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	PAGES	67
	AREA	GEOTECHNICS

PROJECT: GEOTECHNICAL ASSESSMENT REPORT FOR PV PARK IN LA TORUNEY (SAINT LUCIA)

REVISION CONTROL

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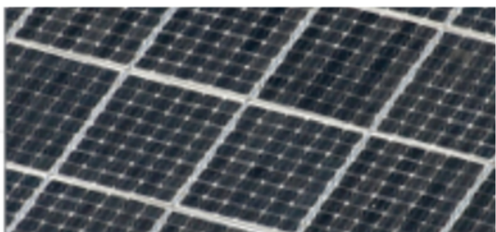
Geotechnical Assessment Report

Survey Site: La Tourney (Saint Lucia)
REF: IG62717

Geotechnical Surveying and Soil Testing for Solar Projects



1. CHAPTER I. EXECUTIVE SUMMARY

1.1. Site and project information

Survey site comprises 6.00 Ha farm, used as a pasture land at the time of undertaking this survey. The project consists PV panels and ancillary buildings as informed by the Client. Development also includes road tracks and pathways, enabling works and miscellaneous construction activities including electrical piping.

1.2. Local geology

La Tourney site area is located at the Saint Lucia SW area, in Vieux Fort district and close to Henaworra International airport.

The area is formed by alluvial quaternary materials that cover the existing volcanic bedrock only in part of the area. In the study area are composed mainly by modern accumulations of silty sands. These usually cover at a different depths volcanic bedrock.

Main geomechanical features are described ahead in the report and are summarized as follow:

- Uppermost Topsoil layer (undifferentiated) deposits leads to quite homogeneous geomechanical responses to dynamic probing.

- Uppermost Quaternary layer (undifferentiated) deposits leads to quite heterogeneous geomechanical responses to dynamic probing.

- Volcanic Weathered bedrock appears below sands o directly below topsoil in flatter slopes areas.

1.3. Field Works and Geomechanical Considerations

1.3.1. Field works

10 dynamic percussion light tests (DPL Panda-2) have been performed up to 3.00 m maximum depth. Key soil geomechanical features have been described by correlating DPL test values with Service Limit State (SLS) calculated results. Furthermore 3 Vertical Electrical Sounding (VES) tests up to 15m depth, 8 compaction tests (DPL compaction) and 6 trench pits have also been performed. Discussion follows ahead.

Field works were carried out within 25th to 28thJuly 2017.

1.3.2. Geomechanical considerations

The general geomechanical trend in this site is two main geomechanical layers which can be covered by organic topsoil (not considered).

Two type soil have been described depending on the geotechnical behavior and lithological composition, which can be directly correlated with ramming constraints.

DPL overlapped plots show that La Tourney site underlain materials consist of thin topsoil GU1 (not differentiated) followed by alluvial quaternary deposits GU2 (silty and gravelly brown sands) overlaying at a different depths volcanic weathered bedrock GU3.

Refusal depth has been reached at some test in both GU2 and GU3.

Geomechanical parametrization follow ahead in the report.

- **Geomechanical Units:**

Geomech. Unit	Depth (m)*	N_{SPT} range*	Description
GU1	0 – 0.80	3 – 60	Brow to grey silty CLAY.
GU2	0.80 – >3.00	5 – 100	Brown solghtly gravelly SAND.
GU3	1.20 - >5.00	>100	Volcanic weathered BEDROCK.

**Average value*

- **DPL compaction model – CBR**

8 DPL-compaction tests have been conducted all around the site as well as on anticipated road tracks in order to determine CBR % ratio. Representative %CBR average values are presented below according to correlations given in this report and according to “ALL Soils” classification.

Average CBR % (ALL Soils)							
DCP1	**DCP2**	**DCP3**	**DCP4**	**DCP5**	**DCP6**	**DCP7**	**DCP8**
19	7	6	20	13	36	20	17

Fair results have been calculated in all tests. Just only one results in which CBR=6 have been obtained. Please note that in subgrade layers with CBR<6% compaction is typically recommended. Highest values have been obtained where actual track overlaps.

A typical natural subgrade classification is as follow:

CBR %	Classification
0 – 5	Very poor Subgrade
5 -10	Poor Subgrade
10 – 20	Average – Very Good Subgrade
20 – 30	Very Good Subgrade
30 – 50	Good Base course
50 – 80	Good Base
80 - 100	Very Good Base

Therefore, La Tourney site natural subgrade would fall under the **Poor subgrade Base** to **Good Base Subgrade** range.

- **Pile foundation system**

Geomechanical parameters for pile foundation have been calculated and are shown in table below. Please note that averaged results are provided meaning that different figures might be reported at each particular location.

Following parameters have been either calculated or taken from lab test results for steel frames driven into the ground.

Geotechnical unit	Depth* (m)	N_{SPT} Max	ϕ (°)	γ sat (Tn/m³)	C' / Cu (Kg/cm²)	E (Kg/cm²)
GU01	0.00 – 0.80	20	1.90	*1.59*	0.83	75
GU02	0.80 - >3	100	39 – 51	*1.56 – 1.80*	0.44 – 1.92	450
GU03	0.80 - >3	100	>50	*>1.90*	>1.00	>500

**Average value*

- **Bearing Capacity and settlements for shallow foundations**

Bearing capacity and allowable settlements have been calculated for a typical mat foundation embedded 0.25 m into the ground following classical model equations as shown in table below. The model has been prepared for the DNO building anticipated construction area.

Geotechnical unit	Depth (m)	N_{SPT} Avg	ϕ (°)	γ sat (Tn/m³)	C' / Cu (Kg/cm²)	E (Kg/cm²)
GU2	>0.80	24	39 - 51	*1.56 – 1.80*	0.44 – 1.92	385

According to above parameters, bearing capacity and settlements have been calculated as well. See report for details.

GU1 (to be confirmed once Substation final location determined)

$q_{allowable}$ (kg/cm²) → 2.20 kg/cm²

Above results are for orientation purposes only as have been calculated for a 2.5 x 4.0m mat foundation embedded 0.25 m into a 5.0 m thick layer. Calculations are based upon notes and comments throughout the document and those should be taken into account when designing bearing structures.

1.4. Particular issues

Refusal depth has been reached at any tests as shown in table below:

Test	Refusal depth (m)	Test	Refusal depth (m)
P1	1.20	**P6**	0.80
P2	1.60	**P7**	1.60
P3	>3.00	**P8**	1.40
P4	1.00	**P9**	1.80
P5	0.60	**P10**	1.00

1.5. Hydrogeology, hydrology

Groundwater level nor high humidity content has not been reached in any point or test (penetration, CBR or trial pit).

Regarding surface water, a water course (stream) has been detected in the east side of west field area (close to TP3).

Please note, as a general remark, water level seasonal variations may affect bearing capacity of soils if water table reaches bearing structures' stress bulb. In such case bearing capacity of soil may drop as much as half of its dry value.

<u>In the full area hydrological and hydrological engineering assessment should be sought after for additional details on flooding preventive actions, if any would be required.</u>

1.6. Chemical content and corrosivity

Physic-chemical test results are show in table below:

Test #	pH	Sulfate (SO_4 mg/kg)	Chloride (mg/kg)	EC (µS/cm)
MTP1	6.99	240	35.1	921
MTP2	6.27	144	39.2	619
MTP5	6.44	528	40.4	829

1.7. Resistivity

Resistivity figures show typical silty clay sand with slight plasticity materials up to survey depths (12m to 15m).

VES 1	Depth Lay	Resistivity Average Value
	0,00 - 1,00	2,41
	1,00 - 2,26	26,8
	>2,26	3,27
VES 2	**Depth Lay**	**Resistivity Average Value**
	0,00 - 1,71	5,57
	1,71 - 3,97	80
	> 3,97	5,78
VES 3	**Depth Lay**	**Resistivity Average Value**
	0,00 - 1,80	3,23
	1,80 - 3,64	12,67
	3,64 - 12,62	1,73
	>12,62	42,43

1.8. Rippability and digging mean

Regarding Geotechnical units GU1 and GU2 medium to high equipment shall be used for excavation purposes. Continuous bedrock and dense gravelly layers are expected at shallow depths in some areas where higher forceful means, as pneumatic hammer and predrilling tools, shall be expected.

1.9. Applicable regulations

Methodology used in the project follow EU geotechnical surveying regulations.

- AGS Guidelines for Good Practice in Site Investigation
- BS-EN ISO 22475:2006 “Geotechnical Investigation and Testing – Sampling Methods and Groundwater Measurements. Technical Principles of Execution”
- Eurocode 7 (2007) “Geotechnical Design - Part 2: Design Assisted by Laboratory Testing” (BS EN 1997-2)
- Eurocode 7 “Geotechnical Design - Part 3: Design Assisted by Fieldtesting” (ENV 1997:3-1999)
- CIRIA R-143
- Regulations related to specific field and laboratory testing have been listed in table 3a and 3b
- NF-XP-94-105 Panda 2 standard
- AWWA C-105 soil corrosivity standard

2. CHAPTER II. PRELIMINARY SITE INFORMATION

2.1. Background

Tecsolgeo, Ltd has been commissioned to undertake this geotechnical survey and assessment report at La Tourney site according to test unit prices included in the Proposal ref. 2016-SL-31 v2 dated June 26th, 2016.

Proposed development comprises 6.0 Ha site located close to La Tourney Town, in Vieux Fort district, (Saint Lucia). According to information provided by the Client, loads will be transmitted to the ground through PV panels mounted on light metal bearing structures, while DNO buildings and ancillary buildings are to be built on shallow foundations (mat foundation) according to client too. PV panels' bearing structures to be designed by client or any appointed subcontractor and will consist of either rammed in metal piles (otherwise called posts) or any miscellaneous foundation support as per own client's design.

Additional information has been provided to complete this report such as:

- General Plant Drawings
- Google Earth map

This report was prepared for the exclusive use of the Client and their consultants for design of this project. In the event that any changes are made in the character, design or layout of the improvements, the Consultant must be contacted to review the conclusions and recommendations contained in this report to determine whether or not modifications are necessary.

The ground surface is relatively flat, raising gradually toward the hills to the north. The natural vegetation consists of grasses and shrubs with larger tropical trees occupying the hillslope to the north. West fields are being cleared at the of undertaking this survey.

2.2. Site Access

The site is accessed from Vieux Fort via the Route Judes Hwy and then arriving to Cedar Heights, by following instructions to Cedar Heights Recreational Grounds until the site entrance (Block 1218B, parcel 232). Please see Annex 1 for additional location details.

2.3. Site Preliminary Information

2.3.1. General geological settings

The Seismic Research from West Indies University provides with relevant local geological information on this area. The site comprises several geologic formations among which the most important are Quaternary and volcanic weathered bedrock.

Surveyed area is located at the Saint Lucia SE area, in the district of Vieux Fort.

Saint Lucia is made up almost entirely of volcanic rocks. Like all of the islands of the Lesser Antilles, Saint Lucia began its life as a series of submarine volcanoes. After many eruptions over millions of years these volcanoes built large topographic features that slowly rose above the surface of the water, joined with neighboring volcanic islands, and grew to the island we see today.

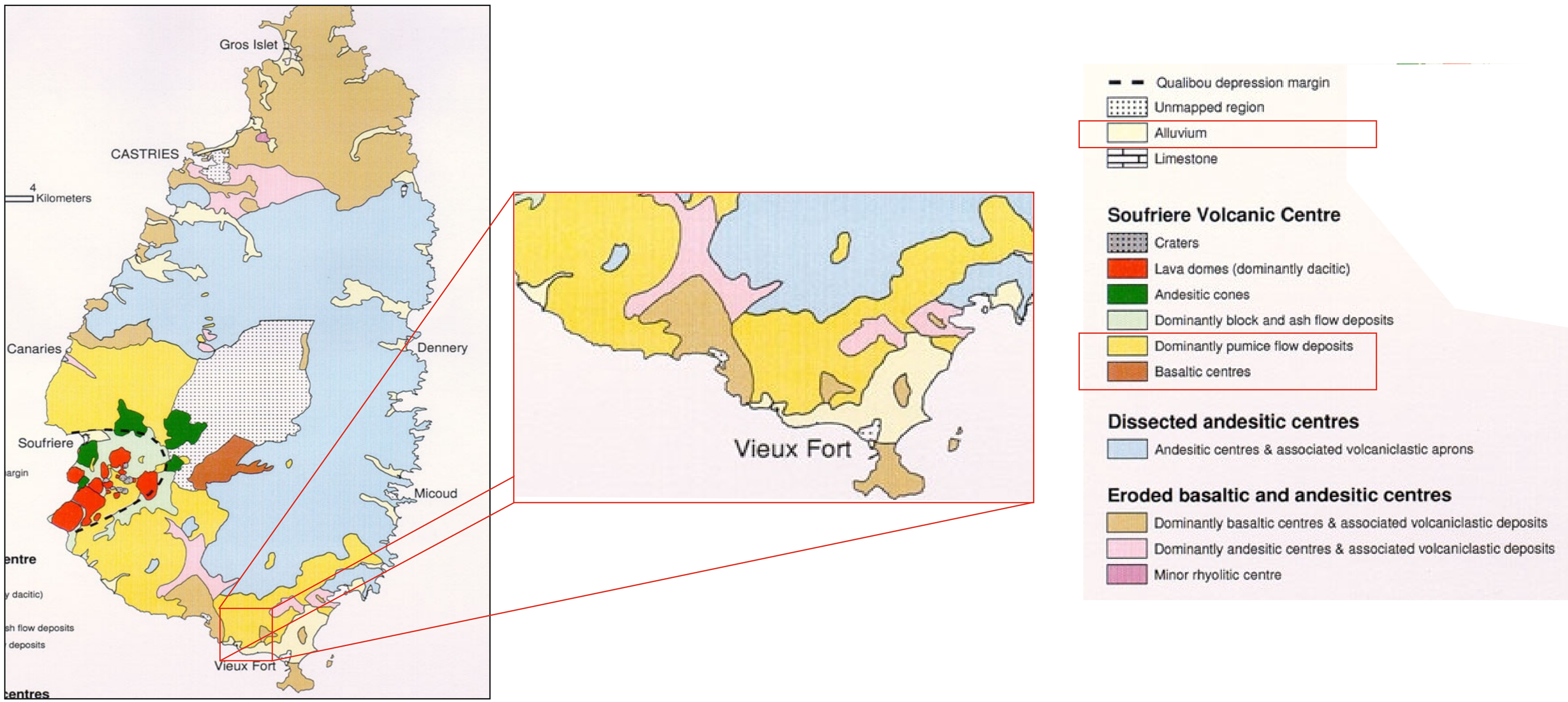
Newman (1965) divided the volcanic centers in Saint Lucia into 3 broad groups based on age and geographic distribution. From oldest to youngest these groups are the Northern, Central and Southern series.

The island of Saint Lucia and the associated undersea ridge on which it is perched are located approximately 150 kilometers from the east edge of the Caribbean plate where it meets the North American plate. The Caribbean plate is under thrust by the North American plate which passes down into the mantle where melting occurs.

Seismic considerations:

Saint Lucia has an intermediate seismic hazard. The island lies in a transition zone where the rate of seismic activity is climbing. The island's seismic hazard is not as low as St. Vincent's but it is not as high as Martinique's.

See in figure 1 below, regional geological context for surveyed area.



Project # IG62717

Date: 03/08/2017

Client: Grupotec

FIGURE 1

General Geology in La Tourney site area

Source: University of the West Indies
Seismic Research Centre

From the geological description above it turns out that Quaternary detritical alluvial sediments (silty sands, and poligenical and heterometrical gravels) and weathered volcanic bedrock dominates the whole area within and in the surroundings of the site.

2.3.2. Hydrology and Hydrogeology

Groundwater level nor high humidity content has not been reached in any point or test (penetration, CBR or trial pit).

Regarding surface water, at the moment of conducting field works, water stream has been detected, which carries surface water from the north side area and cruising the site. Furthermore, in the southeast area can be located the Vieux Fort River, where surface waters would be finally collected.

Also from topographical survey supplied, it seems that at the moment of carry out the survey, there were some bogged areas.

These events suggest that the area is susceptible for surface water accumulation, which may compromise the project if preventive measures are not considerated.

Please note, as a general remark, water level seasonal variations may affect bearing capacity of soils if water table reaches bearing structures' stress bulb. In such case bearing capacity of soil may drop as much as half of its dry value.

In the full area hydrological and hydrological engineering assessment should be sought after for additional details on flooding preventive actions, if any would be required.

See figure 2 below for more details.

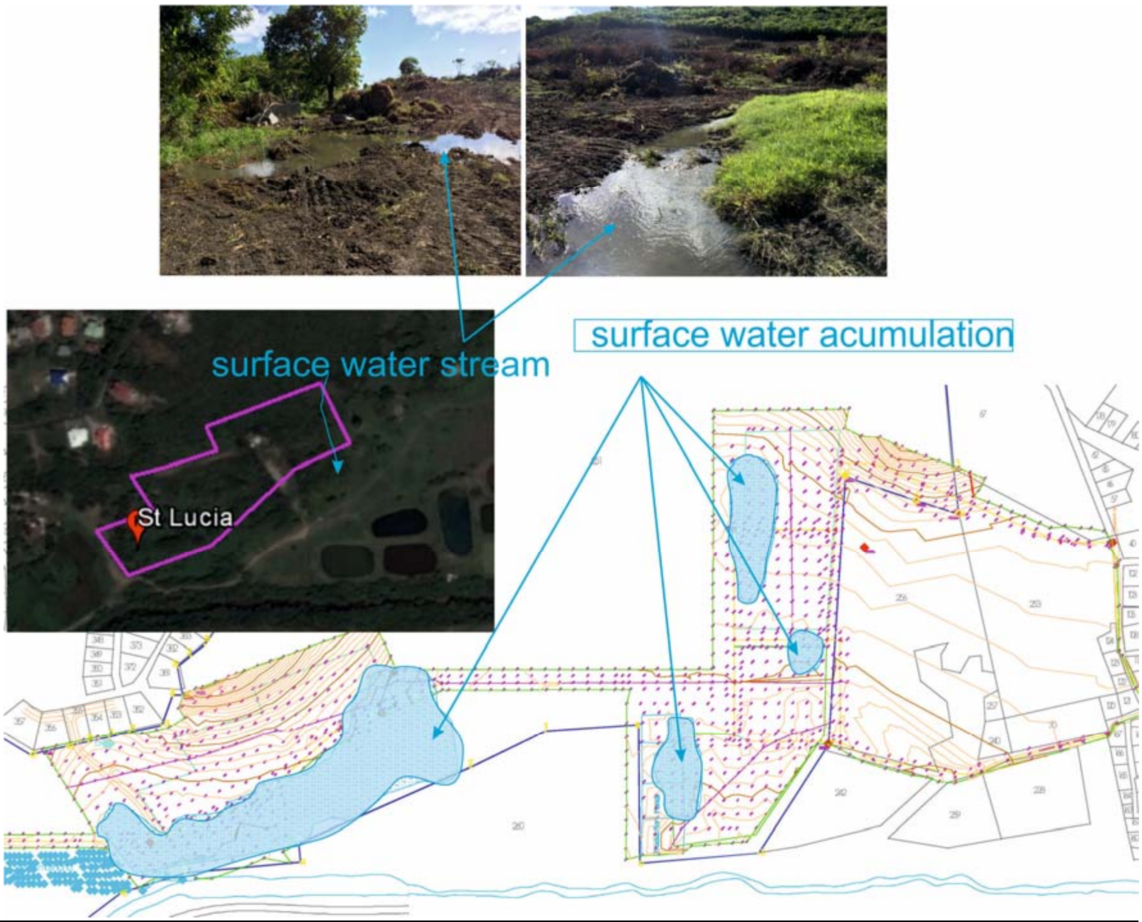


Figure 2: Surface water stream and acumulation areas

2.3.3. Historical Development

Evidences on site from ancient activities have been observed. At the time of the field investigation the site contained several: fish and shrimp ponds at the southwest section of the property and the majority of the site was used as grazing for cattle and horses.

