, Compaction Test, (Proctor).

Address: 10ª. Av. 2-96 zona 8 Mixco Pinares de San Cristóbal, Mixco.

Telephone: 2478-8678

E-mail: ca@ingeotecnia.net

Webpage: www.ingeotecnia.net

Mecánica de Suelos

Testing capabilities: Grain Size Distribution, Liquid / Plastic Limit, Triaxial Cell, Oedemeter / Consolidation, Unconfined Compression, California Bearing Ratio, (CBR), Compaction Test, (Proctor).

Address: 3ª. Av. 6-75 Zona 8, Mixco.

Telephone: 2478-3133

E-mail: mecanicade.suelos@yahoo.com

Webpage: www.mecanicadesuelosgt.com

Mecánica de Suelos y Pavimentos (Mecsypasa)

Testing capabilities: Grain Size Distribution, Liquid / Plastic Limit, Triaxial Cell, Oedemeter / Consolidation, Unconfined Compression, California Bearing Ratio, (CBR), Compaction Test, (Proctor).

Address: 12 Av. A 0-47, Zona 7, Col Quinta Samayoa, Guatemala.

Telephone: 2471 1743

E-mail: info@mecsypasa.com

Webpage: www.mecsypasa.com

Oficina de Ingeniería y Geotecnia (OIG)

Testing capabilities: Grain Size Distribution, Liquid / Plastic Limit, Triaxial Cell, Oedemeter / Consolidation, Unconfined Compression, California Bearing Ratio, (CBR), Compaction Test, (Proctor).

Address: 3 calle 9-80, Zona 15, Colonia Trinidad, Guatemala.

Telephone: 2369-1771

E-mail: hector.oig@gmail.com

Pala

Testing capabilities: Grain Size Distribution, Liquid / Plastic Limit, Unconfined Compression, California Bearing Ratio, (CBR), Compaction Test, (Proctor).

Address: 36 Av. "A" 13-50, Zona 5, Jardínes de la Asunción Sur, Guatemala.

Telephone: 2336-1050

E-mail: info@corporacionpala.com

Webpage: www.corporacionpala.com

Servicio de Ingeniería El Pilar

Testing capabilities: Grain Size Distribution, Liquid / Plastic Limit, Direct Shear, Unconfined Compression, California Bearing Ratio, (CBR), Compaction Test, (Proctor).

Address: Carretera a San Marcos, Contiguo a Gasolinera Texaco La Esperanza Quetzaltenango.

Telephone: 7772-0817

Servicios Unificados de Ingeniería

Testing capabilities: Grain Size Distribution, Liquid / Plastic Limit, Direct Shear, Unconfined Compression, California Bearing Ratio, (CBR), Compaction Test, (Proctor).

Telephone: 5523-4590

E-mail: suisa2020@gmail.com

Webpage: www.suisa.com.gt

Suelos y Cimentaciones

Testing capabilities: Grain Size Distribution, Liquid / Plastic Limit, Direct Shear, Unconfined Compression, California Bearing Ratio, (CBR), Compaction Test, (Proctor).

Address: 20 Av. “A” 8-26 Zona 11, Guatemala.

Telephone: 2460-0556

E-mail: suelos.cimentaciones@yahoo.com

Webpage: www.suelosycimentaciones.com

GEO-PROFESSIONALS

Ing. Sergio Aycinena

Field of expertise: Geotechnical engineering, drilling and construction.

Address: 2a. calle "A" 11-67, Zona 15, Col. Tecún Umán, Guatemala.

Telephone: 2218-4343

E-mail: info@geocimsa.com

Ing. Wilma de Leon

Field of expertise: Site investigation, geotechnical reporting and geotechnical design.

Address: 8ª. calle 1-69, Zona 1, Guatemala.

Telephone: 2232-8297

E-mail: wilmadeleon@hotmail.com

Ing. Ernesto Calzia

Field of expertise: Geotechnical engineering, geology, mining, drilling and construction.

Address: 12 calle 1-24, Zona 10, Edificio Casa Veranda 1er. nivel, Oficina 101, Guatemala.

Telephone: 2201-6600

E-mail: ecalzia@rodio-swissboring.com

Dr. Juan Pablo Ligorría

Field of expertise: Geophysical survey.

