

**CHINA NON-FERROUS METAL INDUSTRY'S FOREIGN ENGINEERING AND
CONSTRUCTION CO.LTD**

**GEOTECHNICAL INVESTIGATION
PHASE I
FACTUAL REPORT
SOUTH ALUMINIUM SMELTER (SALCO) IRAN**

July 2015

**خدمات مهندسی مکانیک خاک
مهندسان مشاور / پیمانکار تخصصی ژئوتکنیک
SOIL ENGINEERING SERVICES
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خدمات مهندسی مکانیک خاک
مشاور و پیمانکار تخصصی ژئوتکنیک



NFC Iran Representative Office

Att: Mr. Su Zewei

Re: Submission of Factual Report Phase I and II
Geotechnical Investigation for South Aluminium Smelter Plant

Dear Sir,

Please find enclosed 3 hard copies and one electronic copy (CD) of the above mentioned report.

We take the opportunity to thank you and your colleagues for the sincere cooperation during investigation and report preparation.

Shall there be any questions, please do not hesitate to contact us.

Regards,

Bahram Amini

Managing Director



CC: One hard copy submitted to VFB

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1. SECTION ONE

GENERAL

1.1. Introduction

This report presents the factual geotechnical investigation results for Phase I prepared under the provisions of Contract for Geotechnical survey for South Aluminum Smelter Project (SALCO) in Iran between China Non-ferrous Metal Industry's Foreign Engineering and Construction Co.Ltd (NFC) and Soil Engineering Services (SES), Contract No.NFC-SALCO-2015-ZC001 dated March 2015.

The geotechnical investigation works have been divided by NFC into two Phases, namely Phase I and Phase II. The present report deals with Phase I.

1.2. Purpose

This report serves the purpose of furnishing the following information:

- Description of site geology,
- Description of site seismicity,
- Determination of stratification and geotechnical parameters for various layers,
- Determination of soil electrical resistivity and corrosiveness,
- Evaluation of chemical characteristics of soil and water to provide recommendations for type of cement to be used for foundation works,
- Evaluation of liquefaction potential and recommendation for remedial works if necessary,

1.3. Site Location

The site is located approximately at 53°14'56" E and 27°23'34" N at south west entrance (see Figure 1) and is at about 10 km north of Lamerd in Fars Province. In close proximity to this site, a Geotechnical investigation was previously carried out in 2006 which is shown as "old site".

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1.4. Brief Project Descriptions

The project is the largest national aluminium complex in Iran. The general contractor is NFC of China, one of the leading companies in the world in aluminium smelter field. Based on an International Tender, NFC was assigned the works in 2011, with a possibility of updating the works on an annual basis. Production is estimated to be 300,000 Tons/ year, using 400 KA technology. Access is given to 200 Ha for the construction purposes.

1.5. Background

The following list briefly describes the documentary background to this present report.

a) Site Daily Reports

Site daily reports were prepared by SES site manager and were submitted to NFC representatives. These reports comprised of various aspects of site activities including:

- Borehole location and elevation;
- Date and weather conditions;
- Depth of borehole;
- Changes in groundwater level;
- Field description of each stratum encountered;
- Depth at which water level was first encountered and depth to water level at the start and end of each working shift;
- Drilling rig type, make and number and name of operator;
- Details and depths of all in situ tests carried out and all field installations;
- Details of casing size and depth when each sample or drill run was completed and where in situ tests were carried out;
- Depth below ground level for change of strata and depth of all samples/drill runs;
- Details of delays and breakdowns;
- Recorded photo or movies;
- Any other relevant information.

b) Weekly Progress Report

These reports were prepared by SES project manager and submitted to NFC on a weekly basis with information details as mentioned above.

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c) Monthly Progress Report

The report were submitted every month, encompassing details as explained in a) above, but also incorporating progress details, percentages of works performed and analysis of delays (if any).

1.6. Geotechnical Investigation Scope of Works

Geotechnical investigation scope of works in accordance with contract for Phase I report are shown in Figure 2 and outlined in Table 1.