2.4. Field survey and assessment plan

Current geotechnical survey was planned according to the following schedule (next page):

Phase	Work description
1	Design and scope of investigation
2	Check historical records/sources of information: - previous investigations - previous use - mining/landfill - geology/groundwater - adjacent sites - services records
3	Location of field tests on a site map Validation of test location with the client
4	Define supervision requirements and organization
5	Soil testing/fieldwork
6	Soil sample collection and delivery to laboratory
7	Preliminary report including fieldworks and preliminary soil model
8	Laboratory testing in accredited laboratory
9	Geotechnical soil modeling, data analysis
10	Geotechnical reporting
11	Follow-up

Table 1: Geotechnical investigation phases

3. CHAPTER III. FIELD EXPLORATION AND LABORATORY TESTING

3.1. Testing standards

Field tests have been conducted according to the following standards, where applicable:

Test	Field/Lab	BS/ASTM/Others
Disturbed sample	Field test	ASTM D1586
Panda 2 TEST	Field test	NF-XP-94-105
DCL compaction	Field test	NF-XP-94-105
Trial Pits	Field test	BS5930
VES Tests	Field test	ASTMD6431-99

Table 2: Field tests carried out

Lab tests are being conducted according to the following standards, where applicable:

Test	Field/Lab	Standard
Particle size distribution	Lab test	UNE 103.101
Plasticity	Lab test	UNE 103.103-104
Chloride content of soils	Lab test	UNE-EN 1744-1. AP-9.
pH	Lab test	UNE-ISO 10390:2012
Electrical conductivity of soils	Lab test	UNE 77308:01
Direct shear strength (unconsolidated, undrained)	Lab test	UNE 103.401/98
Densities (dry, wet)	Lab test	UNE 103.301
Moisture content	Lab test	UNE 103.300
Sulfate content	Lab test	UNE 103 102:1995

Table 3: Lab tests carried out

Classification of soils according to CIRIA-143R:1995 and USCS standards.

3.2. Soil surveying and testing

3.2.1. Field works description

Field works have been carried out within a building-free area. The site comprises fairly flat topography except for toward the hill northern areas for each field and an irregular shape. At the time of undertaking the survey, western field is being cleared. The site contain several vegetation that shall be removed prior to any construction works take place.

- **Percussion DPSH testing**

Ten (10) dynamic percussion light tests (DPL), Panda-2 type, have been undertaken onsite in order to determine:

a. Hard layer presence depth and/or refusal depth;
b. Geomechanical behavior of soils according to rammed in metal bars energy (MPa).

For the current exercise we have proposed the use of a light weight variable energy penetrometer (Panda 2) which is comparable to DPSH up to 5m depth. NF-XP-94-105 standard applies to this test. Actual test consist of driving a rod with sacrificial cone into the ground, using a manual standardized weight falling over. Energy is measured through a heading device. The rammed in distance is then measured each blow being the tip dynamic resistance obtained. The Dutch formula is used therein to obtain actual dynamic figures. An integrated software (TDD) is used for record and calculation purposes.

- **Disturbed sample test**

3 disturbed samples have been collected in MTP1, MTP2 and MTP4 tests from trench pit walls. Samples have been then delivered to our accredited soil testing laboratory. Samples were taken with hand shovel and put into a plastic bag and then stored in a portable sample case in order to avoid loss of moisture.

- **DCP – CBR in situ Test**

Eight (8) DPL compaction tests have been conducted on site for in situ CBR determination purposes within main building areas and road tracks. Main difference with conventional DPL test is the cone area which is, in this case, 2 sqcm

- **Vertical Electrical Sounding (VES)**

Three (3) VES tests have been conducted including anticipated substation areas in accordance to ASTMD6431-99. Underground soil electrical sounding has long demonstrated to be a very useful geophysical technique to appraise large areas where electrical devises are to be installed. Different methods are available though. The tetraelectrodic disposal method has been chosen to meet resistivity testing goals.

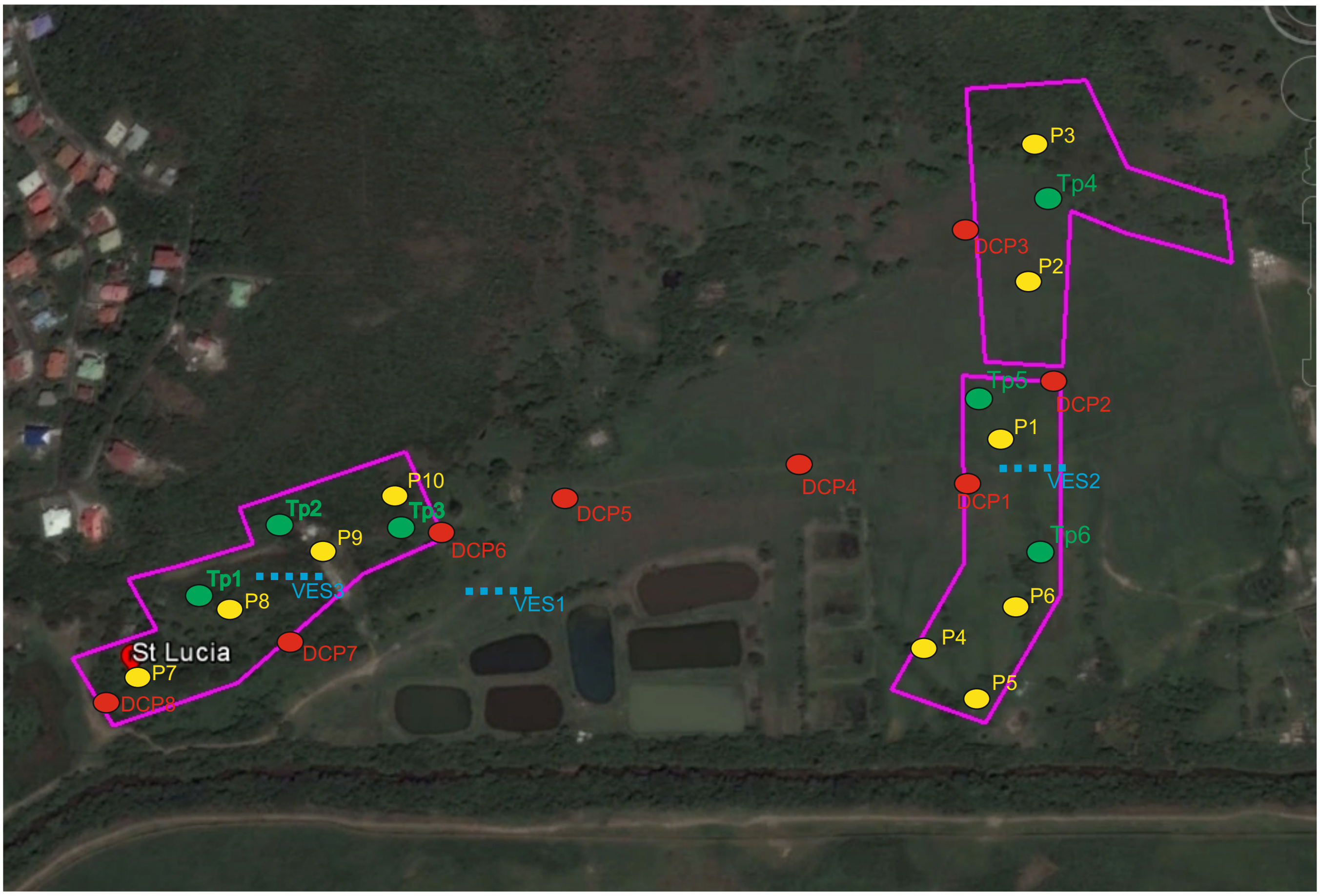
Vertical Electrical Soundings (VES) is very well known and well documented in the industry; in this method DC electrical currents are transmitted to the ground by means of four pikes rammed into the ground: two pikes (electrodes) are used to introduce DC currents into the ground and two additional pikes (voltage or potential electrodes) are used to obtain resistance values. Electrical current is measured a few seconds after first shot and electric potential difference arises between both electrodes. Pikes are aligned following the building main direction. Measurement is then repeated by placing pikes at different distances, allowing different measurements to be performed at different depths. In the current exercise we’ve used a Wenner array.

- **Trial Pits (TP)**

Six (6) Trial pits have been conducted all around the site. A comprehensive soil description survey has been undertaken by describing trial pits as well as outcrops within and in the surroundings of the site area. Trench Pits have been excavated up to 3.80 m depth by means of a mechanical excavator. Trial pits description help us to geotechnical/geological interpretation from resistance tests performed

3.2.2. Location of in situ tests

In situ tests have been distributed within site boundaries as shown in fig. 2 and 3 next page.



Project IG62717	La Tourney Site Test Location Map - PV area Trial Pits, Penetration and CBR Test
Date: 02/08/2017	Dinamical Penetratión Test Trial Pit CBR Test VES Test
Client: Grupotec	
FIGURE 3	*Not to scale *approximate tests location

In situ tests See in table below coordinates points for all tests performed (DPL, DCL, Trial Pits and SEVS):

La Tourney Saint Lucia					
	X	Y	UTM ZONE 19	X	Y
P1	720500,00	1519669,00	DCP5	720162,00	1519621,00
P2	720518,00	1519785,00	DCP6	720067,00	1519597,00
P3	720527,00	1519901,00	DCP7	719951,00	1519514,00
P4	720445,00	1519534,00	DCP8	719822,00	1519459,00
P5	720480,00	1519480,00	TP1	718881,00	1519531,00
P6	720518,00	1519571,00	TP2	719952,00	1519600,00
P7	719834,00	1518477,00	TP3	720046,00	1519597,00
P8	719894,00	1519525,00	TP4	720524,00	1519867,00
P9	719998,00	1519585,00	TP5	720480,00	1519706,00
P10	720041,00	1519629,00	TP6	720530,00	1519595,00
DCP1	720467,00	1519652,00	VES1	720124,00	1519572,00
DCP2	720536,00	1519709,00	VES2	720524,00	1519653,00
DCP3	720477,00	1519819,00	VES3	719937,00	1519552,00
DCP4	720342,00	1519644,00			

Table 4: **Test Point coordinates**

3.2.3. Field and laboratory test results

3.2.3.1 Field test results

- **Dynamic percussion test results**

Individual dynamic percussion light test (DPL) plots are presented in *Annex 2: DPL plots (individual plots)* at the end of this report. In Table 4 DPL test number and investigation depths are shown.

Test	Refusal depth (m)	Test	Refusal depth (m)
P1	1.20	**P6**	0.80
P2	1.60	**P7**	1.60
P3	>3.00	**P8**	1.40
P4	1.00	**P9**	1.80
P5	0.60	**P10**	1.00

Table 5: Refusal depths

In Figure 4 DPL plots have been overlapped in order to clearly identify geomechanical families.

Correlation with Service Limit State (SLS, $\mathbf{q_u}$) is as follow:

$$\mathbf{q_u\ (kg/cm^2) = [q_d\ (MPa)/12]\cdot 10}$$

Such correlation provides with actual direct bearing capacity.

Cross sections showing DPSH results laid on a topographic elevation have been set out and are presented next page (Figures 5 and 6).

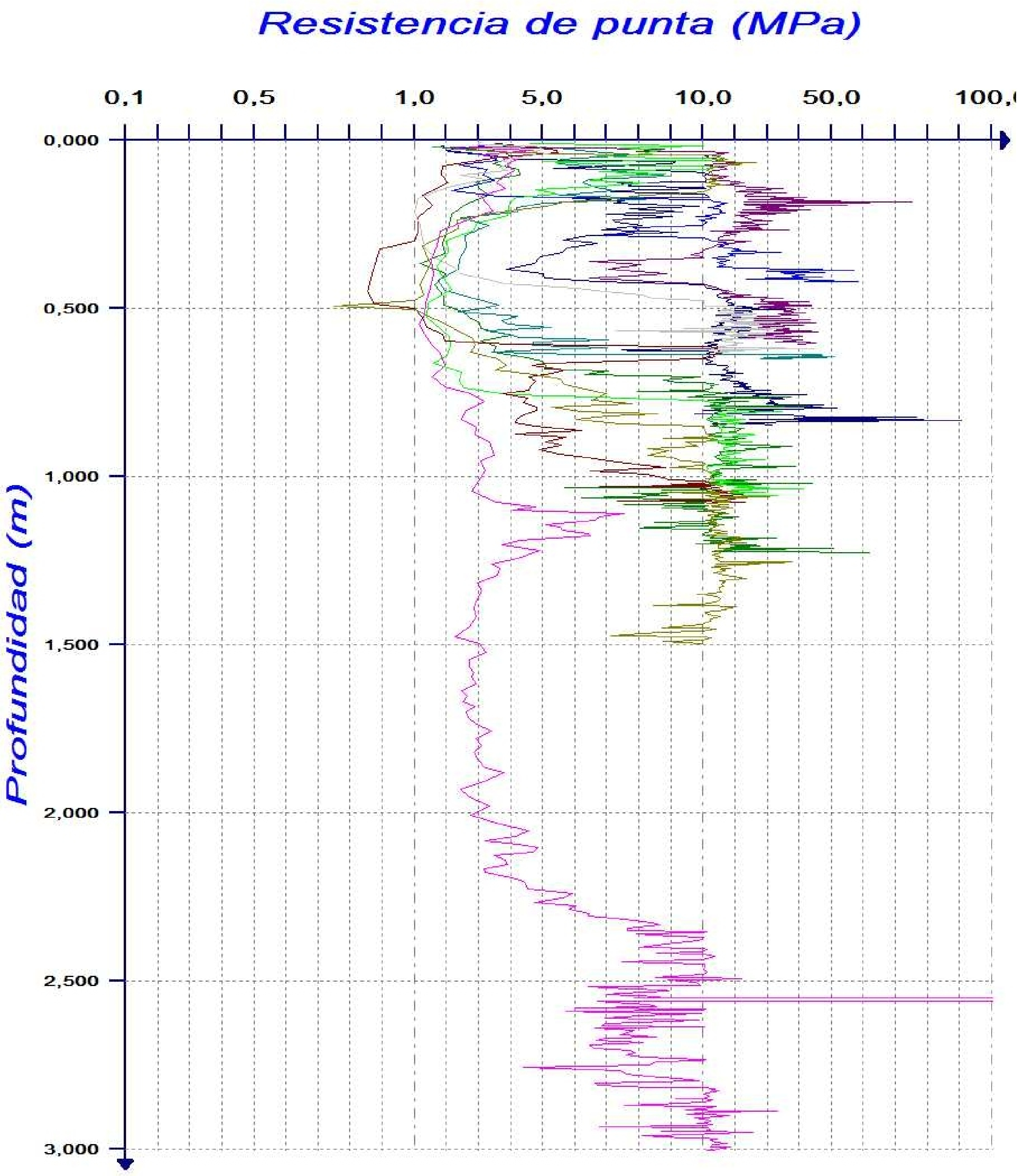
For Ultimate Limit State (ULS, $\mathbf{q_h}$), or Ultimate Bearing Resistance, correlation with N_{SPT} is as follow (table 6. Granular soils with <30% of particles below 20mm). Where higher clay content is found figures this correlation may change slightly.

	Very loose	Lose	Semi-Dense		Dense		Very dense
ϕ (º)	30	32	34	36	38	40	42
N_{SPT}	10	15	22	30	36	45	55
q_d (MPa)	0	2	4	7	15	21	30

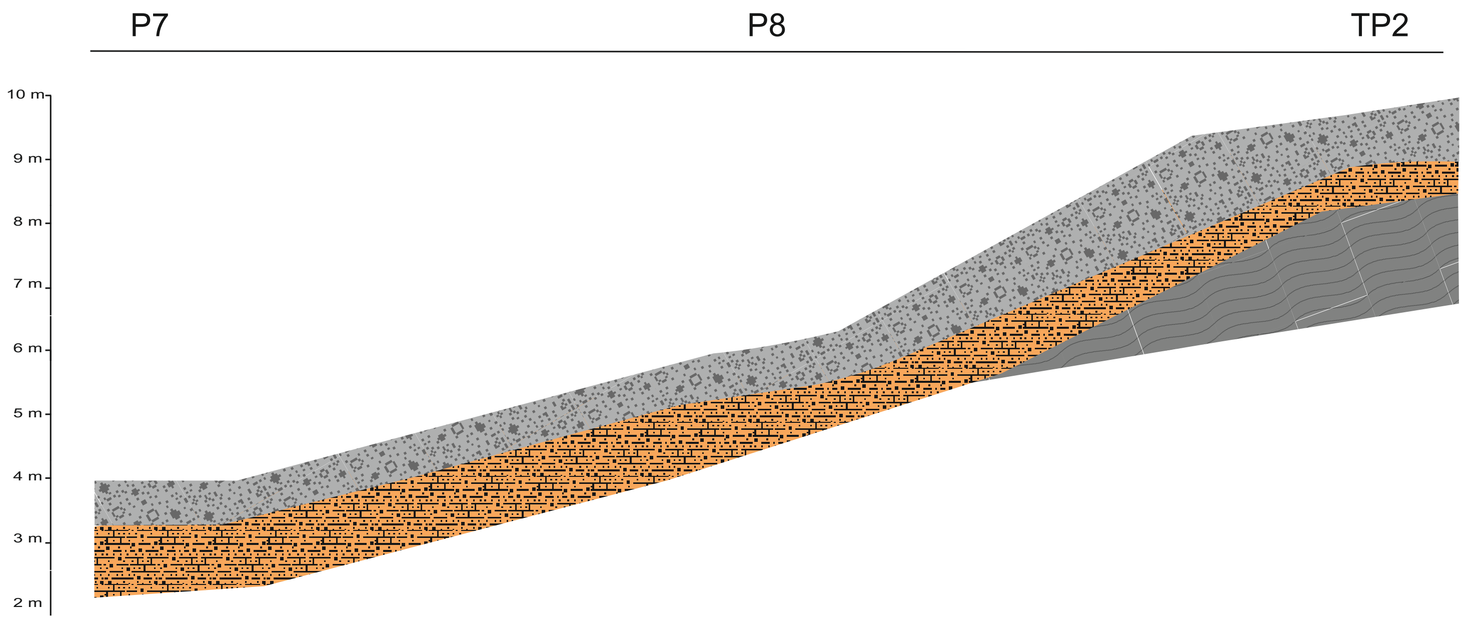
Table 6: N_{SPT}, q_d and internal friction angle correlations

Results show that most of the site geomechanical behavior lay under 5.0 Mpa, that is, semi-dense materials.