Telephone: 2339-3389

E-mail: jpligorria@gmail.com

Ing. Luis Leiva

Field of expertise: Site investigation, geotechnical reporting and geotechnical design.

Address: 20 Avenida A 8-26 Zona 11, Guatemala.

Telephone: 2460-0556

E-mail: suelos.cimentaciones@yahoo.com

Ing. Hector Paiz

Field of expertise: Site investigation and geotechnical reporting.

Address: 36 Av. "A" 13-50, Zona 5, Jardínes de la Asunción Sur, Guatemala.

Telephone: 2336-1050

E-mail: info@corporacionpala.com

Ing. Carlos Perez

Field of expertise: Geology.

Address: 10ª. Av. 2-96 zona 8 Mixco Pinares de San Cristóbal, Mixco.

Telephone: 2478-8678

E-mail: ca@ingeotecnia.net

Ing. Ricardo Rodas

Field of expertise: Geotechnical engineering design, drilling and construction.

Address: 3a. calle "A" 8-68, Zona 10, Guatemala Ciudad

Telephone: 2201-2400

E-mail: rrodas@sti.com.gt

Ing. Rodolfo Rosales

Field of expertise: Site investigation, geotechnical reporting and geotechnical design.

Telephone: 5523-4590

E-mail: suisa2020@gmail.com

Ing. Federico Rosenberg

Field of expertise: Geotechnical engineering, geology, mining, drilling and construction.

Address: 12 calle 1-24, Zona 10, Edificio Casa Veranda 1er. nivel, Oficina 101, Guatemala.

Telephone: 2201-6600

E-mail: frosenberg@rodio-swissboring.com

Dr. Rodolfo Semrau

Field of expertise: Site investigation, geotechnical reporting and geotechnical design.

Address: 11 Av. 18-42 Zona 11, Colonia Bosques de Mariscal, Guatemala.

Telephone: 2472-7255

E-mail: dr.rodolfosemrau@gmail.com

Ing. Hector Valdez

Field of expertise: Site investigation, geotechnical reporting and geotechnical design.

Address: 3 calle 9-80, Zona 15, Colonia Trinidad, Guatemala.

Telephone: 2369-1771

E-mail: hector.oig@gmail.com

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VITA

Fernando Rafael Callejas Benitez was born in Guatemala City, Guatemala on April 3, 1978. In October 18 2000, he received his B.S. in Civil Engineering from the Universidad del Valle del Guatemala, Guatemala City, Guatemala. In May 2002, he received his M.S. degree in Structural Engineering from the Universidad del Valle del Guatemala, Guatemala City, Guatemala. In May 2016, he received his D.E. in Geotechnical Engineering from the Missouri University of Science and Technology, Rolla, Missouri, USA.

He started to work for Swissboring Overseas Corp. Ltd. in February 18, 2000 as draftsman then promoted to design engineer in 2001 then promoted to project manager in 2002. In 2003 he was sent to Spain to receive training in geotechnical engineering working for Rodio Cimentaciones Especiales for eight months at the same time he received certificates in Ground Improvement and Rock Mechanics from CEDEX, Madrid, Spain. In 2005 he was promoted to Technical Director for Central America working for Rodio-Swissboring. In 2013 he joined Multiproyectos as Director of Engineering and Construction, position that he held until the present.

During the period of 2004 to 2010 he was instructor of Soil Mechanics II course in Universidad del Valle de Guatemala, in 2004 he was instructor of Advanced Soil mechanics in Mariano Galvez University, in 2012 he was instructor of Foundations course in Universidad del Valle de Guatemala.

He has published paper about earthquake engineering and geotechnical site characterization recommendations as co-author. In 2011, he won the “La Excelencia” Award in the infrastructure category of the Guatemalan Construction Chamber for the project “Diagonal 6 – Proceres Underpass”.

Fernando Rafael Callejas Benitez has been a member of the Guatemalan Engineers Society (CIG) since 2001, Guatemalan Society of Structural and Seismic Engineering (AGIES) since 2003, American Concrete Institute (ACI) since 2006, Deep Foundations Institute (DFI) since 2011. He has been President of Guatemalan Society for Soil Mechanics and Geotechnical Engineering (AMSIG) from 2010 to 2014.