Table 1- GEOTECHNICAL INVESTIGATION SCOPE OF WORKS

ITEM	DESCRIPTION	TOTAL WORK VOLUME	PHASE I
1	Drilling and In-Situ Tests		
1.1	General Boreholes	4,605 m	3,327
1.2	Sampling Boreholes	3,334 m	2,418
1.3	Vane Boreholes (Including Testing)	676 m	260
1.4	SPT Boreholes (Including Standard Penetration Tests)	1,546 m	801
1.5	CPT Boreholes (Including Dutch Cone Penetration Tests)	1,580 m	788
1.6	Pressuremeter Tests	135 Nos.	42
1.7	Plate Loading Tests	10 Nos.	6
1.8	California Bearing Ratio Test	7 Nos.	7
1.9	Lefranc	78 Nos.	38
1.10	Shear Wave Test (Down Hole Tests)	8 Nos.	4
1.11	Soil Resistivity Survey (Goelectric)	16 Nos.	8
1.12	Trial Pits	24 m ³	14
1.13	Water Level Measurement And Monitoring	35 Nos.	15
1.14	Soil Sampling (Disturbed & Undisturbed)	2,659 Nos.	1,915
1.15	Water Sampling	190 Nos.	85
Table Continued			

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1.16	Back Filling to the Bored Holes	11,850 m	0
2	Laboratory Soil & Water Tests		
2.1	Bulk Density, Moisture Content, Specific Gravity, Organic Content and etc.	4,674 Nos.	1,750
2.2	Atterberg limit tests	792 Nos.	433
2.3	Grain size distribution	836 Nos.	287
2.4	Normal Consolidation Test	13 Nos.	63
2.5	High Pressure Consolidation Test	12 Nos.	0
2.6	Direct Shear Strength Test	57 Nos.	25
2.7	Unconfined Shear Strength Test	57 Nos.	201
2.8	Permeability Test	95 Nos.	8
2.9	Static Tri-Axial Test (UU)	4 Nos.	36
2.10	Static Tri-Axial Test (CU)	4 Nos.	5
2.11	Chemical Soil Test	19 Nos.	41
2.12	Chemical Water Test	19 Nos.	7

The boreholes were drilled by the following rigs and cone penetration apparatuses as presented in Table 2.

Table 2- DRILLING RIG TYPE

Drilling Rig Type	Maximum Drilling Capacity (m)	Nos.
OGB	300	6
GX-ITD	150	3
XY1	150	3
XY2	500	2
XY3	700	1
G2A	50	1
SKB4	300	1
XY2PC	300	2
Craelius-D200	200	1
DCPT(Gauda)	—	2
SUM		22

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With reference to Figure 3, borehole locations are shown for various types of borehole. General boreholes were drilled by continuous coring method, sampling, shear vane and SPT boreholes were carried out by dry rotary drilling with water and core barrels are used occasionally.

Standard penetration Tests (SPT's) were performed at required depths according to ASTM D1586. The number of blows for the penetration of a standard sampler of 45cm length with a hammer weighing 63.5 kg falling from a height of 76 cm was registered for every 15cm of penetration. The SPT will be considered "refusal" if one of the following conditions are met:

- A total of 100 blows,
- Recording 50 blows for any 15cm penetration,
- No penetration for 10 consecutive blows,

For undisturbed sampling at required depths, Shelby or thin- wall tubes were used. Shelbys are provided with a ball check valve to hold the sample inside the tube upon withdrawal. The tubes were immediately waxed at both ends to keep samples in their natural condition. The tubes were transferred to the central laboratory at Tehran for laboratory testing. Depths of samples are given in borehole logs.

Dutch cone penetration tests were performed by two 20^T apparatuses (Gauda type). The tip was mechanical and Begemann type. The cone has a standard cross sectional area of 10cm², pushed inside soil by means of 1m long rods. The sleeve has 150 cm² area and 20 cm length. The apparatuses were calibrated just before transportation to the site.

1.6.1. General Boreholes

General boreholes were drilled to the maximum depth of 50 m. Core samples were collected continuously. SPT's were performed in a few general boreholes in order to collect more comprehensive data. General boreholes are included in Enclosure C.

1.6.2. Sampling Boreholes

Sampling boreholes were drilled to the maximum depth of 50.5 m, under the same condition as SPT Boreholes and attached in Enclosure E. SPT tests were also carried out in some of the sampling boreholes.

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1.6.3. Shear Vane Boreholes

Shear vane borehole were drilled to carry out shear vane tests at 2m intervals. The tests are generally employed for soft soils and therefore the maximum depth attained was 10 m. Rectangular blades were used, for which the height is twice the diameter. When pushed near the required depth, the blades are held about 10cm above the borehole base, then rotated by means of a torque-meter from the top and the required torque is registered for later use and the probable correction of actual readings. Then the system is gently pushed to the registered depth so that the tip of the blade is at least 5x blade diameter in undisturbed soil. (The torsion is applied slowly with a view to shear the soil by a 90 degree rotation in 15 minutes. The reading is recorded and the vane is rotated 10 more times and remoulded strength is registered consequently. The vane results are presented in Enclosure F.