Table 7 below shows Dynamic percussion DPL test results correlated to N_{SPT} value for each test at each depth

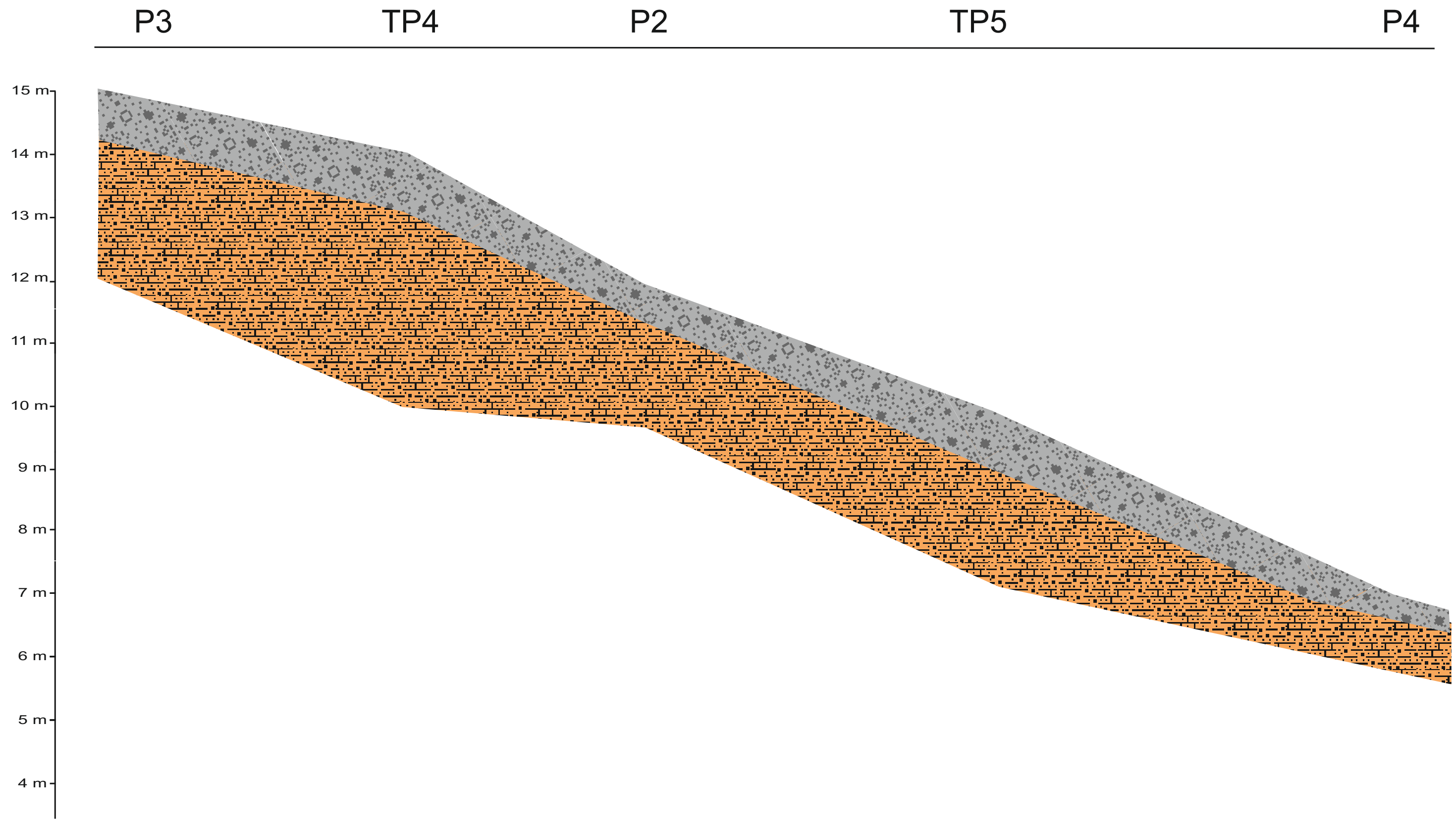


Project IG62717	La Tourney Site - Panda tests Superposition tests	
Date: 02/08/2017	Comments:	TECSOL geo
Client: Grupotec		
FIGURE 4		



Not exact topographical correction

Legend	Project IG62717	CROSS SECTION La Tourney (Saint Lucia)	TECSOL geo
GU1 - Topsoil	Date: 08/08/2017		
GU2 - Alluvial Quaternary	Client: Grupotec		
Gu2 - Volcanic Bedrock	FIGURE 5		



Not exact topographical correction

Legend

- GU1 - Topsoil
- GU2 - Alluvial Quaternary
- Gu2 - Volcanic Bedrock

Project IG62717	CROSS SECTION La Tourney (Saint Lucia)
Date: 08/08/2017	
Client: Grupotec	
FIGURE 6	

TECSOL geo

Depth/N30	P1	P2	P3	P4	P5	P6	P7	P8	P9	P10
0,2	20	8	9	16	17	60	19	8	31	9
0,4	17	5	5	7	50	53	7	2	5	4
0,6	44	7	3	9	104	78	4	3	4	47
0,8	58	32	5	72		100	19	22	13	53
1	98	44	8	100			44	15	24	100
1,2	100	31	13				54	37	38	
1,4		74	9				100	100	42	
1,6		100	7				100		27	
1,8			7						100	
2			8							
2,2			10							
2,4			18							
2,6			83							
2,8			20							
3			31							

Table 7: Dynamic Percussion DPL tests (N_{30}=number of blows/30cm shown).

- o **USCS classification**

According to USCS, soils are classified as follow (Table 8):

	MTP1	MTP2	MTP4
Soil classification (USCS)	*SW*	*SW*	*SC*

Table 8: USCS Soil Classification

- **CBR in situ Test**

In situ DPL compaction - CBR tests were conducted.

CBR results have been deduced from DPL (compaction mode) test results by using following correlation:

$$\mathbf{CBR\ (\%) = 2.14 \cdot q_d\ (MPa)}$$

Average results are shown in table 9 below as per “ALL SOILS” state soil model (NAFVAC classification).

Average CBR % (ALL Soils)							
DCP1	**DCP2**	**DCP3**	**DCP4**	**DCP5**	**DCP6**	**DCP7**	**DCP8**
19	7	6	20	13	36	20	17

Table 9: Average CBR value

Fair results have been calculated in all tests. Just only one result (for DCP3) in which CBR=6 have been obtained. Please note that in subgrade layers with CBR<6% compaction is typically recommended. Highest values have been obtained where actual track overlaps.

A typical natural subgrade classification is as follow:

CBR %	Classification
0 – 5	Very poor Subgrade
5 -10	Poor Subgrade
10 – 20	Average – Very Good Subgrade
20 – 30	Very Good Subgrade
30 – 50	Good Base course
50 – 80	Good Base
80 - 100	Very Good Base

Therefore, La Tourney site natural subgrade would fall under the **Poor Subgrade** to **Very good Base Subgrade** range.

Individual DCP plots are presented in *Annex 2: DPL plots (individual plots)* at the end of this report.

- **Vertical Electrical Sounding (VES tests)**

VES test results are provided in Annex 7 at the end of this report.

Resistivity figures show typical silty clayed sand with slight plasticity materials up to survey depths (12m to 15m).

VES 1	Depth Lay	Resistivity Average Value
	0,00 - 1,00	2,41
	1,00 - 2,26	26,8
	>2,26	3,27
VES 2	**Depth Lay**	**Resistivity Average Value**
	0,00 - 1,71	5,57
	1,71 - 3,97	80
	> 3,97	5,78
VES 3	**Depth Lay**	**Resistivity Average Value**
	0,00 - 1,80	3,23
	1,80 - 3,64	12,67
	3,64 - 12,62	1,73
	>12,62	42,43

Table 10: VES test results

- **Trial pits**

In Annex 4 a detailed description of trial pits is provided. Generally speaking, La Tourney site underground soil is made up of an uppermost topsoil layer, followed by alluvial quaternary layer (slightly gravelly silty sands), and volcanic weathered bedrock underneath which extends up to 3.00m depth.

3.2.3.2. Laboratory test results

Laboratory test results are summarized in Table 11 below:

Test	Sample: MTP1 (Trench Pit 1)	Sample: MTP2 (Trench Pit 2)	Sample: MTP5 (Trench Pit 5)
Soil classification (USCS)	SM	MH-OH	SM
Soil description	Consolidated clayey SAND	Browny CLAY	Slightly consolidated SAND
Particle size distribution	4.2% G 69.0% S 26.8% F	4.2% G 69.0% S 26.8% F	15.3% G 78.0% S 6.7% F
Plasticity	NP	Ll: 72.39 Lp: 42.11 Lp: 30.28	NP
Sulfate (mg/kg SO4) content	240	144	528
Density (dry, wet) (gr/cm^3)	1.56/1.30	1.59/1.16	1.80/1.59
Moisture content			
Shear stress test – cohesion (kp/cm^2)	1.92	0.81	0.44
Shear Stress test-friction angle (º)	39.96	1.90	51.85
Uniaxial compression (kp/cm^2)	-		

Table 11: Lab test results

- **Chemical tests and corrosivity assessment**

Physic-chemical test results are show in table below.

Test #	pH	Sulfate (SO_4 mg/kg)	Chloride (mg/kg)	EC (µS/cm)
MTP1	6.99	240	35.1	921
MTP2	6.27	144	39.2	619
MTP5	6.44	528	40.4	829

Table 12: Physic-chemical test results

Chemical soil composition can be differentiated in two different groups, which are the chemical composition of all water soluble components presents in the soil and soil mineralogy and composition.

First group together with moisture content, pH and contaminants presence is closed linked to the soil conductivity ability and its potential corrosivity capacity for buried elements. Regarding mineralogy and soil composition, are related to its special features (sulfates, carbonates,..).

Once said this, there are two ways to classify soil corrositvity, for parameter harshness or corrosivity numerical scales (AWWA C-105 and DVGW GW9).

According to physicochemical test results in page table 12, from AWWA C-105 results moderate corrosivity category, and from DVGW GW9 results as corrosive soil.

Regarding soil agressivity issue, sulfate content values obtained from most of samples tested involves the use of a regular concrete which will ensure the concrete durability against external agents.

3.3. Interpretation and Distribution of geotechnical units

3.3.1. Interpretation

Field and laboratory test results' comprehensive analysis is required in order to properly define the geological/geotechnical model.

By looking figures and Tables attached two main geomechanical unit (GU2 and GU3) have been described. Distribution on the unit can be examined from DPL tests results attached and N_{SPT} Assessment.

3.3.2. Distribution of geotechnical units (GU)

Geotechnical units have been differentiated according to a common mechanical behavior of the ground:

GU2 – Alluvial Quaternary Detritic Unit is mainly made up by medium to hard slightly gravelly and silty sand and conform the quaternary detritical sediments that dominates the whole area within and in the surroundings of the site.

During trial pit excavation have been detected compacted dense sandy interlayers with rock behavior which may cause refusal while ramming works. See picture 7 below for details.

Figure 7: Dense sandy layer on left picture. Usual GU1 appearance for right one

This unit have been interpreted from dynamic penetration test 1 to 8 below topsoil GU1 unit and up to 3 meters or refusal depth. Have also been logged from trial pit 1 and 4 to 6.

It is a quite homogeneous unit, with higher N_{SPT} values in those layers where the presence of gravels and high compacity strata can induce refusal results.

Layers wedge out horizontally. Therefore, the lithological vertical sequence vary at different locations within the site as has been detected.

See linked videos below with excavation procedure for that pits:

TP1: https://youtu.be/3MRUG9DT3TA
TP4: https://youtu.be/K3VUbjAOSww
TP5: https://youtu.be/PTeIZjyMiyA
TP6: https://youtu.be/aITi2QOxMoc

GU3 – Weathered volcanic Bedrock is mainly made up highly weathered basalt sand and conform the bedrock that dominates the whole area within and in the surroundings of the site. It's defined as a very dense unit, with N_{SPT} values within 24 and 100.

During trial pit excavation have been detected in trial pit 2 and 3 and is not excavated by the use of low to medium lower equipment. See picture 8 below for details.

Figure 8: Bedrock presence in TP2. Bedrock lithology detail.

This unit have been interpreted from dynamic penetration test 9 to 10 and cause the penetration refusal. Have also been logged from trial pit 2 to 3.

See linked videos below with excavation procedure for that pits:

TP2: https://youtu.be/lt50AdYNxGk
TP3: https://youtu.be/VBWFQ2j3-q8

N_{SPT} typical value ranges for GU2 and GU3 described summarized in table 13 below.

Geomech. Unit	Depth (m)*	N_{SPT} range*	Description
GU1	0 – 0.80	3 – 60	Brow to grey silty CLAY.
GU2	0.80 – >3.00	5 – 100	Brown slightly gravelly SAND.
GU3	1.20 - >5.00	>100	Volcanic weathered BEDROCK.

Table 13: SPT Range values for diferent Geomechanical units

3.3.3. Relevant assumptions to the geomechanical model

(a) Data collected is representative for the whole surveyed area;
(b) Interpretation made for the whole area may not correspond to the real state.
(c) Test results are representative for the geomechanical unit they've been collected from;
(d) Individual N_{DPSH} plots should be relied upon where DPL test has been performed (see annex).

Once said this, described geomechanical units (GU2 and GU3) have caused refusal for penetration test and for trial pit excavation, so may also cause refusal for ramming works.

Picture bellow in the next page (figure 9) shows zoned areas and includes an estimated ramming viability map.

Low ramming viability is considered for these areas where weathered volcanic bedrock appears at a low depth, just below topsoil or where GU2 is a thin layer. Medium ramming viability is considered for these areas where dense and gravelly interlayers have been identified inside GU2 unit, and where these layers have caused penetration refusal.

This both areas as mainly located in western fields, and is in line with the topography, such that where isn't flat topography is most probably getting problems for ramming works.

Furthermore, on plain areas, where there is flat topography, ramming works would be feasible at least until maximum depth reached in this survey, and have been classified as high ramming viability area.

Ramming Viability

- Low
- Medium
- High

**Estimated. Real case may differ*

Project # IG62717	La Tourney Site - Ramming Viability map
Date: 07/08/2017	
Client: Grupotec	
FIGURE 8	

3.3.4. Groundwater level (GWL)

Groundwater level nor high humidity content has not been reached in any point or test (penetration, CBR or trial pit).

Groundwater level is probably located at the time of undertaking field works at a deeper depth than maximum reached. Should be noted that groundwater levels at the site would be subject to fluctuations and seasonal variation, and could be significantly higher in the wet season. In addition, run off from the adjacent hill to the north of the site may result in flooding of the site during periods of intensive rainfall.

Flood protection measures should be considered for this site.

Please note, as a general remark, that water level seasonal variations may affect bearing capacity of soils if water table reaches bearing structures' stress bulb. In such case bearing capacity of soil may drop as much as half of its dry value as effective pressures apply.

4. CHAPTER IV. GEOTECHNICAL DESIGN

4.1 Site preparation

Following some general recommendations on grading and backfilling are provided. Please note that such recommendations shall be adapted and tailored to each particular foundation and bearing structure and by no means they should be implemented without previous approval from appointed engineer and engineering supervision.

4.1.1. Grading

DPL compaction - CBR tests results in tested locations suggesting that no additional subgrade compaction would be required in most locations. Please note that subgrade soils having CBR<6% are typically required to undergo compaction in order to increase bearing capacity and own strength

Earthwork recommendations are intended for use in structural areas that will support improvements such as the substation, equipment pads and roads. For the solar array areas, we understand the panels will generally follow existing topography with very little or no grading.

Structural areas are hereby defined as any area sensitive to settlement of compacted soil. These areas include, but are not limited, to equipment pads, buildings, and pavement areas. Solar array areas where piles will be driven into undisturbed native soils are not considered structural areas for earth work purposes. However, if minor grading is necessary within array areas and compacted fill will be used to support pile foundations, then these should be considered structural areas and the earthwork recommendations presented below would apply.

Topsoil and organic soils (if present) should be removed from below any proposed structures, buildings, access roads, and parking structural areas. The exposed subgrade should be proof-rolled and inspected by qualified geotechnical engineering personnel prior to construction of buildings and placement of roadway and parking lot granular fill. Any loose/soft or wet areas identified at the time of proof-rolling that are unable to uniformly be compacted should be sub-excavated and removed. The excavations created through the removal of these materials should be backfilled with approved engineered fill consistent with Argentinean and international civil engineering regulations.

4.1.2. Excavation and backfill

Approved engineered fill to backfill any foundations should consist of clean, granular fill. Geotechnical inspections and testing of engineered fill are required to confirm acceptable quality. Fill should be placed in 20 cm to 30 cm thick lifts and compacted to 95% to 98% Proctor depending on each particular circumstances. Compaction to 95% of standard may be acceptable for general site grading and in any service trenches although local engineering regulations must prevail in any case as stated above.

Ancillary buildings and miscellaneous structures may be founded on conventional concrete slab, or mat compacted engineered fill overlying overburden. Engineered fill underneath building foundations should be compacted to conveniently approved% of standard density (typically 95 to 98%), as verified by geotechnical testing and inspections. Exterior footings of buildings and structures founded in native soils or engineered fill should be constructed below the frost penetration depth or be insulated to provide equivalent protection.

Typical engineering recommendations include the removal of first 45cm topsoil layer (if required) otherwise stated by the engineer in charge. It should be then filled in well graduated gravel compacted to the approved Proctor ratio as discussed above.

4.2.Geotechnical issues

4.2.1. General issues

Two main geotechnical unit have been differentiated. Overall site is quite heterogeneous and is made up of slightly gravelly sands layers and weathered volcanic bedrock, covered by uppermost topsoil all over the site. When aproaching high percentage and dense gravel layers as well as bedrock unit, DPL device delivers increasingly higher blow/cm figures causing penetration refusal. That is observed in DPL test plots.

4.2.2. Particular issues

Based on individual DPL tests performance and laboratory tests following particular issues may be highlighted.

a. Refusal depth has been reached in almost all performed DPL tests where the presence of bedrock and dense levels; The presence of gravels and dense sand levels may cause occasional issues when driving piles into the ground.
b. Two main geotechnical and lithological behavior zones have been identified.

c. These conclusions have been defined from test points performed and have been extrapolated to the whole area.
d. Both defined units may cause refusal when ramming works.