The relationship is as follows:

$$T = \pi c_u \left(\frac{d^2 \cdot h}{2} + \frac{d^3}{6} \right)$$

Where,

T= torque at failure

d= overall vane width

h= vane length

1.6.4. SPT Boreholes

SPT boreholes were originally drilled down to 25.5 m depth maximum. Standard penetration tests were carried out generally at 3m intervals. Additional SPT's with maximum depth of 50m were performed as replacement or complementary in sampling and general boreholes (see Enclosure G).

1.6.5. Dutch Cone Penetration Tests (CPT's)

These tests were carried out to the maximum possible depth for which a 20^T Dutch Cone apparatus could penetrate. Results are shown in Enclosure H. It must be mentioned here that some penetration tests were performed to shallower depths, in order to gather data for shallow foundations.

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Due to heavy desiccation of top 5m of soil, at times, it was mandatory to drill the first few meters by means of a drilling rig first for a few CPT tests , and then to penetrate the cone from there on.

1.6.6. Electrical Resistivity Tests

Electrical resistivity tests were carried out for 8 points in various directions.

The method employed is Wenner Four Electrode. Electrical current is induced through the two outer electrodes and the drop in voltage is measured by the two inner receiving electrodes. As the electrode spacing is increased, lower depths of soil are affected.

Test results are provided in Enclosure I. The test points are shown in Figure 49. A sketch of Wenner Four Electrode Method and site activity is presented in Figure 50.

1.6.7. Plate Loading Tests

Six plate loading tests were performed in trial pits. The test depth was 3m and plates were loaded vertically. Two plate sizes used are 30 cm and 45 cm in diameter. An Enerpac Jack (50T capacity) and 3 deformation dial gauges (0.01mm reading) were used. The results are presented Enclosure J. Test method is according to ASTM 1194.

1.6.8. Trial Pits

A total of 14 m³ of trial pits were carried out and the results are illustrated in Enclosure K. The soil profile encountered is in accordance with soil conditions obtained in nearby boreholes.

1.6.9. Shear Wave Tests

Seismic cross-hole tests are performed in which two cross-hole seismic profiles of p-wave and s-wave is taken (BH-109, BH145, BH202 and BH-297).

Geophysical explorations are used to study tectonic characteristic and formation geology, identify geological layering and estimate dynamic properties of materials, considering of behaviour of soil under earthquake.

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Geophysical methods are divided into two groups. In the first group the physical quality natural source is considered. This group includes gravity, magnetic, radiometry, and thermometry and seismicity. The second group includes measuring the response of non-natural or synthetic source, seismic refraction, induced polarization and electromagnetic.

The most common method used in engineering geophysical investigation is seismic and electric study. Seismic methods include refraction, reflection, down hole, cross hole, and tomography cross hole. In present studies, downhole method is used.

This method is commonly used to determine compression and shear wave velocities versus depth. These velocity data are used to help assess seismic response and determine stratigraphy of a particular site

In downhole seismic survey, a seismic source is placed on the surface near a borehole, and a geophone is placed at selected depths in the borehole. The raw data obtained from a downhole survey are travel times for compression and shear waves from the source to the geophones and the distance between the source and geophones.

Table 3 presents equations of dynamic parameter, compression velocity (V_p), shear velocity (V_s) and density.

Table 3- EQUATIONS OF DYNAMIC PARAMETERS, COMPRESSION VELOCITY (V_p), SHEAR VELOCITY (V_s) AND DENSITY

Dynamic Parameters	Solid	Fluid
Poisson's ratio	$\nu = \frac{\left(\frac{V_p}{V_s}\right)^2 - 2}{2\left[\left(\frac{V_p}{V_s}\right)^2 - 1\right]}$	0.5
Wave Velocity ratio	$\left(\frac{V_p}{V_s}\right)^2 = \frac{2.2\nu}{1-2}$	∞
Shear Modulus	$G = \rho \cdot V_s^2$	0
Young Modulus	$E = 2G(1+\nu)$	0
Bulk Modulus	$K = \frac{1}{3} \cdot \frac{E}{1-2\nu}$	ρV_p^2

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ρ , V_p and V_s are density, longitudinal wave velocity and wide wave velocity, respectively. Approximate values of Poisson's ratio for igneous rocks, sedimentary rocks and soil are 0.25, 0.33 and 0.1 to 0.49, respectively.

The results of shear wave tests are included in Enclosure L.

1.6.10. Pressuremeter Tests

The Pressuremeter tests were performed for determination of the stress-strain behaviour of subsurface layers. According to ASTM D4719 the Pressuremeter consists of two main elements: a radial expendable cylindrical probe, which is placed inside the borehole at the desired test depth and a monitoring unit, which remains on the ground surface. The following geotechnical parameters can be obtained from Pressuremeter test results:

- Deformation modulus
- Undrained shear strength (C_u) for clays and weak rocks
- Shear modulus
- Pressure Limit

The pre-boring Pressuremeter (P.B.P), size AX, Type GC, was used in this project. Based on expansion of a cylinder that is placed inside the borehole the pressure-volume variation during testing is recorded. Pressuremeter test is performed by 60 mm probe placed inside the slotted tube.

The procedure to analyse the Pressuremeter test data is as follows:

- 1) Determination of pressure difference between cell and core (P_c) based on calibration test,

- 2) Determination of hydrostatic pressure between cell and control unit (P_δ) ,

$$P_\delta = H \times \gamma_1$$

H: Depth of cell position with respect to control unit

γ_1 : Unit weight of fluid

- 3) Pressuremeter test data correction based on the above mentioned items 1 to 3,

$$P = P_{\text{pressurimeter}} + P_\delta - P_c$$

- 4) Drawing pressure-volume change curve,

- 5) Determination of ΔP , ΔV and V_m from the sketch,

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ΔP : Corrected pressure difference in the linear part of the graph

ΔV : Corrected volume difference in the linear part of the graph

V_m : Corrected volume in the middle of the linear part of the graph

Based on the values of item 5 and V_0 (primary volume), Pressuremeter modulus E_p (Menard modulus E_M) is calculated:

$$E_p = E_m = 2(1 + \nu)(V_0 + V_m) \frac{\Delta P}{\Delta V}$$

According to elasticity equation, shear modulus, G is obtained from the equation:

$$G = (V_0 + V_m) \frac{\Delta P}{\Delta V}$$

The Pressuremeter modulus (E_p) is related to Oedometer modulus (E'_s) with the following relation:

$$E'_s = E_{oed} = E_p / \alpha \quad (\alpha = 0.67 \text{ for clay and } 0.5 \text{ for silt})$$

The summaries of Pressuremeter test results in boreholes are presented in Enclosure M.

The values of Pressuremeter modulus obtained from Pressuremeter tests are converted to Young modulus as follow:

$$E_s = \frac{E_p (1 + \nu)(1 - 2\nu)}{(1 - \nu)}$$

ν : Poisson's ratio (taken equal to 0.3)

1.6.11. California Bearing Ratio Tests

California Bearing Ratio (CBR) tests are performed according to ASTM D4928-04. The results are presented in Enclosure N.

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1.6.12. Permeability Tests (Lefranc)

Permeability tests (seepage tests, Lefranc) were carried out in certain boreholes at various depths under constant and falling heads. Flush bottom or well point filter in uniform soil methods were employed (see Enclosures O). The results which were originally presented in the Interim Report are revised and reviewed for this report.

1.6.13. Soil Sampling

Sampling was carried out under ASTM D 1587 and samples were transferred to the central laboratory in Tehran, where, they were subjected to various tests as described below.

1.6.14. Core Box Photographs

Photographs were taken of the cores in core boxes and are presented in Enclosure P. Each photograph has a standard colour code along with core views.

1.7. Laboratory Testing

In order to classify subsurface soils profile and determine their physical, chemical and mechanical properties, series of tests were undertaken in the laboratory testing programme. Some photographs are also presented in Enclosure AA. All the samples initially were subjected to a visual/ manual inspection (ASTM D 2488-90), and then were classified according to Unified Soil Classification System (USCS, ASTM D2487).

1.7.1. Grain Size Distribution & Atterberg Limits

Test method ASTM D422 was used for the grain size and hydrometer analysis (see Enclosure Q). Atterberg limits (ASTM D4318) results are also presented in Enclosure R.