4.3. Foundation design

4.3.1. General correlations

Following correlations have been used for calculation purposes, where required:

- Trunk resistance (τ) = $2.5 \cdot N_{SPT} \cdot 0.8$; 0.5 correction factor may be included should underground water table be present.
- q_u: $Cu = 1/2qu$
- Elastic modulus; D'Appolonia (1975) for fine sands: $E' = 350 \cdot Log(N_{SPT})$
  Elastic modulus; D'Appolonia (1975) for coarse sands: $E' = 500 \cdot Log(N_{SPT})$
  Elastic modulus; Butler (1975) for clay: $Eu = 400 \cdot Cu$
- Friction angle; Gimenez Salas: $\phi = 34.9 - 0.338 \cdot PI$ / Osaki: $15 + (N_{SPT} \cdot 20)^{1/2}$
- Lateral earth pressure (A=active, 0=passive).
  $K_A = (1 - \sin(\phi))/(1 + \sin(\phi))$
  $K_0 = 1 - \sin(\phi')$; Jaky (1944)

4.3.2. Pile foundation system

For the purpose of this report, it is assumed that the preferred option for support of the solar panel frames is steel pile driven to depths adequate to provide the necessary uplift and lateral support. Please be advised that where high $N_{SPT/DPSH}$ figures are found ramming in may be hindered or even unfeasible. In any case, following recommendations shall apply to the ramming in process as a whole (Table 14).

Case #	*Procedure*	*Action*
1	Stud tip reaches necessary foundation depth	No additional actions required.
2	Stud tip does not reach necessary foundation depth. However, minimal bedrock depth without pull-out tests calculated by geologists	Stud can remain in the ground, load bearing capacity is either determined or confirmed by geologist.
3	Stud tip reaches minimal bedrock depth with pull-out tests; the ramming process is stopped before deformation of stud. Stud is attached to the pile driving device which pulls it out although stud cannot be pulled-out.	Stud can remain in the ground; load bearing capacity is either determined or confirmed by geologist; pull-out test must be performed in every single affected stud (in 1 every 3 in case more than 10 tests have been successfully carried out).
4	Stud tip reaches minimal bedrock depth with pull-out tests; the ramming process is stopped before deformation of stud. Stud is attached to pile driving device which pull out	Pre-drillings down to necessary depth for loose soils is required. Drilled holes shall be filled in gravelled sand or miscellaneous materials, then

	the stud.	studs driven down to necessary ramming depth.
5	Stud does not reach necessary bedrock depth with pull-out test.	Pull-out stud and pre-drill

Table 14: Ramming procedures on different scenarios

Geomechanical parameters for stainless steel piles are provided in Table 15. Please note that where laboratory test results were not available, geomechanical parameters have been calculated taking **peak N_{SPT} values** in order to consider worst scenarios for the pile ramming process all over the site.

Geotechnical unit	Depth* (m)	N_{SPT} Max	ϕ (º)	γ sat (Tn/m³)	C' / Cu (Kg/cm²)	E (Kg/cm²)
GU01	0.00 – 0.80	20	1.90	*1.59*	0.83	75
GU02	0.80 - >3	100	39 – 51	*1.56 – 1.80*	0.44 – 1.92	450
GU03	0.80 - >3	100	>50	*>1.90*	>1.00	>500

**Average value*

Table 15: Geomechanical parameters for metal pile design (calculated).

4.3.3. Bearing capacity of soils and allowable settlements for shallow mat foundations

In table 16 geomechanical parameters have been calculated according to N_{SPT} results in DNO building anticipated area following classical equations and same correlation as above. Please note that average has been set forth in order general conditions for mat foundation design are considered, although both lower and higher figures may be found.

Geotechnical unit	Depth (m)	N_{SPT} Avg	ϕ (º)	γ sat (Tn/m³)	C' / Cu (Kg/cm²)	E (Kg/cm²)
GU2	>0.80	24	39 - 51	*1.56 – 1.80*	0.44 – 1.92	385

Table 16: Geomechanical parameters for shallow foundation design (mat foundation).

GU1 (calculated for Substation location proposed)

$q_{allowable}$ (kg/cm^2) → 2.20 kg/cm^2)
Max Settlement (cm) → 0.50

Table 17: Calculated bearing capacity

Above results are for orientation purposes only as have been calculated for a 2.5 x 4.0m mat foundation embedded 0.25 m into a 5.0 m thick layer. Calculations are based upon notes and comments throughout the document and those should be taken into account when designing bearing structures.

4.3.4. Safety factor analysis

Main aim associated to safety factor is to prevent the structure behavior acting near the failure region. This is done in order to anticipate any unexpected strength not planned in the project or wrong information within the models used.

$$Fs = Q_h/Q_{adm}$$

Safety factor recommended in the bearing capacity equation is SF=3 (Terzagui).

4.3.5. Basic Seismic Data

Volcanic and seismic activity in Saint Lucia is monitored by the Seismic Research Unit at the University of the West Indies in Trinidad and Tobago. A continuous seismic monitoring system was established in Saint Lucia in 1982. Saint Lucia is not located in a critical earthquake zone

Saint Lucia lies in a transition zone where the rate of seismic activity is increasing. The Seismic Research Centre reports that there have been at least five swarms of shallow earthquakes in Saint Lucia in the last 100 years. They occurred in 1906, 1986, 1990, 1999 and 2000. At least three of these swarms seem to have been triggered by a larger tectonic earthquake. The last tectonic earthquake of note was of magnitude 7.75 in 1953 and caused partial collapse of buildings previously damaged by fire and caused some damage to buildings in the capital city of Castries.

The design basic earthquake (DBE) would be an event with an annual probability of exceedance of 0.0021which is equivalent to a 10% chance of exceedance over a 50-year period. The predicted mean plus one standard deviation peak horizontal ground acceleration value for this event, would lie between 10% and 15% of gravity (Shepherd, et al 1987a). In order to estimate the ground motion at site, a suitable distance-attenuation relationship is required.

To date, there has not been sufficient strong motion data recorded to allow a regional attenuation relationship to be developed for Saint Lucia. Therefore, relationships developed for other parts of the world have been used to estimate a range of possible values for peak ground acceleration. The predicted peak horizontal ground acceleration for the two postulated MCE events ranges from 0.10 g to 0.37 g, depending on the magnitude and the attenuation law assumed. The mean value is 0.23 g.

It is recommended that the design earthquake should be assumed to be within zone 3 with a design earthquake of 7.5 at a distance of 25 to 100 kilometres.

4.3.6. Soil Liquefaction

Soil liquefaction describes a phenomenon whereby a saturated or partially saturated soil substantially loses strength and stiffness in response to an applied stress, usually earthquake shaking or other sudden change in stress condition, causing it to behave like a liquid.

The phenomenon is most often observed in saturated, loose (low density or uncompacted), sandy soils. This is because a loose sand has a tendency to compress when a load is applied; dense sands by contrast tend to expand in volume or 'dilate'.

Report provided by the client, and redacted by Strata Engineering Consultant Ltd. in 2012, suggested this phenomenon to be highly regarded, as they drilled some compact layers of saturated gravelly silty sands at depths ranging from 10.7 to 18.3 m that were likely to liquefaction.

TecsolGeo shallow geotechnical survey, with depths up to 3.80 m maximum, these layers has not been detected. Taking into account seismic background, an earthquake with enough energy to induce this phenomenon can't be ruled out.

Once said this, Strata Engineering Consultant Ltd propose different piles type (H, precast pile,…) up to weathered volcanic bedrock to avoid the effect of liquefaction.

From our point of view, and from data reported, this phenomenon would occur at depths that rarely would affect in ground surface where shallow foundation piles would be driven considering geomechanical parameters proposed for described units.

In the other hand, structures to be constructed doesn't support high loads, and are not classified as residential buildings or others that could involve risk for persons.

In the event that this phenomenon occurs wouldn't entail human damages in this area and material losses could be repaired.

Barcelona, as of August 28th, 2017.

Ramon Pérez

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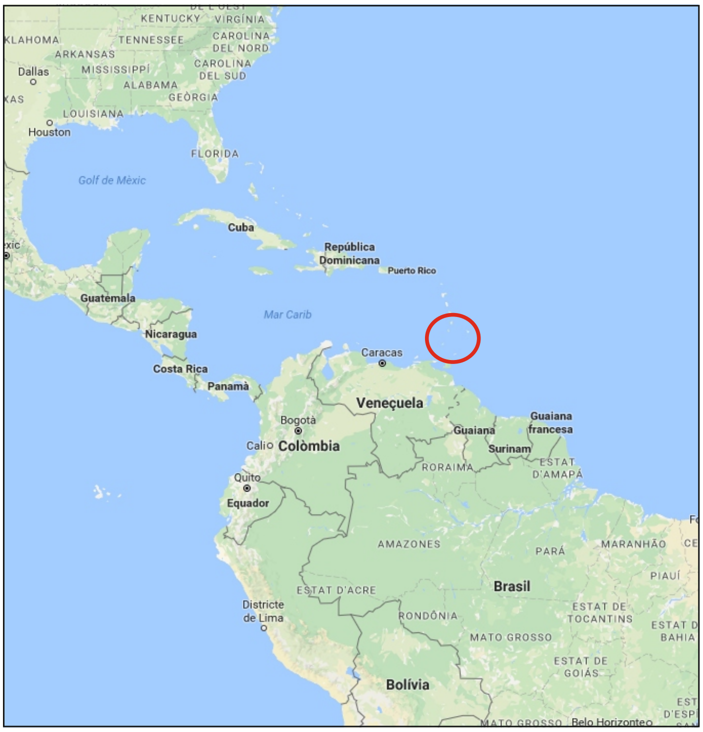
5. CHAPTER V. LIMITATIONS AND EXEPTIONS

1. This report including its conclusions, recommendations and findings should be related to the terms and conditions and the scope of works agreed between the Consultant and the Client. In case PRELIMINARY REPORT is watermarked in any page the information contained in the report shall NOT be considered for final construction design.
2. Both the Executive Summary and the Conclusions and Recommendations sections of this report should not be specifically relied upon out of the content of the whole report and particularly of the context and the development, if any, proposed.
3. Any assessments made in this report are based on the ground conditions as revealed by the exploratory works, which may include boreholes, open pits or any further geotechnical and geophysical techniques, together with the results of any field or laboratory testing undertaken and, where appropriate, other relevant data which may have been obtained for the sites including previous site investigation reports. Any special conditions appertaining to the site which have not been revealed by the abovementioned site investigation may therefore have not been taken into account in the report. The assessment may be subject to amendment in the light of additional information becoming available. Any amendments shall be issued after initial version has been accepted by the Client. This Consultant will inform the Client about any version released after the initial report has been accepted.
4. Any recommendations and interpretations contained in this report represent the consultant's opinion only. This opinion has been arrived at in accordance with currently accepted geotechnical and geophysical industry practices at the time of reporting and based on current legislation in force at that time.
5. Tecsolgeo, Ltd do not hold any responsibility on steel pile's structural calculations including connections between the stud and the superstructure, pile's resistance to external acting forces and pile's own structural settings including pile length.
6. Ramming in and pullout tests shall be carried out whenever it is required in order to obtain actual traction and actual load parameters of soils validating or disregarding any assumptions and recommendations in this report. Lateral load test must be carried out on site in order to validate working hypothesis. Both horizontal and vertical stress shall be accounted for in all tests.
7. Estimated refusal depths are for orientation purposes. Only locations where dynamic percussion light tests (DPL) tests have been carried out hold actual information on refusal depth. Refusal depths different from those obtained in this report shall be conveniently determined.
8. Driving machine shall be powered enough to achieve desired ramming depth.
9. Final studs (stain steel piles) design and length is the solely responsibility of The Client.
10. In no case should studs be hammered until upper end is deformed. If required, upper excess stud length shall be cut off.
11. Should further assessment on corrosion of studs is needed, additional soil samples shall be tested and full corrosivity tests performed.

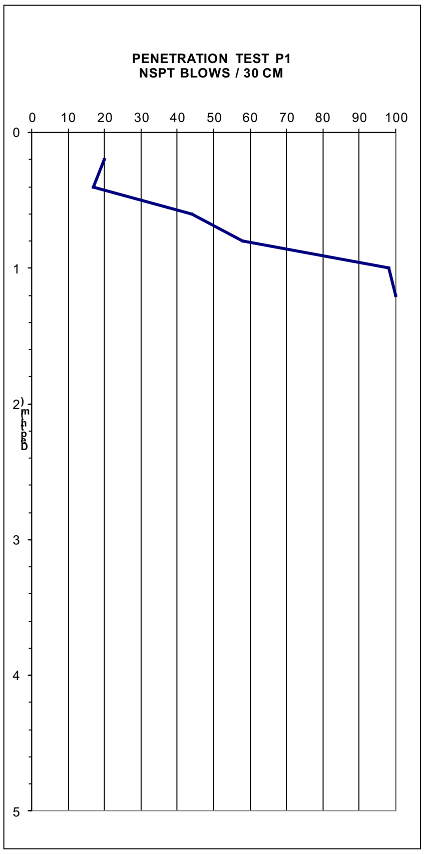
12. Where the data available from previous site investigation reports, supplied by the Client, have been used, it has been assumed that the information is correct. No responsibility can be accepted by the Consultant for inaccuracies within the data supplied.
13. The opinion of possible configuration of strata between or beyond exploratory holes, pit locations or such, or on the possible presence of features based on visual, verbal or published evidence, is for guidance only and no liability can be accepted for the accuracy.
14. Comments on groundwater conditions are based on observations made at the time of the investigation unless otherwise stated. It has to be born I mind that groundwater levels vary due to seasonal or other effects.
15. The copyright in this report and other plans and documents prepared by the Consultant is owned by him and no such report, plan or document may be reproduced, published or adapted without his written consent. Complete copies of this report may, however, be made and distributed by the Client as an expedient in dealing with matters related to its commission.
16. This report is prepared and written in the context of the proposals stated in the introduction to this report and should not be used in a differing context. Furthermore, new information, improved practices and legislation may necessitate an alteration to the report in whole or in part after its submission. Therefore, with any change in circumstances or after the expiry of one year from the date of the report, the report should be referred to the Consultant for re-assessment and, if necessary, re-appraisal.
17. Field works have been undertaken by Tecsòl, SL (Spanish NIF: B61847091) using own equipment, devices and employees duly registered in the Spanish Social Security System. Laboratory works have been partially subcontracted to laboratories located in Spain.
18. Survey activities devoted to bearing capacity and allowable settlements modeling (namely DPL and trial Pits) have reached up to 3.80 m depth only. Whatever the materials and structures beyond that limit are, they may have not been taken into consideration for bearing capacity and settlements analysis of shallow foundations. Calculated bearing capacity and allowable settlement model are for information only; design figures should be taken from each individual test undertaken on site.