1.7.2. Soil & Water Chemical Tests

Sulphate and chloride ions content and PH values (soil and water) as well as conductivity (water) of certain selected samples were determined according to BS 1377 1990 to evaluate their effect on concrete works. The results are shown in Enclosure S.

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1.7.3. Quick Direct Shear Tests

Under the requirements of ASTM D 3082, certain samples of the top crust (dry) or its vicinity were tested for determination of shearing properties. The results are shown in Enclosure T.

1.7.4. Triaxial Tests

Unconsolidated Undrained (UU) triaxial tests were carried out in accordance with ASTM D 2850 and the results are presented in Enclosure U. Consolidated Undrained (CU) test results are shown in Enclosure V.

1.7.5. Consolidation Tests

In order to determine the soil compressibility and deformation behavior, ASTM D 2435 guideline was used for the tests. The results are shown in Enclosure W. The tests were kept under two conditions, under water for saturated soil samples and if recovered from above water table, they were protected by plastic covers to stop samples from drying.

1.7.6. Unconfined Compression Tests

ASTM D 2166 was used to conduct these tests. Samples were tested under their natural moisture content. The results are presented in Enclosure X.

1.7.7. Specific Gravity of Soil Grains (G_s)

Samples were tested according to ASTM D 854.

1.7.8. Permeability Tests

Permeability tests were performed on undisturbed samples as shown in photographs enclosed in Enclosure AA. Samples were removed after tests for grain size distribution and Atterberg limit analysis to check relevance of permeability results against soil type. The results are presented in Enclosure Y.

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1.7.9. Swelling Tests

The tests were performed in 3 categories:

- Free swell under no pressure (Enclosure Z/A)
- Loaded to 2 kg/cm² pressure and then inundated to measure the swell (Enclosure Z/B)
- Inundated initially but loaded to stop swell in order to measure swelling pressure (Enclosure Z/C)

1.7.10. Photographs

Selective photographs of test samples are shown in Enclosure AA. These photographs show certain features of the samples and their condition, including those which were deemed not suitable for testing and hence were rejected.

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2. SECTION TWO

RESULTS

2.1. Introduction

This Section presents the results of geotechnical study based on information outlined in Section one. Furthermore, Geological and Seismic studies are addressed and presented herein.

2.2. Geological Description of Site

The geological site condition is presented in Annex I..

2.3. Site Seismology

The site seismology is presented in Annex II.

2.4. Stratification

2.4.1. Geotechnical Units

The ground conditions encountered in Phase I have been subdivided into the geotechnical units shown in Table 4 below and presented in our geotechnical sections A-A to S-S (See Enclosure A). The Recent Alluvium unit is fine sedimentary deposits of 60 to 80 m thick overlying Aghajari formation.

Table 4- SUMMARY OF GEOTECHNICAL UNITS

ORIGIN	UNIT NAME	DEPTH (m)	DESCRIPTION
RECENT ALLUVIUM	UNIT I	0 to -5	(CL, CL/ML, CL/CH) LEAN CLAY; very stiff to hard consistency, dry, overconsolidated, organics (e.g. decomposed plants and tree roots) for 10 to 15 cm.
	UNIT II	-5 to -15	(CL, CL/ML, CL/CH) LEAN CLAY; occasionally ML medium stiff to stiff to very stiff consistency, light brown, moist, low plasticity, overconsolidated to slightly overconsolidated.
	UNIT III	-15 to -25	(CL, CL/ML, CL/CH) LEAN CLAY; occasionally ML stiff, very stiff to hard, consistency. Light brown, moist, low plasticity.
	UNIT IV	-25 to EOB - 50	(CL, CL/ML, CL/CH) LEAN CLAY; occasionally ML hard consistency. Light brown, moist, low plasticity.

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2.5. Groundwater Table

Groundwater table within Lamerd region has undergone fluctuations in the past 15 years due to several reasons, and therefore adopting a current measured groundwater table may lead to several problems during the project lifetime. Hence, to implement a best groundwater table in the design we have reviewed and assessed the available information as below;

- The existing published information shows there are 940 numbers of deep wells, 46 numbers of spring waters and 8 numbers of drainage systems in Lamerd region, the rate of groundwater table extraction from the mentioned sources have dramatically increased from 2001 and the measurement records from 2001 to 2006 reveal a groundwater level drop for 2.9m and 1.6m in the upper and lower region of Lamerd field, respectively. (Source: *Groundwater Study for Lamerd Region 272* , Iranian Energy Ministry, 2012)
- The trend observed in raining rates indicates decreasing pattern from 85 mm to 30 mm from 2001 to 2006, and this trend will last during drought years but it is likely to increase after drought period.
- Groundwater level reported in a geotechnical investigation report previously carried out in this site shows groundwater table ranging from 2.5 m to 4 m.
- In our present investigation, Groundwater levels are measured from many piezometers and borehole logs. Figure 46 shows groundwater table variations measured weakly from piezometers from month of May 2015 to June 2015. They indicate groundwater level between 5 m and 5.7 m.

It can be noted that the groundwater levels depend predominantly on two key factors which are the amount of groundwater extractions from the wells in the region and annual rainfall. Currently the rate of water extraction is high and the amount of raining is low, however if the regulations on groundwater extraction change or the drought period ends the groundwater table is likely to rise. Thus, for the purpose of design, SES suggests adopting a groundwater table at a constant level of 3 m.

2.6. Assessment of Geotechnical Parameters

Geotechnical parameters required for designs include soil index properties, strength, stiffness and consolidation parameters. These are discussed in the following sections and finally recommended parameters are presented in Section 2.7.

Geotechnical data obtained from the investigation is vast and the size of the area under study is large, therefore in order to draw a conclusive assessment several attempts were made to classify these data into zones. The zones were created finally on the basis of locality of the structures as no clear distinction could be observed from North to South or West to East of the site. Initially the site was divided into 5 zones. The zones 1 to 5 comprise the following structures as described in Table 5 and Figure 3:

Table 5- ZONES FOR GEOTECHNICAL DATA CLASSIFICATION

ZONE NO.	STRUCTURES NO.
1	4042, 4071
2	3011, 3021, 3031, 3041, 3051, 4011, 4021, 4051
3	4011, 4031, 4051
4	4051
5	6031, 6041, 6051, 6061, 6071, 6081

The zone classifications were carried out for some *in-situ* data such as SPTs, CPTs , field shear vane, Pressuremeters and undrained shear strength (S_u) derived from unconfined compression laboratory tests.

2.6.1. Soil Index Properties

Soil Index properties comprise of grain size distribution, bulk density, specific gravity, moisture content, and Atterberg limits. The summary of results indicates the followings;

- Grain size distribution: Soils typically comprise of lean CLAY (CL, CL/ML, CL/CH), and occasionally SILT (ML),
- Bulk density: The values obtained are consistent in depth and is 19.9 to 21 kN/m³ , with the best estimate to be 20 kN/m³ (See Figure 5),
- Specific gravity: The values are consistent and typically are 2.70.

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- Moisture content: The values ranging from 20% to 27%, with the best estimate to be 21% and 25% for depth from 0 to 5m and 5m to 50m, respectively (See Figure 6),
- Atterberg limits: Liquid limits range from 30 to 46, with a best estimate to be 37. Plastic limits vary from 17 to 27, with a best estimate to be 23, Plastic Index also ranges from 8 to 28, with a best estimate to be 18. Shrinkage limits are in the order of 18. (See Figures 6 to 8).

2.6.2. Soil Strength Parameters

The strength parameters or undrained shear strength, S_u , can be derived from all field test data including, SPTs, CPTs, shear vanes and Pressuremeter tests. Furthermore, laboratory tests data such as unconfined compression; unconsolidated undrained and pocket penetrometers test results are used. In addition to that, some existing information from previous geotechnical investigations carried out near this site were reviewed. The approach in obtaining strength parameters are explained and discussed herein.

2.6.2.1. Undrained Shear Strength from SV

Undrained shear strength results for peak values are presented in Figure 9. For depth from 5 m to 10 m, the results are varying from minimum 25 kPa to maximum 160 kPa with accumulation of data around a value of 60 kPa. The intent of design is to consider the lower bound situations and therefore it is not suggested to adopt S_u equal to 60 kPa, and the lower bound range of 25 kPa to 30 kPa shall be adopted.

According to Mitchell, Clays with sensitivity values in between 1 to 2 is slightly sensitive, and it becomes medium sensitive for values between 2 to 4. Any values above 4 are very sensitive. Therefore, based on results shown in Figure 10, it can be concluded that the Clays for this site are slightly to medium sensitive, with some occasional very sensitive points.

2.6.2.2. Undrained Shear Strength from SPT

Figures 11 to 16 present summaries of SPT data for all boreholes and for five zones. These data are firstly used for differentiating the stiffness of soils at every zone. It also shows that in each specific zone, the variations of data exist. Table 6 outlines correlation of SPT “N” values and stiffness of soils.