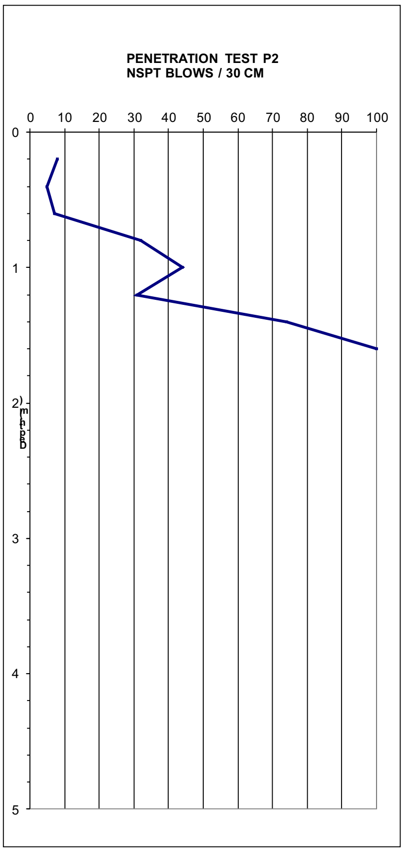
6. CHAPTER VI. ANEXES

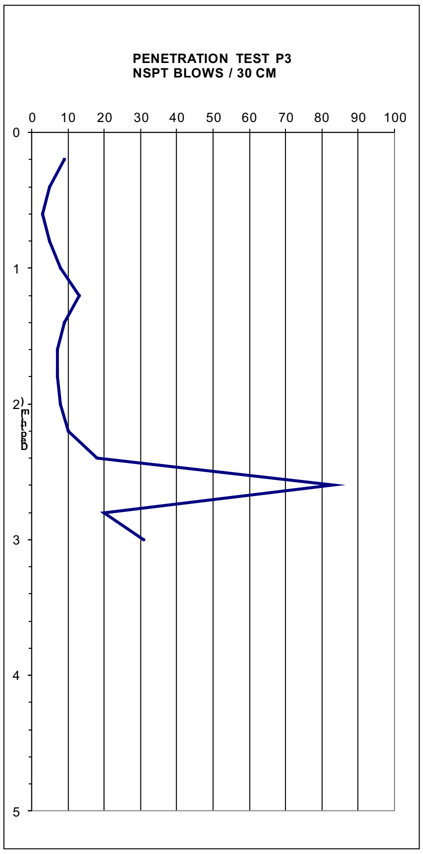
Annex 1: Site Location Map

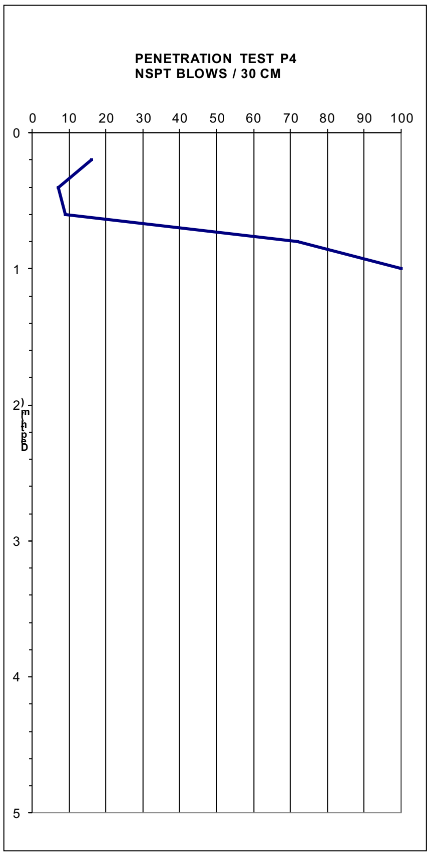


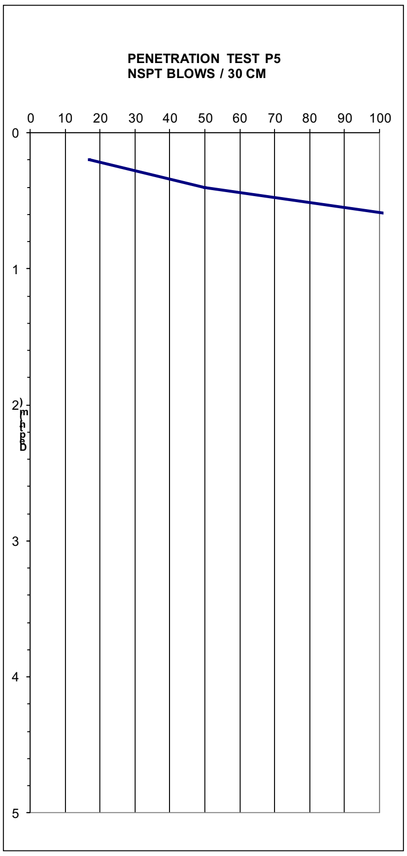
Annex 2: DPL plots

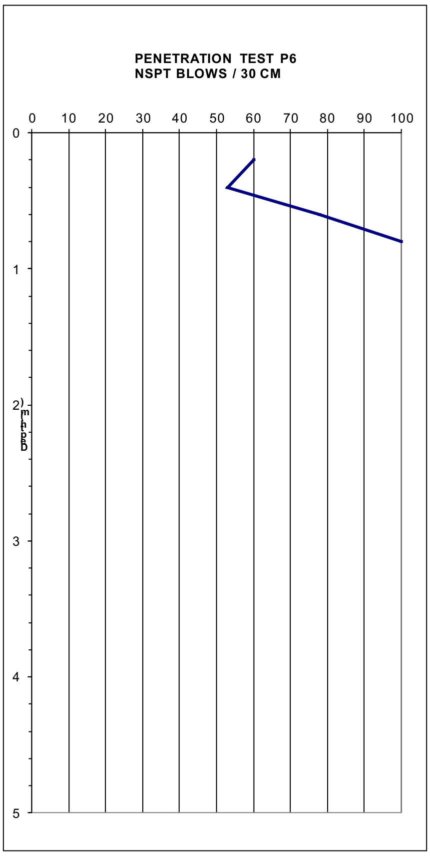


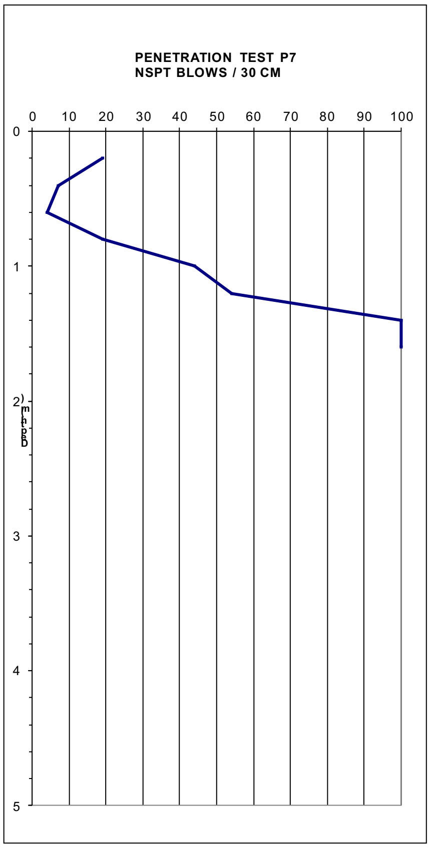


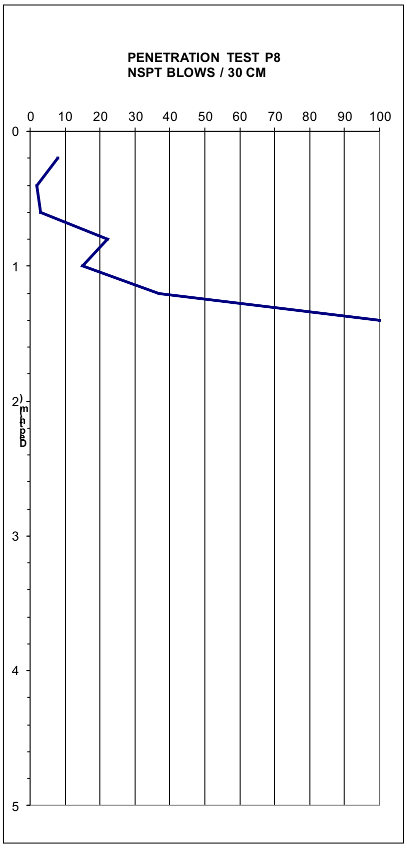









PENETRATION TEST P7
NSPT BLOWS / 30 CM
0 10 20 30 40 50 60 70 80 90 100
0 1 2 3 4 5
Depth (m)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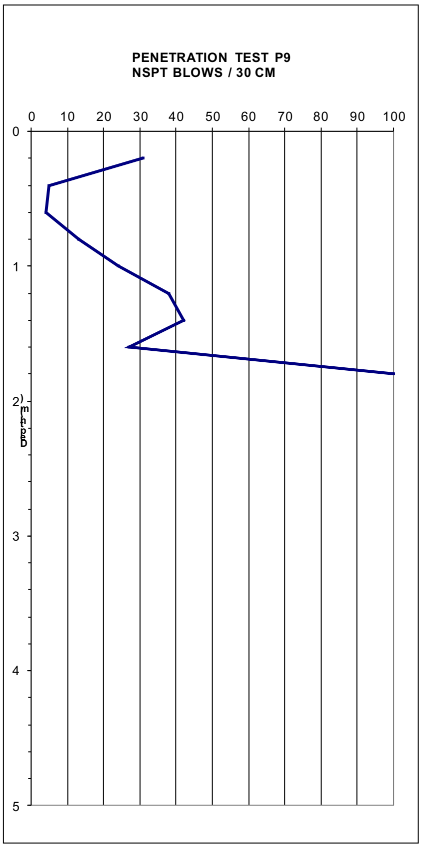
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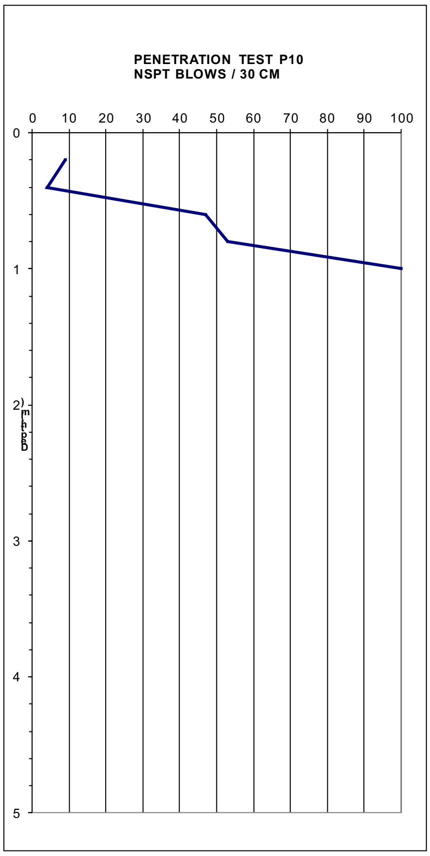
Depth (m)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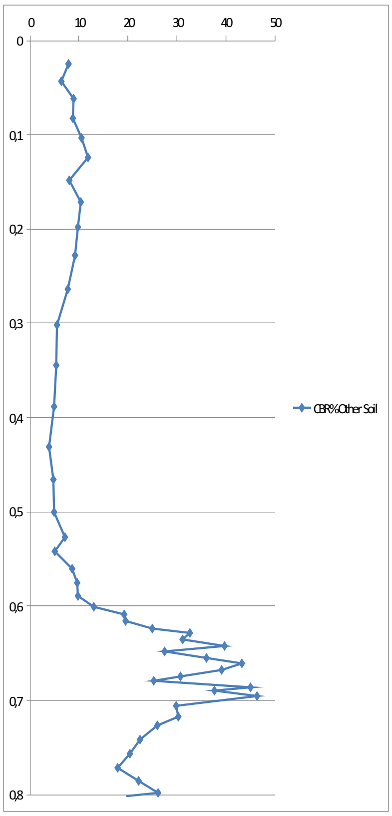
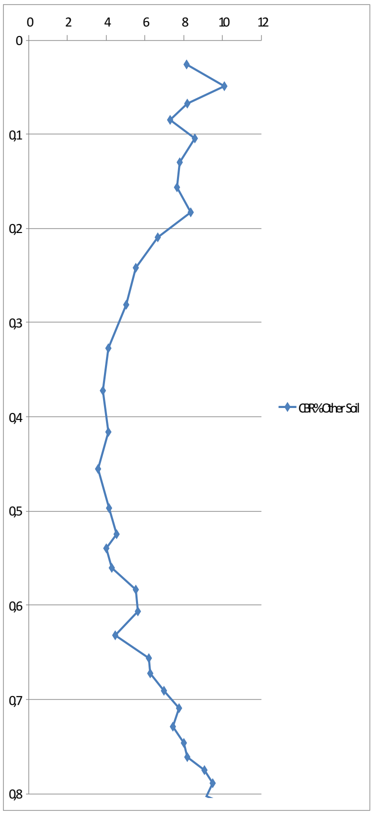
0 1 2 3 4 5
Depth (m)

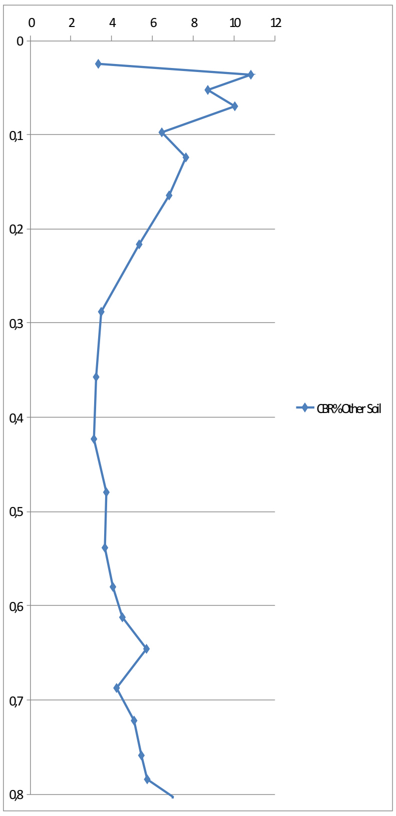
Annex 3: DCL / CBR plots



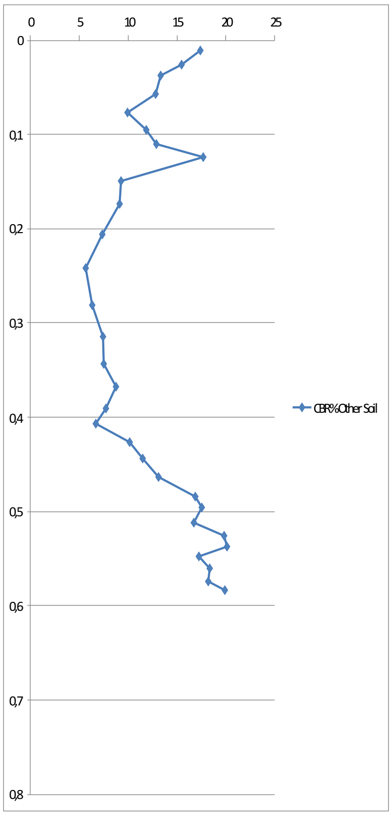
CBR 1



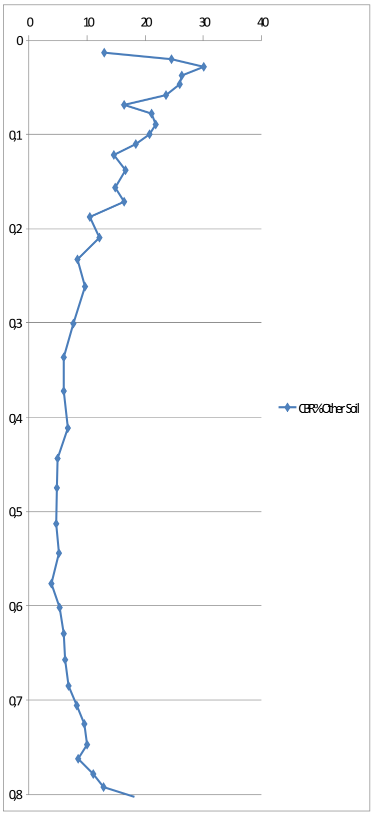
CBR 2



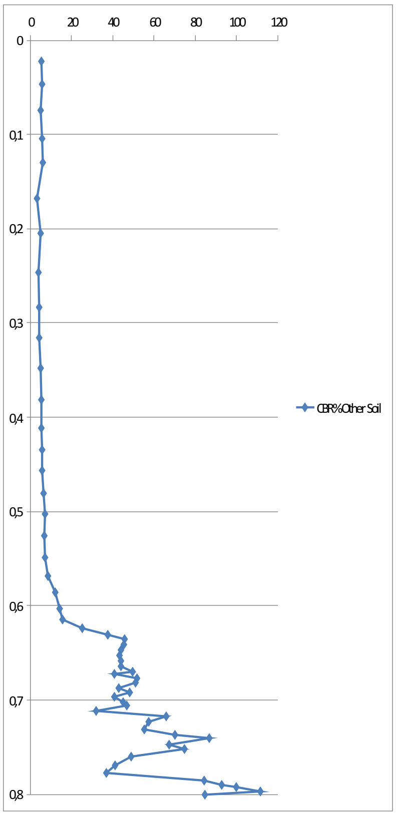
CBR 3



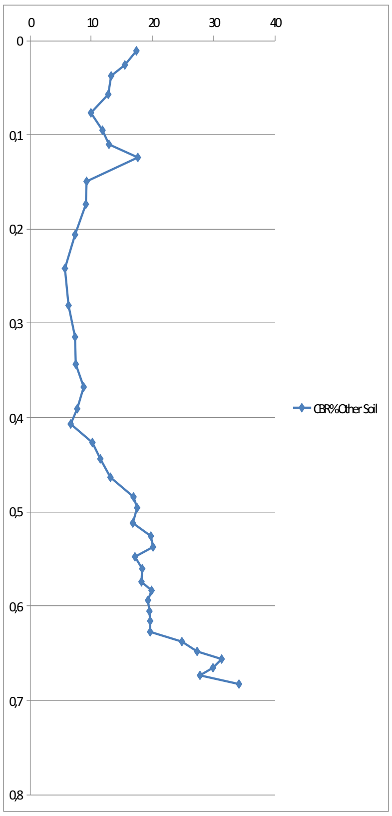
CBR 4



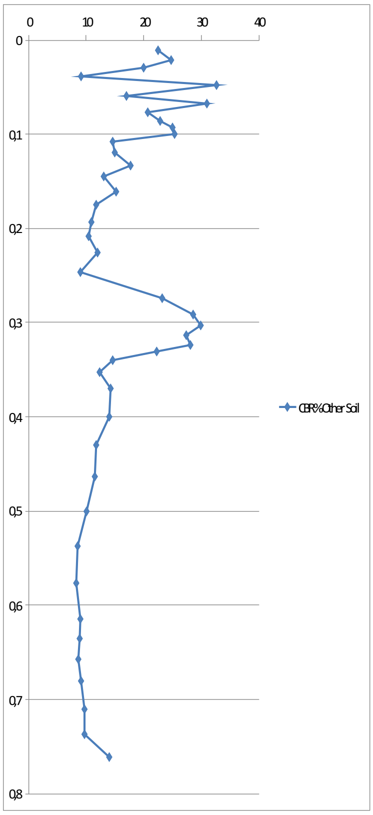
CBR 5



CBR 6


0
10
20
30
40
0
0,1
0,2
0,3
0,4
0,5
0,6
0,7
0,8
CBR% Other Soil

CBR 7


0
10
20
30
40
0
0,1
0,2
0,3
0,4
0,5
0,6
0,7
0,8
CBR% Other Soil

CBR 8

Annex 4: Trial Pit Testing Log

TRIAL PIT TP1
Date: 27/07/17
Project: IG62717

PV Geotechnical Survey in La Tourney (Saint Lucia)

GW	SOIL STRATUM: DEPTH (cm)	SOIL STRATUM: LEVEL (cm)	SOIL STRATUM: LEGEND	SAMPLE	LITHOLOGY DESCRIPTION
	20, 40, 60, 80	100			Topsoil. Silty clay. Dark browny grey and high organic content.
	100, 120, 140, 160, 180, 200, 220, 240, 260, 280	290		MTP1	Browny Slightly gravelly sands. Compact to dense sandy layers.
	300, 320, 340, 360, 380, 400, 420, 440				TRIAL PIT COMPLETE AT 2.90 m



NOTES:

X: 718881; Y: 1519531
https://youtu.be/3MRUG9DT3TA

TRIAL PIT TP2
Date: 27/07/17
Project: IG62717

PV Geotechnical Survey in La Tourney (Saint Lucia)

GW	SOIL STRATUM: DEPTH (cm)	SOIL STRATUM: LEVEL (cm)	SOIL STRATUM: LEGEND	SAMPLE	LITHOLOGY DESCRIPTION
	20, 40, 60	80		MTP2	Topsoil. Silty clay. Dark browny grey and high organic content.
	80, 100, 120, 140	160			Browny Slightly gravelly sands. Compact to dense sandy layers.
	160, 180, 200, 220, 240, 260, 280	300			Basalt bedrock.
	300, 320, 340, 360, 380, 400, 420, 440				TRIAL PIT COMPLETE AT 3.00 m



NOTES:

X: 719952; Y: 1519600
https://youtu.be/lt50AdYNxGk

TRIAL PIT TP3
Date: 27/07/17
Project: IG62717

PV Geotechnical Survey in La Tourney (Saint Lucia)

GW	SOIL STRATUM DEPTH (cm)	LEVEL (cm)	LEGEND	SAMPLE	LITHOLOGY DESCRIPTION
	20, 40, 60, 80	80			Topsoil. Silty clay. Dark browny grey and high organic content.
	100, 120	130			Browny Slightly gravelly sands. Compact to dense sandy layers.
	140, 160, 180, 200, 220	220			Basalt bedrock.
	240, 260, 280, 300, 320, 340, 360, 380, 400, 420, 440				TRIAL PIT COMPLETE AT 2.20 m



NOTES:
X: 720046; Y: 1519597
https://youtu.be/VBWFQ2j3-q8

TRIAL PIT TP4
Date: 27/07/17
Project: IG62717

PV Geotechnical Survey in La Tourney (Saint Lucia)

GW	SOIL STRATUM DEPTH (cm)	LEVEL (cm)	LEGEND	SAMPLE	LITHOLOGY DESCRIPTION
	20, 40, 60, 80	80			Topsoil. Silty clay. Dark browny grey and high organic content.
	100, 120, 140, 160, 180, 200, 220, 240, 260, 280, 300, 320, 340, 360, 380	390			Browny Slightly gravelly sands. Compact to dense sandy layers.
	400, 420, 440				TRIAL PIT COMPLETE AT 3.90 m



NOTES:
X: 720524; Y: 1519867
https://youtu.be/K3VUbjAOSww

TRIAL PIT TP5
Date: 27/07/17
Project: IG62717

PV Geotechnical Survey in La Tourney (Saint Lucia)

GW	DEPTH (cm)	LEVEL (cm)	SAMPLE	LITHOLOGY DESCRIPTION
	0–100	100		Topsoil. Silty clay. Dark browny grey and high organic content.
	100–290	290	MTP5	Browny Slightly gravelly sands. Compact to dense sandy layers.
				TRIAL PIT COMPLETE AT 2.90 m



NOTES:

X: 720480; Y: 1519706
https://youtu.be/PTelZjyMiyA

TRIAL PIT TP6
Date: 27/07/17
Project: IG62717

PV Geotechnical Survey in La Tourney (Saint Lucia)

GW	DEPTH (cm)	LEVEL (cm)	SAMPLE	LITHOLOGY DESCRIPTION
	0–80	80		Topsoil. Silty clay. Dark browny grey and high organic content.
	80–390	390		Browny Slightly gravelly sands. Compact to dense sandy layers.
				TRIAL PIT COMPLETE AT 3.90 m



NOTES:

X: 720530; Y: 1519595
https://youtu.be/alTi2QOxMoc

Annex 5: Laboratory Report

GEOTÈCNIA I CONTROL DE QUALITAT S.A.