Table 6- CORRELATION OF SPT N VALUE AND STIFFNESS FOR SATURATED FINE-GRAINED SOILS

N	DESCRIPTION
0-2	Very soft
3-5	Soft
6-9	Medium
10-15	Stiff
15-30	Very Stiff
>30	Hard

Adopting Table 6 classification, substratum soils have been classified for stiffness into four units as presented in Table 4. The “N” values for all zones show scattered values with increasing trends in depth.

In addition to above, attempts were made to find a correlation value from S_{uv} , shear vane with SPTs. The correlation obtained ranges widely from 1.3 to 11, and considered inconclusive. On the other hand, Design Manual for Soil Mechanics NAVFAC DM-7.1, 1986, presents a relationship between SPT N and undrained shear strength from laboratory tests, unconfined compressive strength test, for clays of low plasticity and clayey silts (See Figure 17). The correlation adopts a coefficient of 3.7 based on Sowers relation ($S_u = 3.7 \times N$) and the results are presented in Figure 18. Table 7 outlines the range of lower bound values for undrained shear strength from SPT tests.

Table 7- RANGE OF LOWER BOUND UNDRAINED SHEAR STRENGTH FROM SPT

UNIT	DEPTH (m)	S _u (kPa)
I	0-5	75<
II	5 - 15	26 to 44
III	15 - 25	45 to 75
IV	> 25	75 <

2.6.2.3. Undrained Shear Strength from CPT

Clay consistency can be distinguished with reference to unconfined compressive strength (Q_u) as outlined in Table 8.

Table 8- CLAY CONSISTENCY BASED ON UNCONFINED COMPRESSIVE STRENGTH

CONSISTENCY	FIELD IDENTIFICATION	UNCONFINED COMPRESSIVE STRENGTH Q _u (kPa)
Very soft	Easily penetrated several inches by fist	Less than 25
Soft	Easily penetrated several inches by thumb	25-50
Medium	Can be penetrated several inches by thumb with moderate effort	50-100
Stiff	Readily indented by thumb but penetrated only with great effort	100-200
Very stiff	Readily indented by thumb nail	200-400
Hard	Indented with difficulty by thumb nail	Over 400

Based on Table 8, CPT q_c results were categorized for five zones and are plotted versus depth in Figures 19 to 23.

Undrained shear strength (S_u) may be calculated by correlations of cone resistance, a total overburden pressure and cone factor.

A cone factor, N_k , is calculated from undrained shear strengths from vane shear tests (S_{uv}), carried out in the boreholes adjacent to CPTs and by employing cone resistance q_c and effective stress.

$$N_k = \frac{(q_c - \sigma'_{v0})}{S_{uv}} \quad (\text{Robertson, 2002})$$

As it can be seen in Figure 24, N_{kt} is adopted as 23 based on available data and engineering judgment for lightly overconsolidated clay. Figure 25 shows undrained shear strength derived from CPTs.

The CPT results indicate the cone resistance mainly have increased from 13m, resulting in subdivision of Unit II to II-a and II-b. For Unit III, also a sublayer exists from 20m to 25m, marked as Unit III-a and III-b. Table 9 summarises lower bounds of undrained shear strength.

Table 9- LOWER BOUND UNDRAINED SHEAR STRENGTH FROM CPT

UNIT	DEPTH (m)	S_u (kPa)
I	0-5	>100
II-a	5 - 13	35
II-b	13-15	80
III-a	15 - 20	80
III-b	20-25	170
IV	> 25	450

2.6.2.4. Undrained Shear Strength from Pressuremeter

In order to have an estimation of undrained shear strength, Baguelin et al (1978) and Briaud et al. (1985). presented the following equation;

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$$\frac{S_u}{P_a} = 0.21 \left[\frac{P_l}{P_a} \right]^{0.75}$$

Where;

P_a = Atmospheric pressure,

P_l = limit pressure,

The results for S_u versus depth are shown in Figure 26.