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Telf. 93 574 93 91-Fax. 93 574 93 92

CUSTOMER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L. NIF: B61847091	SAMPLE Nº:	.2017/4262		
WORK:	SANTA LUCIA	REFERENCE:	TP1 (2.50m)		
		SAPLE TYPE:	■ INALTERADA	☐ REMOLDEJADA	
WORK Nº:	728	TEST TYPE:	☐ CD	☐ CU	■ UU
SUBMITTER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L.	DATE REPORT:	11/08/17	REPORT CODE:	2017/8992

RESULTS REPORT

DETERMINATION OF SHEAR RESISTANT PARAMETERS OF A SOIL SAMPLE

UNE 103401:1998

MASSES AND DIMENSIONS OF THE CUTTING BOX

MASSES

			CAIXA 1	CAIXA 2	CAIXA 3
Mass top half box	m_c (g)	=	1008.56	1008.56	1008.56
Mass base plate	m_b (g)	=	856.08		

DIMENSIONS

SQUARE BOX

NORMAL PRESSURE			$1 kg/cm^2$	$2 kg/cm^2$	$3 kg/cm^2$
Internal dimension	L_1 (mm)	=	60.00	60.00	60.00
Internal dimension	L_2 (mm)	=	60.08	60.08	60.08
Height base	h_b (mm)	=	20.00		
Height top half box	h_c (mm)	=	25.20	25.20	25.20
Initial area test piece	$A=L_1*L_2(mm^2)$	=	3604.80	3604.80	3604.80
Vol. ini. test piece	$V = A*h/1000\ (cm^3)$	=	81.47	86.15	82.55

(h = initial height test piece)

CIRCULAR BOX

NORMAL PRESSURE			$1 kg/cm^2$	$2 kg/cm^2$	$3 kg/cm^2$
∅ inner box	D (mm)	=	50	50	50
Height top half box	h_c (mm)	=	25	25	25
Initial area test piece	$A=(\pi*D^2)/4\ (mm^2)$	=	1963.49	1963.494	1963.49
			44.37	46.93	44.96

PREPARATION OF THE SAMPLE

COHESIVE SOIL			SÒLS NO COHESIUS			
UNCHANGED	COMPACTED (SATURATED / NON-SATURATED)		DRY SAND	COMPACT DRY SAND	SATURATED SAND	MEDIUM SATURATED SAND
Hacked	Dry density ρ_d (g/cm^3)	=	Poured	Dynamic compaction	Boiled	Dynamic compaction
				Stactic compaction		
	Energy E (J/cm^3)	= 0.583		Vibration	Boiled	
	Lím. plàstic	=		Poured		

DATA OF THE ASSEMBLY AND OF THE SAMPLE

	NORMAL PRESSURE		$1 kg/cm^2$	$2 kg/cm^2$	$3 kg/cm^2$
Scroll speed	v(mm/min)	=	1.00	1.00	1.00
Distance from the top edge of the box to the base plate	h_1 (mm)	=	40.50	40.50	40.50
Distance from the top edge of the box in the porous plate sup. (cohesive soil)	h_2 (mm)	=	6.00	5.00	6.00
Distance from the top edge of the box in the slotted plate (non-cohesive soils)	h_3 (mm)	=			
Total height of the plates used in the essay (without the SLOTTED SUP in COHESIVE SOILS)	h_t (mm)	=	11.90	11.60	11.60
Initial height of the sample (cohesive soil)	$h(mm)=h_1-h_2-h_t$	=	22.60	23.90	22.90
Initial height of the sample (non-cohesive soils)	$h(mm)=h_1-h_3-h_t$	=			
Distance from the top edge of the piston filling the top edge of the box	h_4 (mm)	=			
Distance from the top edge of the piston filling the top edge of the box with the yoke of loads and in balance	h_5 (mm)	=			
Settlement due to the yoke	h_4-h_5(mm)	=			
Wet mass of the initial sample	m_{hi} (g)	=	73.30	70.60	68.00
Dry mass of the test piece	m_{di} (g)	=	58.32	56.17	54.10
Mass wet the bottom of the sample	m_{hf} (g)	=	70.7	68.1	65.6
Settlement consolidation	h_c(mm)	=			

MOISTURE AND DENSITIES

NORMAL PRESSURE			$1 kg/cm^2$	$2 kg/cm^2$	$3 kg/cm^2$
Initial moisture	$W_i(\%)=100*(m_{hi}-m_{di})/m_{di}$	=	25.69	18.84	18.36
Final moisture	$W_f(\%)=100*(m_{hf}-m_{df})/m_{df}$	=	21.26	16.48	17.35
Index of initial voids	$e_i=(\rho_s/\rho_d)-1$	=	---	---	---
Index of voids	$e=e_{i+}(\Delta h(1+e_i)/h)$	=	---	---	---

(Δh = change of height -mm- sample)

NORMAL PRESSURE		$1 kg/cm^2$	$2 kg/cm^2$	$3 kg/cm^2$
Dens. apar. ini.	$\rho i\ (g/cm^3) = m_{hi}/V=$	1.65	1.50	1.51
Dens. seca ini.	$\rho_d\ (g/cm^3)=m_{di}/V=$	1.31	1.20	1.20
Dens. apar. fin	$\rho\phi\ (g/cm^3) = m_{hi}/V=$	1.59	1.45	1.46
Particle density	$\rho_s=$	---		
Initial saturation	$S_i(\%) = (W_i*\rho_s)/e_i=$	---	---	---

MOISTURE AND DENSITIES: AVERAGE VALUES

Initial moisture	$W_i(\%)=100*(m_{hi}-m_{di})/m_{di}$	=	**20.96**
Final moisture	$W_f(\%)=100*(m_{hf}-m_{df})/m_{df}$	=	**18.36**
Index of initial voids	$e_i=(\rho_s/\rho_d)-1$	=	**---**
Índex de buits	$e=e_{i+}(\Delta h(1+e_i)/h)$	=	**---**

(Δh = change of height -mm- sample)

Dens. apar. ini.	$\rho\iota\ (g/cm^3) = m_{hi}/V=$	**1.56**
Dry density	$\rho_d\ (g/cm^3)=m_{di}/V=$	**1.24**
Dens. apar. fin	$\rho\phi\ (g/cm^3) = m_{hi}/V=$	**1.50**
Particle density	$\rho_s=$	**---**
Initial saturation level	$S_i(\%) = (W_i*\rho_s)/e_i=$	**---**

EL DIRECTOR DEL LABORATORI

JAVIER VICENTE

EL TÈCNIC RESPONSABLE

GUILLEM RODRÍGUEZ

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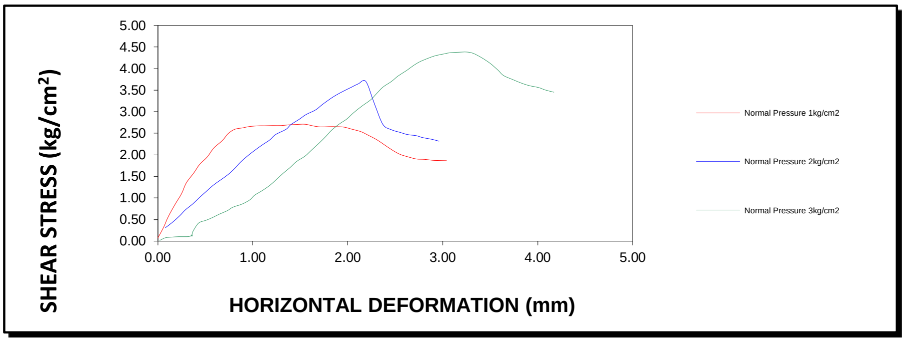
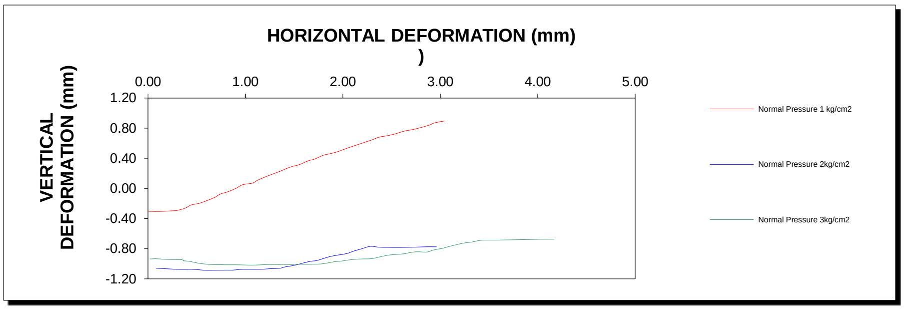
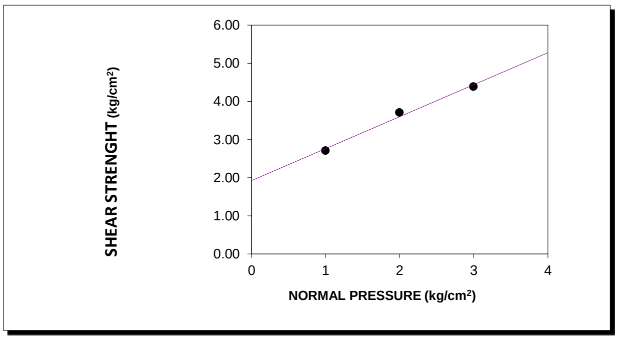
Telf. 93 574 93 91-Fax. 93 574 93 92

CUSTOMER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L. NIF: B61847091	SAMPLE Nº:	.2017/4262
WORK:	SANTA LUCIA	REFERENCE:	TP1 (2.50m)
		SAPLE TYPE:	☒ INALTERADA ☐ REMOLDEJADA
WORK Nº:	728	TEST TYPE:	☐ CD ☐ CU ☒ UU
SUBMITTER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L.	DATE REPORT:	11/08/17 REPORT CODE: 2017/8992







Normal pressure (kg/cm^2)	1	2	3
Shear strenght (kg/cm^2)	2.71	3.70	4.39

COHESION:

1.9234	**kg/cm^2**	**188.63**	**kPa**

INTERNAL ANGLE OF FRICTION **φ= 39.96º**

OBSERVATIONS:

OVER CONSOLIDATED SAND

EL DIRECTOR DEL LABORATORI

JAVIER VICENTE

EL TÈCNIC RESPONSABLE

GUILLEM RODRÍGUEZ

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Telf.: 93 574 93 91
email: laboratori@gcq.es

CLIINT:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L. - NIF: ESB61847091
WORK:	SANTA LUCIA
Nº WORK:	728
SUBMITTER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L.

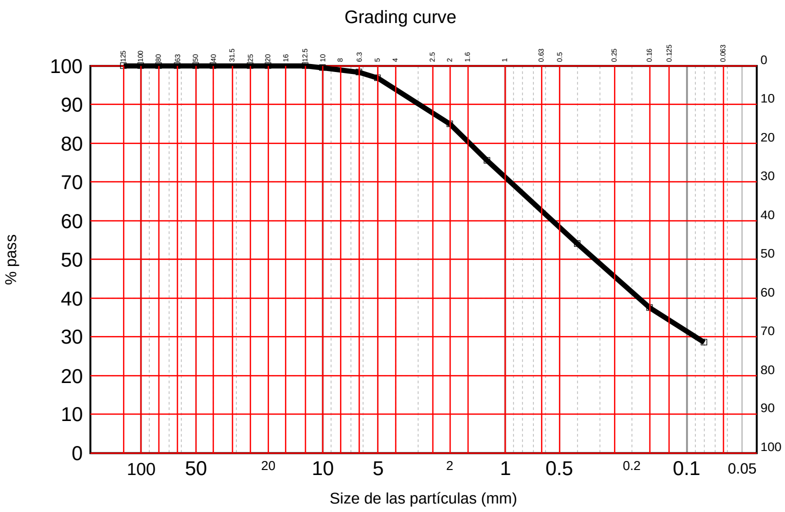
SAMPLE Nº:	.2017/4262		
LOCALITZACIÓ:	TP1 (2.50m)		
DATE COLLECTION:	31/07/2017		
DATE REPORT:	11/08/2017	**REPORT CODE:**	2017/8997

RESULTS REPORT

Method of analysis	Washing and sifting

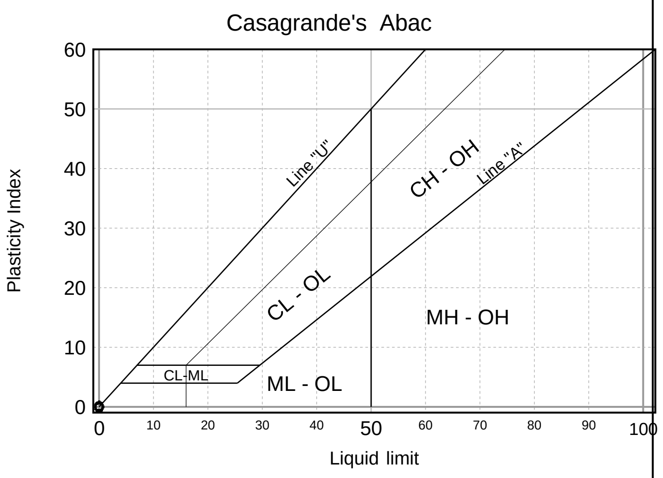
Essay 0188 - SOIL. GRANULOMETRY. 103101/NLT104 S/UNE 103101 ó NLT104

Sieve (mm)	125	100	80	63	50	40	25	20	12.5	10	6.3	5	2	1.25	0.4	0.16	0.08
Pass (%)	100	100	100	100	100	100	100	100	100	99	98	97	85	75	54	37	28.6



Particle size distribution S/ASTM-D 2487/00

Blocs		More de 300 mm.	0.0%
Boulders		De 75 a 300 mm.	0.0%
Gravels (4.2%)	Thick	De 19 a 75 mm.	0.0%
	Thin	De 4.75 a 19 mm.	4.2%
Sands (69.0%)	Thick	De 2 a 4.75 mm.	10.9%
	medium	De 0.425 a 2 mm.	30.2%
	Thin	De 0.075 a 0.425 mm.	27.9%
Silts and clays		Less de 0.075 mm.	26.8%



ATTEMBERG'S LÍMITS S/UNE 103.103:94 y UNE 103.104:93

Liquid limit	Not available
Plastic limit	No Plastic
Plasticity Index	No Plastic

LABORATORI DIRECTOR

Firmado digitalmente por: Javier Vicente Minguez Geotecnia i Control de Qualitat SA

JAVIER VICINTE MÍNGUEZ

RESPONSIBLE TECHNICIAN

Firmado digitalmente por: Guillem Rodriguez Perello Geotecnia i Control de Qualitat SA

GUILLEM RODRÍGUEZ PERELLÓ

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CLIINT:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L. - NIF: ESB61847091	**SAMPLE Nº:**	.2017/4262		
WORK:	SANTA LUCIA	**LOCALITZACIÓ:**	TP1 (2.50m)		
Nº WORK:	728	**DATE COLLECTION:**	31/07/2017		
SUBMITTER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L.	**DATE REPORT:**	11/08/2017	**REPORT CODE:**	2017/8998

RESULTS REPORT

DETERMINATION OF DINSITY FROM A SOIL UNE 103.301-94		
WET DINSITY	g/cm³	**1.564**
DRY DINSITY	g/cm³	**1.302**

SULPHATES. ACCORDING TO UNE 103-201-96		
% SO3	%	**0.0200**
SO3	mg/kg	**200**
SO4	mg/kg	**240**

DETERMINATION OFS CHLORIDES IN SOIL		
CHLORIDES	mg/kg	**35.1**

DETERMINATION OF PH IN SOIL	
PH	**6.99**

DETERMINATION OF CONDUCTIVITY FROM A SOIL		
CONDUCTIVITY	µS/cm	**921**

DESCRIPTION OF THE SAMPLE
CONSOLIDATED CLAYEY SAND

ESSAYS
DINSITY - TALL UU - SULPHATES - CHLORIDES - PH - CONDUCTIVITY - GRANULOMETRY - LÍMITS ATTERBERG

LABORATORI DIRECTOR

Firmado digitalmente por:
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Geotecnia i Control de Qualitat SA

JAVIER VICINTE MÍNGUEZ

RESPONSIBLE TECHNICIAN

Firmado digitalmente por:
Guillem Rodriguez Perello
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CUSTOMER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L. NIF: B61847091	SAMPLE Nº:	.2017/4263
WORK:	SANTA LUCIA	REFERENCE:	TP2 (0.50m)
		SAPLE TYPE:	☐ INALTERADA ☐ REMOLDEJADA
WORK Nº:	728	TEST TYPE:	☐ CD ☐ CU ☐ UU
SUBMITTER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L.	DATE REPORT:	11/08/17 REPORT CODE: 2017/8993

RESULTS REPORT

DETERMINATION OF SHEAR RESISTANT PARAMETERS OF A SOIL SAMPLE

UNE 103401:1998

MASSES AND DIMENSIONS OF THE CUTTING BOX

MASSES

			CAIXA 1	CAIXA 2	CAIXA 3
Mass top half box	m_c (g)	=	1008.56	1008.56	1008.56
Mass base plate	m_b (g)	=	856.08		

DIMENSIONS

SQUARE BOX

NORMAL PRESSURE			$1kg/cm^2$	$2kg/cm^2$	$3kg/cm^2$
Internal dimension	L_1 (mm)	=	60.00	60.00	60.00
Internal dimension	L_2 (mm)	=	60.08	60.08	60.08
Height base	h_b (mm)	=	20.00		
Height top half box	h_c (mm)	=	25.20	25.20	25.20
Initial area test piece	$A=L_1{*}L_2(mm^2)$	=	3604.80	3604.80	3604.80
Vol. ini. test piece	$V = A{*}h/1000\ (cm^3)$	=	85.43	87.60	89.04
(h = initial height test piece)					

CIRCULAR BOX

NORMAL PRESSURE			$1kg/cm^2$	$2kg/cm^2$	$3kg/cm^2$
∅ inner box	D (mm)	=	50	50	50
Height top half box	h_c (mm)	=	25	25	25
Initial area test piece	$A=(\pi{*}D^2)/4\ (mm^2)$	=	1963.49	1963.494	1963.49
			46.53	47.71	48.50

PREPARATION OF THE SAMPLE

COHESIVE SOIL			SÒLS NO COHESIUS			
UNCHANGED	COMPACTED SATURATED	COMPACTED NON-SATURATED	DRY SAND	COMPACT DRY SAND	SATURATED SAND	MEDIUM SATURATED SAND
Hacked	Dry density ρ_d (g/cm^3) =		Poured	Dynamic compaction	Boiled	Dynamic compaction
	Energy E (J/cm^3) = 0.583			Stactic compaction		
				Vibration	Boiled	
	Lím. plàstic =			Poured		

DATA OF THE ASSEMBLY AND OF THE SAMPLE

	NORMAL PRESSURE		$1kg/cm^2$	$2kg/cm^2$	$3kg/cm^2$
Scroll speed	v(mm/min)	=	1.00	1.00	1.00
Distance from the top edge of the box to the base plate	h_1 (mm)	=	40.50	40.50	40.50
Distance from the top edge of the box in the porous plate sup. (cohesive soil)	h_2 (mm)	=	4.90	4.60	4.20
Distance from the top edge of the box in the slotted plate (non-cohesive soils)	h_3 (mm)	=			
Total height of the plates used in the essay (without the SLOTTED SUP in COHESIVE SOILS)	h_t (mm)	=	11.90	11.60	11.60
Initial height of the sample (cohesive soil)	$h(mm)=h_1-h_2-h_t$	=	23.70	24.30	24.70
Initial height of the sample (non-cohesive soils)	$h(mm)=h_1-h_3-h_t$	=			
Distance from the top edge of the piston filling the top edge of the box	h_4 (mm)	=			
Distance from the top edge of the piston filling the top edge of the box with the yoke of loads and in balance	h_5 (mm)	=			
Settlement due to the yoke	h_4-h_5(mm)	=			
Wet mass of the initial sample	m_{hi} (g)	=	77.90	81.00	79.70
Dry mass of the test piece	m_{di} (g)	=	54.74	56.92	56.00
Mass wet the bottom of the sample	m_{hf} (g)	=	75.8	78.9	77.6
Settlement consolidation	h_c(mm)	=			

MOISTURE AND DENSITIES

NORMAL PRESSURE			$1kg/cm^2$	$2kg/cm^2$	$3kg/cm^2$
Initial moisture	$W_i(\%)=100{*}(m_{hi}-m_{di})/m_{di}$	=	42.31	46.00	42.75
Final moisture	$W_f(\%)=100{*}(m_{hf}-m_{df})/m_{df}$	=	38.56	41.49	40.48
Index of initial voids	$e_i=(\rho_s/\rho_d)-1$	=	---	---	---
Index of voids	$e=e_{i+}(\Delta h(1+e_i)/h)$	=	---	---	---
(Δh = change of height -mm- sample)					

NORMAL PRESSURE		$1kg/cm^2$	$2kg/cm^2$	$3kg/cm^2$
Dens. apar. ini.	$\rho i\ (g/cm^3) = m_{hi}/V=$	1.67	1.70	1.64
Dens. seca ini.	$\rho_d\ (g/cm^3)=m_{di}/V=$	1.18	1.19	1.15
Dens. apar. fin	$\rho\phi\ (g/cm^3) = m_{hi}/V=$	1.63	1.65	1.60
Particle density	$\rho_s=$	---		
Initial saturation	$S_i(\%) = (W_i{*}\rho_s)/e_i=$	---	---	---

MOISTURE AND DENSITIES: AVERAGE VALUES

Initial moisture	$W_i(\%)=100{*}(m_{hi}-m_{di})/m_{di}$	=	**43.69**
Final moisture	$W_f(\%)=100{*}(m_{hf}-m_{df})/m_{df}$	=	**40.18**
Index of initial voids	$e_i=(\rho_s/\rho_d)-1$	=	**---**
Índex de buits	$e=e_{i+}(\Delta h(1+e_i)/h)$	=	**---**
(Δh = change of height -mm- sample)			

Dens. apar. ini.	$\rho\iota\ (g/cm^3) = m_{hi}/V=$	**1.67**
Dry density	$\rho_d\ (g/cm^3)=m_{di}/V=$	**1.17**
Dens. apar. fin	$\rho\phi\ (g/cm^3) = m_{hi}/V=$	**1.63**
Particle density	$\rho_s=$	**---**
Initial saturation level	$S_i(\%) = (W_i{*}\rho_s)/e_i=$	**---**

EL DIRECTOR DEL LABORATORI

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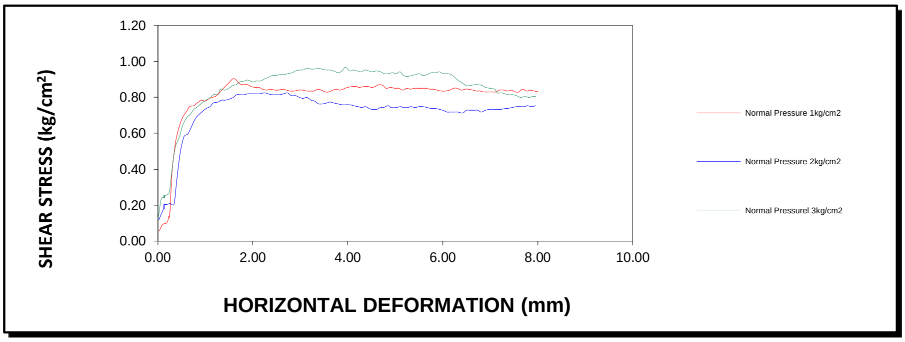
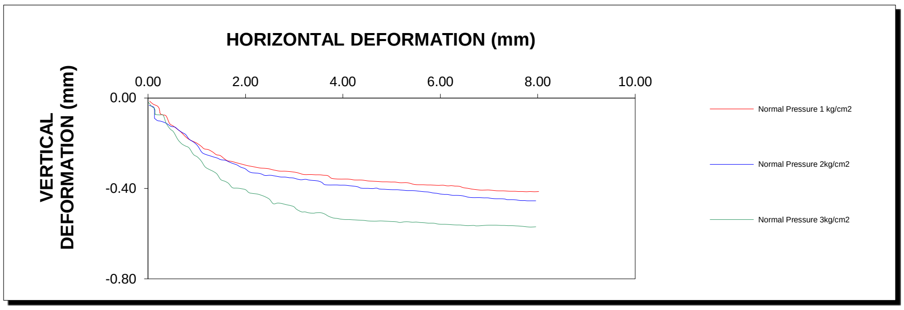
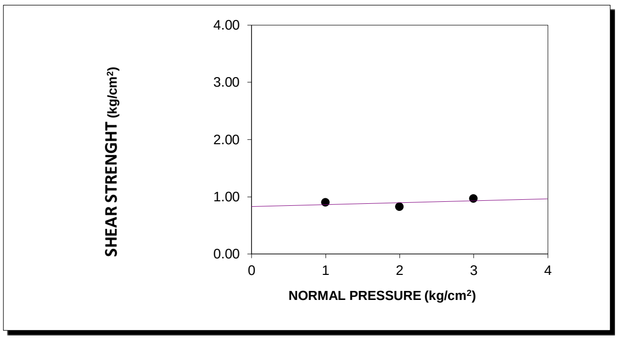
CUSTOMER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L. NIF: B61847091
WORK:	SANTA LUCIA
WORK Nº:	728
SUBMITTER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L.

SAMPLE Nº:	.2017/4263				
REFERENCE:	TP2 (0.50m)				
SAPLE TYPE:	☒ INALTERADA		☐ REMOLDEJADA		
TEST TYPE:	☐ CD	☐ CU	☒ UU		
DATE REPORT:	11/08/17	REPORT CODE:	2017/8993		







Normal pressure (kg/cm^2)	1	2	3
Shear strenght (kg/cm^2)	0.90	0.83	0.97

COHESION:			
0.8319	kg/cm^2	81.58	kPa

INTERNAL ANGLE OF FRICTION φ= **1.90º**

OBSERVATIONS:

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WORK:	SANTA LUCIA	**LOCALITZACIÓ:**	TP2 (0.50m)
Nº WORK:	728	**DATE COLLECTION:**	31/07/2017
SUBMITTER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L.	**DATE REPORT:**	11/08/2017
		REPORT CODE:	2017/8999

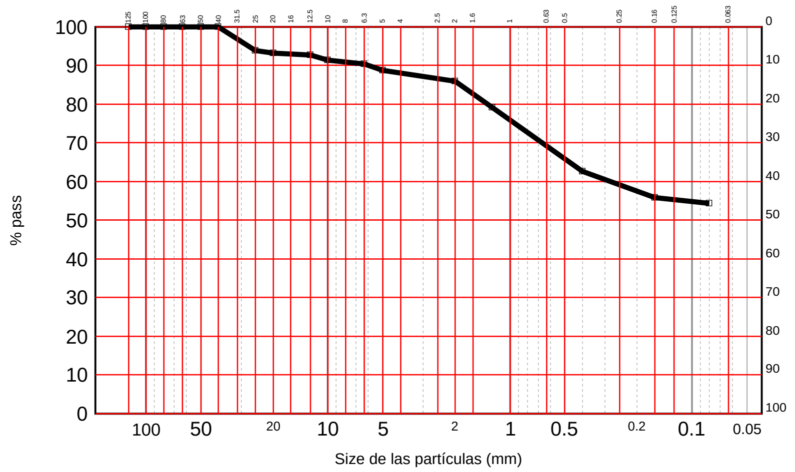
RESULTS REPORT

Method of analysis	Washing and sifting

Essay 0188 - SOIL. GRANULOMETRY. 103101/NLT104 S/UNE 103101 ó NLT104

Sieve (mm)	125	100	80	63	50	40	25	20	12.5	10	6.3	5	2	1.25	0.4	0.16	0.08
Pass (%)	100	100	100	100	100	100	94	93	93	91	90	89	86	79	63	56	54.3

Grading curve



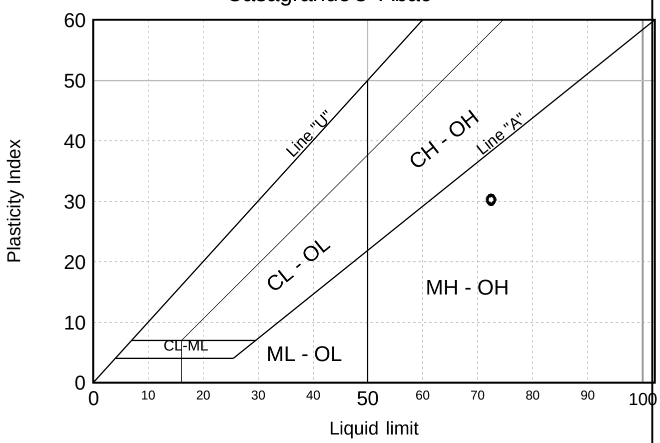
Particle size distribution S/ASTM-D 2487/00

Blocs		More de 300 mm.	0.0%
Boulders		De 75 a 300 mm.	0.0%
Gravels (11.5%)	Thick	De 19 a 75 mm.	6.8%
	Thin	De 4.75 a 19 mm.	4.7%
Sands (37.6%)	Thick	De 2 a 4.75 mm.	2.6%
	medium	De 0.425 a 2 mm.	22.7%
	Thin	De 0.075 a 0.425 mm.	12.3%
Silts and clays		Less de 0.075 mm.	50.9%

ATTEMBERG'S LÍMITS S/UNE 103.103:94 y UNE 103.104:93

Liquid limit	72.39
Plastic limit	42.11
Plasticity Index	30.28

Casagrande's Abac



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WORK:	SANTA LUCIA	**LOCALITZACIÓ:**	TP2 (0.50m)		
Nº WORK:	728	**DATE COLLECTION:**	31/07/2017		
SUBMITTER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L.	**DATE REPORT:**	11/08/2017	**REPORT CODE:**	2017/9000

RESULTS REPORT

DETERMINATION OF DINSITY FROM A SOIL UNE 103.301-94		
WET DINSITY	g/cm^3	**1.594**
DRY DINSITY	g/cm^3	**1.168**

SULPHATES. ACCORDING TO UNE 103-201-96		
% SO_3	%	**0.0120**
SO_3	mg/kg	**120**
SO_4	mg/kg	**144**

DETERMINATION OFS CHLORIDES IN SOIL		
CHLORIDES	mg/kg	**39.2**

DETERMINATION OF PH IN SOIL	
PH	**6.27**

DETERMINATION OF CONDUCTIVITY FROM A SOIL		
CONDUCTIVITY	µS/cm	**619**

DESCRIPTION OF THE SAMPLE
BROWN CLAY WITH FRAGMINTS OF ROOTS

ESSAYS
DINSITY - TALL UU - SULPHATES - CHLORIDES - PH - CONDUCTIVITY - GRANULOMETRY - LÍMITS ATTERBERG

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CUSTOMER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L. NIF: B61847091	SAMPLE Nº:	.2017/4264
WORK:	SANTA LUCIA	REFERENCE:	TP5 (3.00m)
		SAPLE TYPE:	☑ INALTERADA ☐ REMOLDEJADA
WORK Nº:	728	TEST TYPE:	☐ CD ☐ CU ☑ UU
SUBMITTER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L.	DATE REPORT:	11/08/17 REPORT CODE: 2017/8994

RESULTS REPORT

DETERMINATION OF SHEAR RESISTANT PARAMETERS OF A SOIL SAMPLE

UNE 103401:1998

MASSES AND DIMENSIONS OF THE CUTTING BOX

MASSES

			CAIXA 1	CAIXA 2	CAIXA 3
Mass top half box	m_c (g)	=	1008.56	1008.56	1008.56
Mass base plate	m_b (g)	=	856.08		

DIMENSIONS

SQUARE BOX

NORMAL PRESSURE			1kg/cm^2	2kg/cm^2	3kg/cm^2
Internal dimension	L_1 (mm)	=	60.00	60.00	60.00
Internal dimension	L_2 (mm)	=	60.08	60.08	60.08
Height base	h_b (mm)	=	20.00		
Height top half box	h_c (mm)	=	25.20	25.20	25.20
Initial area test piece	$A=L_1*L_2$(mm^2)=		3604.80	3604.80	3604.80
Vol. ini. test piece	V = A*h/1000 (cm^3) =		124.37	0.00	124.37

(h = initial height test piece)

CIRCULAR BOX

NORMAL PRESSURE			1kg/cm^2	2kg/cm^2	3kg/cm^2
∅ inner box	D (mm)	=	50	50	50
Height top half box	h_c (mm)	=	25	25	25
Initial area test piece	A=(π*D^2)/4 (mm^2)	=	1963.49	1963.494	1963.49
			67.74	0.00	67.74

PREPARATION OF THE SAMPLE

COHESIVE SOIL			SÒLS NO COHESIUS			
UNCHANGED	COMPACTED (SATURATED / NON-SATURATED)		DRY SAND	COMPACT DRY SAND	SATURATED SAND	MEDIUM SATURATED SAND
Hacked	Dry density ρ_d (g/cm^3) =		Poured	Dynamic compaction / Stactic compaction	Boiled	Dynamic compaction
	Energy E (J/cm^3) = 0.583			Vibration	Boiled	
	Lím. plàstic =			Poured		

DATA OF THE ASSEMBLY AND OF THE SAMPLE

	NORMAL PRESSURE		1kg/cm^2	2kg/cm^2	3kg/cm^2
Scroll speed	v(mm/min)	=	1.00		1.00
Distance from the top edge of the box to the base plate	h_1 (mm)	=	40.50		40.50
Distance from the top edge of the box in the porous plate sup. (cohesive soil)	h_2 (mm)	=	6.00		6.00
Distance from the top edge of the box in the slotted plate (non-cohesive soils)	h_3 (mm)	=			
Total height of the plates used in the essay (without the SLOTTED SUP in COHESIVE SOILS)	h_t (mm)	=			
Initial height of the sample (cohesive soil)	h(mm)=h_1-h_2-h_t	=	34.50		34.50
Initial height of the sample (non-cohesive soils)	h(mm)=h_1-h_3-h_t	=			
Distance from the top edge of the piston filling the top edge of the box	h_4 (mm)	=			
Distance from the top edge of the piston filling the top edge of the box with the yoke of loads and in balance	h_5 (mm)	=			
Settlement due to the yoke	h_4-h_5(mm)	=			
Wet mass of the initial sample	m_{hi} (g)	=	76.10		63.30
Dry mass of the test piece	m_{di} (g)	=	67.75		56.35
Mass wet the bottom of the sample	m_{hf} (g)	=	77.3		64.3
Settlement consolidation	h_c(mm)	=			

MOISTURE AND DENSITIES

NORMAL PRESSURE			1kg/cm^2	2kg/cm^2	3kg/cm^2
Initial moisture	W_i(%)=100*(m_{hi}-m_{di})/m_{di}	=	12.32		13.06
Final moisture	W_f(%)=100*(m_{hf}-m_{df})/m_{df}	=	14.15		16.77
Index of initial voids	e_i=(ρ_s/ρ_d)-1	=	---	---	---
Index of voids	e=e_{i+}(Δh(1+e_i)/h)	=	---	---	---

(Δh = change of height -mm- sample)

NORMAL PRESSURE		1kg/cm^2	2kg/cm^2	3kg/cm^2
Dens. apar. ini.	ρi (g/cm^3) = m_{hi}/V=	1.82		1.81
Dens. seca ini.	ρ_d (g/cm^3)=m_{di}/V=	1.62		1.60
Dens. apar. fin	ρϕ (g/cm^3) = m_{hi}/V=	1.85		1.85
Particle density	ρ_s=	---		
Initial saturation	S_i(%) = (W_i*ρ_s)/e_i=	---	---	---

MOISTURE AND DENSITIES: AVERAGE VALUES

Initial moisture	W_i(%)=100*(m_{hi}-m_{di})/m_{di}	=	**12.69**
Final moisture	W_f(%)=100*(m_{hf}-m_{df})/m_{df}	=	**15.46**
Index of initial voids	e_i=(ρ_s/ρ_d)-1	=	**---**
Índex de buits	e=e_{i+}(Δh(1+e_i)/h)	=	**---**

(Δh = change of height -mm- sample)

Dens. apar. ini.	ρι (g/cm^3) = m_{hi}/V=	**1.81**
Dry density	ρ_d (g/cm^3)=m_{di}/V=	**1.61**
Dens. apar. fin	ρϕ (g/cm^3) = m_{hi}/V=	**1.85**
Particle density	ρ_s=	**---**
Initial saturation level	S_i(%) = (W_i*ρ_s)/e_i=	**---**

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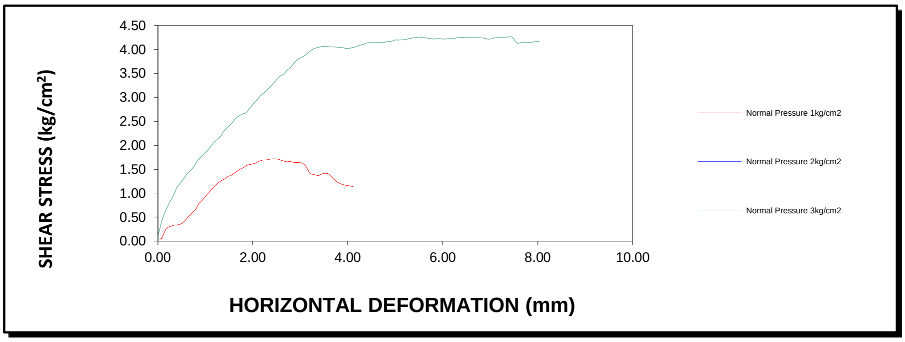
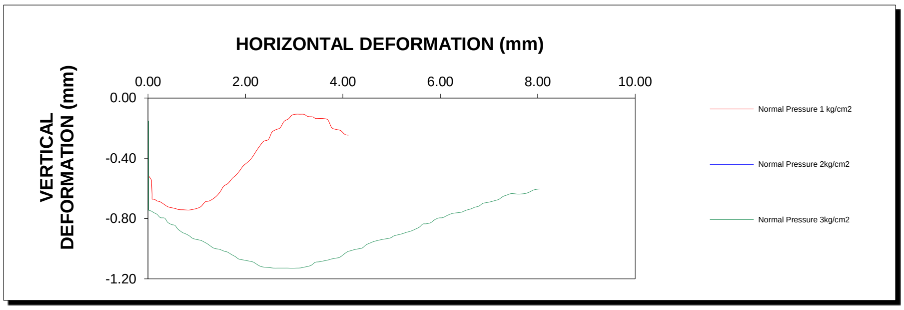
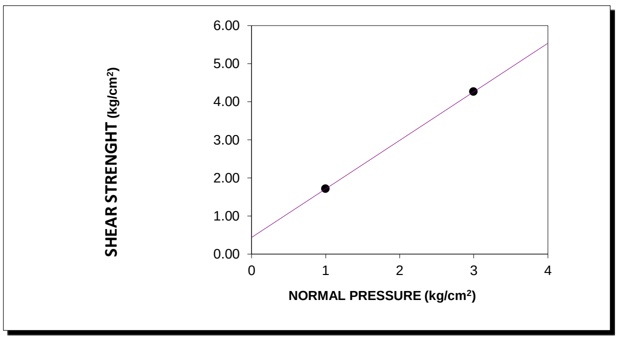
CUSTOMER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L. NIF: B61847091
WORK:	SANTA LUCIA
WORK Nº:	728
SUBMITTER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L.

SAMPLE Nº:	.2017/4264		
REFERENCE:	TP5 (3.00m)		
SAPLE TYPE:	☒ INALTERADA	☐ REMOLDEJADA	
TEST TYPE:	☐ CD	☐ CU	☒ UU
DATE REPORT:	11/08/17	REPORT CODE:	2017/8994







Normal pressure (kg/cm^2)	1		3
Shear strenght (kg/cm^2)	1.72		4.26

COHESION:			
0.4431	kg/cm^2	43.45	kPa

INTERNAL ANGLE OF FRICTION φ= **51.85º**

OBSERVATIONS:

SORRA SOBRECONSOLIDADA. NOMÉS S'HAN POGUT OBTENIR DOS PROVETES PER AL ASSAIG

EL DIRECTOR DEL LABORATORI

JAVIER VICENTE

EL TÈCNIC RESPONSABLE

GUILLEM RODRÍGUEZ

LABORATORI AMB DECLARACIÓ RESPONSABLE PER AL CONTROL DE QUALITAT DE L'EDIFICACIÓ LLIURADA EL 15 DE SETEMBRE DE 2010 AMB NÚMERO L0600088

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CLIINT:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L. - NIF: ESB61847091
WORK:	SANTA LUCIA
Nº WORK:	728
SUBMITTER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L.

SAMPLE Nº:	.2017/4264		
LOCALITZACIÓ:	TP5 (3.00m)		
DATE COLLECTION:	31/07/2017		
DATE REPORT:	11/08/2017	**REPORT CODE:**	2017/9001

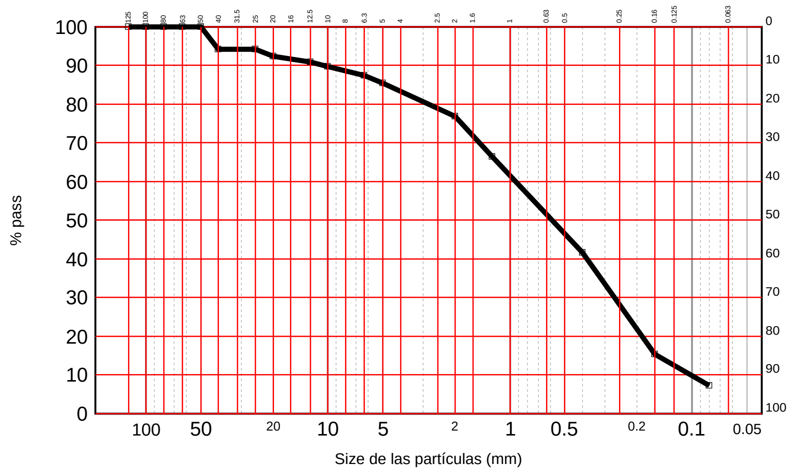
RESULTS REPORT

Method of analysis	Washing and sifting

Essay 0188 - SOIL. GRANULOMETRY. 103101/NLT104 S/UNE 103101 ó NLT104

Sieve (mm)	125	100	80	63	50	40	25	20	12.5	10	6.3	5	2	1.25	0.4	0.16	0.08
Pass (%)	100	100	100	100	100	94	94	92	91	90	87	85	77	66	42	15	7.2

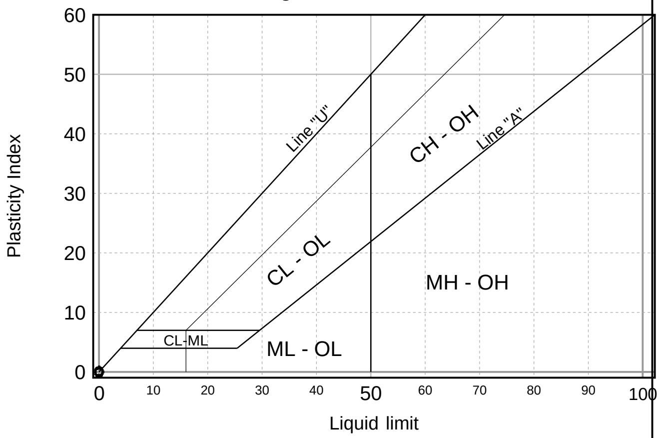
Grading curve



Particle size distribution S/ASTM-D 2487/00			
Blocs		More de 300 mm.	0.0%
Boulders		De 75 a 300 mm.	0.0%
Gravels (15.3%)	Thick	De 19 a 75 mm.	7.9%
	Thin	De 4.75 a 19 mm.	7.4%
Sands (78.0%)	Thick	De 2 a 4.75 mm.	7.9%
	medium	De 0.425 a 2 mm.	34.5%
	Thin	De 0.075 a 0.425 mm.	35.6%
Silts and clays		Less de 0.075 mm.	6.7%

ATTEMBERG'S LÍMITS S/UNE 103.103:94 y UNE 103.104:93	
Liquid limit	Not available
Plastic limit	No Plastic
Plasticity Index	No Plastic

Casagrande's Abac



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CLIINT:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L. - NIF: ESB61847091	**SAMPLE Nº:**	.2017/4264
WORK:	SANTA LUCIA	**LOCALITZACIÓ:**	TP5 (3.00m)
Nº WORK:	728	**DATE COLLECTION:**	31/07/2017
SUBMITTER:	TECSÒL. ASSESSORIA TECNICA DEL SÒL, S.L.	**DATE REPORT:**	11/08/2017 **REPORT CODE:** 2017/9002

RESULTS REPORT

DETERMINATION OF DINSITY FROM A SOIL UNE 103.301-94		
WET DINSITY	g/cm³	**1.808**
DRY DINSITY	g/cm³	**1.599**

SULPHATES. ACCORDING TO UNE 103-201-96		
% SO3	%	**0.0440**
SO3	mg/kg	**440**
SO4	mg/kg	**528**

DETERMINATION OFS CHLORIDES IN SOIL		
CHLORIDES	mg/kg	**40.4**

DETERMINATION OF PH IN SOIL	
PH	**6.44**

DETERMINATION OF CONDUCTIVITY FROM A SOIL		
CONDUCTIVITY	µS/cm	**829**

DESCRIPTION OF THE SAMPLE
SAND A LITTLE CONSOLIDATED

ESSAYS
DINSITY - TALL UU - SULPHATES - CHLORIDES - PH - CONDUCTIVITY - GRANULOMETRY - LÍMITS ATTERBERG

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Annex 7: VES Report

1. Applicable regulations and Standards

Field test methodology in this report follows strict UK and EU regulations.

- Technical Instruction For Soil Resistivity Measurement
- ASTM G57-06 (Test method for field measurement of soil resistivity using the Wenner four-electrode method) norm
- OHSAS and related labour safety codes
- DIN/VDE norms and IEEE standards

2. Introduction and general issues

TecsolGeo Ltd.l, SL has been commissioned to undertake a resistivity soil test survey within La Tourney site, Saint Lucia, where a PV solar development project is anticipated.

Electrical survey has been carried out on a natural non-cropped field. Field test results have been devoted to obtaining resistivity values of soil underground in Ohm·m. The resistivity equivalent model for further calculations and design tasks have been obtained.

Four total resistivity tests have been undertaken within property boundaries on a subparallel arrangement.

This resistivity survey follows national and international electrical surveying standards. Homogeneous ground conditions allowed us to extrapolate interpretation of collected data to the whole property area.

3. Methodology

3.1. Used method

Underground soil electrical surveying has long demonstrated to be very helpful for soil resistivity modelling purposes. Different techniques and methods are available though. The tetraelectrodic disposal method has been chosen to meet resistivity modelling goals in current exercise.

Vertical Electrical Surveys (VES) are very well known and documented in the industry; in this method DC electrical currents are transmitted to the ground by means of four pikes rammed into the ground: two pikes (electrodes) are used to introduce DC currents into the ground and two additional pikes (voltage or potential electrodes) are used to obtain resistant values. Final electrical current values are measured after a few seconds and electric potential difference arises between both electrodes.

Pikes are aligned following the building main direction. The measurement is then repeated by placing pikes at different distances, that allowing different measurements at different depths.

In order to calculate the actual resistivity that shall be devoted to earthing design, pikes drawing can be applied more accurately by using either Wenner, Schlumberger, Pole-Dipole or Dipole-Dipole arrangements.

Wenner method has been selected herein due to high accuracy in horizontal terrains. In such method, the component obtained on field tests plus intensity and voltage data, sets out the raw Wenner values array.

All above has been used to obtain data field curves. From raw field measurements, curves and apparent resistivity figures, a vertical arrange of depth, layer thickness and actual resistivity values are obtained.

3.2. The Wenner test method

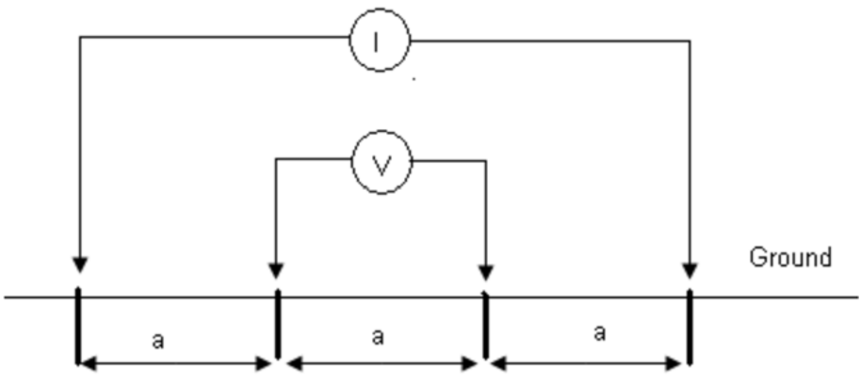
The Wenner four-pin method is the most commonly used technique for soil resistivity measurements. An apparent soil resistivity value is calculated through the following equation:

$$\rho_E = \frac{4 \cdot \pi \cdot a \cdot R_W}{1 + \frac{2 \cdot a}{\sqrt{a^2 + 4 \cdot b^2}} - \frac{a}{\sqrt{a^2 + b^2}}}$$

Where

a = electrode spacing (m)
b = depth of electrodes (m)
R_W = Wenner resistance (Ω)
ρ_E = measured apparent soil resistivity (mΩ)

Wenner method consists of four electrodes, two are for current injection and two for potential measurement as shown below.



In any homogeneous isotropic soil resistivity will be constant. However, if soil is non homogeneous and the electrode spacing varied, a different value of resistivity (**ρa**) will be found for each measurement. This measured value of resistivity is known as the apparent resistivity. The apparent resistivity is a function of the array geometry, measured voltage (Δv), and injected current (I).

3.3. Equipment used

In order to undertake the resistivity test survey the following geoelectrical device has been used:

- Integrated Megger DET 2/2 Automatic Resistivity System, automated measurement device with spontaneous potential correction, measurement precision up 0.5% and internal memory.
- Standard cables VES measurements copper made.
- Data obtained was calculated by means of IPI2 Win measuring software.

4. Results and conclusions

In order to perform a proper geoelectrical survey, norms and regulations as per clause 1.0 above have been followed. Seven sets of electrical resistivity measurements have been carried out, each measurement labelled VES1 to VES3.

Measurements were carried out under good weather conditions, high temperatures and average to high humidity. Filters and spontaneous potential currents were used to correct anomalous interferences that might lead to wrong measures. Electrodes rammed into natural ground.

Such arrangement provides with apparent resistivity values which are then transformed to actual resistivity figures and the equivalent underground model. Measures have been taken at 0.5m to 15m electrode separation and 0.5m each interval in order to obtain field data curves. That would provide with good resistivity variation results from top soil down to 15m depth.

Results correlate well with DPL and trial pit descriptions.

Measurements are quite similar in magnitude among all three tests, i.e., resistivity values are similar at all three test locations. It can therefore be concluded that those same values can be extrapolated across the property.

Data is represented on a bi-logarithmic chart. Interpretation of field data has resulted in a two-three layer model (see below).

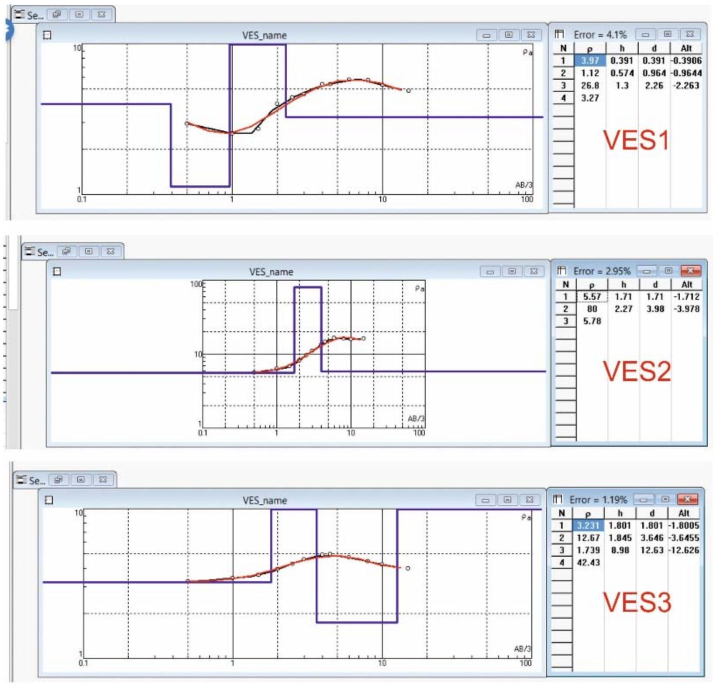


Figure 10: Calculated resistivity curves for 3-4 layers model

VES 1	Depth Lay	Resistivity Average Value (ohm.m)
	0,00 - 1,00	2,41
	1,00 - 2,26	26,8
	>2,26	3,27
VES 2	**Depth Lay**	**Resistivity Average Value (ohm.m)**
	0,00 - 1,71	5,57
	1,71 - 3,97	80
	> 3,97	5,78
VES 3	**Depth Lay**	**Resistivity Average Value (ohm.m)**
	0,00 - 1,80	3,23
	1,80 - 3,64	12,67
	3,64 - 12,62	1,73
	>12,62	42,43

Table 10: Calculated layers resistivity values and interpretated lithologies

Figures below show typical resistivity soil values depending on soil composition or geological background.

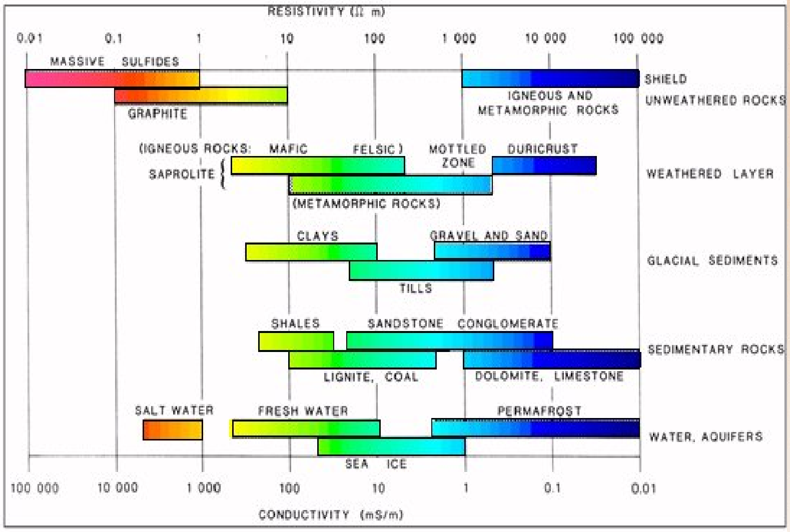


TABLE 1 Source:IEEE142/BS 7430	
Soil Type	Average Resistivity Ωm
Well graded gravel	600-1000
Poorly graded gravel	1000-2500
Clayey gravel	200-400
Silty sand	100-800
Clayey sands	50-200
Silty/clayey sand with slight plasticity	30-80
Fine sandy soil	80-300
Inorganic clays of high plasticity	10-55
Surface soils	1-50
Clay	2-100
Sandy clay	100-150
Moist gravel	50-700
Dry gravel	700-1200
Limestone	5-10000
Porous limestone	30-100
Quartz, crystalline limestone	100-1000
Sandstone	20-2000
Granites	900-1100
Concrete	300-500

Figure 11: Typical Resistivity soil values classified per lithologies

All geotechnical and geological units detected from in situ test soil performed suit with resistivity values determined through resistivity survey carried on.