2.6.2.5. Undrained Shear Strength from Laboratory Testings

Laboratory testings carried out to obtain undrained shear strength (S_u) of clayey soils comprised of:

- 350 x Pocket penetrometers
- 201 x Unconfined Compression (UCS)
- 36 x Unconsolidated undrained triaxial (UU)

UCS, UU and pocket penetrometers data are summarised in Figures 27 to 33 for all data and five (5) zones separately. This shows that the measured undrained shear strengths are scattered for zones and can be summarised as below;

- Unit I is ranged as low as 16 kPa to a maximum value of 136 kPa,
- Unit II is ranged from 25 kPa to 80 kPa, with the strength remaining approximately constant with depth from 5 m to 15 m. In unit II, data are accumulated from 25kPa to 44 kPa.
- Unit III is ranged from 35 kPa to 130 kPa with more data clustered from 40 kPa to 60 kPa, with the strength varying in proportion to depth.
- Unit IV is ranged from 35 kPa to 150 kPa, with the strength varying in proportion to depth. It is evident that data are accumulated from 50 kPa to 90 kPa.

2.6.2.6. Previous Geotechnical Studies

Previous geotechnical studies undertaken nearby our site for Iranian Mines and Mining Industries, Development and Renovation Organization, by MAAB consulting engineers,

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Reference No 50602025L3712500RE11930125 dated March 2014, has reported the followings;

- *From ground level down to 3 m to 5 m, soil strength properties above groundwater level is high,*
- *From 3 m to 10 m, soil is referred as soft with undrained shear strength values ranging from 35 kPa to 80 kPa,*
- *From 10 m to 20 m, undrained shear strength values ranging from 50 kPa to 90 kPa,*
- *From 20 m to 30 m, undrained shear strength values ranging from 120 kPa to 150 kPa.*

2.6.2.7. Design Undrained Shear Strength Profile

Lower bound and best estimate strength parameters were initially established for these units based on *in-situ* and laboratory testings, and some degree of variations in data are observed. These variations are also evident from all *in-situ* data, particularly CPT results compared to SPT results. When performing CPTs, human errors have minimum to no influence on the parameters recorded, and therefore it is considered as a very reliable *in-situ* testing. Hence, it can be concluded that the site conditions vary, resulting in variations of data.

For design purpose, more emphasis is given to lower bound data. The reason for such judgment is due to the fact that the site is vast and estimation of strength parameters for each foundation based on adjacent boreholes is impossible and impractical. On the other hand, a failure of one single shallow foundation is likely to have a severe outcome for the structure. Thus, assigning a single value for each unit is proposed that is representative of the lower bound values to ensure that inherently lower strength parameters are captured and hence the design is safe. Table 10 presents adopted undrained shear strength values for design.

Table 10- ADOPTED DESIGN VALUES FOR UNDRAINED SHEAR STRENGTH

UNIT	DEPTH (m)	S_u (kPa)	REMARKS
I	0-5	30	Groundwater table fluctuations, increase in groundwater table reduces S_u dramatically to values similar to Unit II
II	5 - 15	30	Average lower bound value from laboratory tests and SPTs. Lower bound parameters from CPTs
III	15 - 25	50	Average lower bound value from laboratory tests and SPTs
IV	> 25	75	Average lower bound value from laboratory tests and SPTs

2.6.3. Quick Shear Strength Parameters

A total number of 25 Quick Shear Test (DST) has been carried out in Unit I and top of Unit II to estimate effective cohesion and angle of frictions. Figure 34 shows the results obtained from tests and the adopted values are presented in Table 11.

2.6.4. Soil Modulus of Elasticity

Modulus of elasticity of soils may be evaluated from Pressuremeters, CPTs and a correlation presented by Ladd (1972) based on stress history (overconsolidation ratio) on undrained behaviour E_u/S_u (See Figures 35 to 36).

The relation used to obtain modulus of elasticity from Pressuremeter is outlined below.

To derive modulus of elasticity from CPTs, we have adopted the following equations:

$$E_u = \alpha \cdot q_c$$

$$E' = 1.35 \times E_u$$

Where;

E_u : Constrained Modulus of Elasticity,

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α : Coefficient of constrained Modulus, 3 to 8 for q_c less than 0.7 MPa, and 2 to 5 for q_c greater than 0.7 MPa,

q_c : CPT cone resistance

E' : Modulus of elasticity,

The coefficient of α equal to 8 and 5 are adopted for q_c less than 0.7 MPa and q_c greater than 0.7 MPa, respectively.

It is evident that the minimum and maximum values are estimated from Pressuremeters and the correlation presented above. For design purpose, it is recommended to interpret the modulus of elasticity from Ladd (1972) correlation.

2.6.5. Preconsolidation and Overconsolidation Ratio

The Preconsolidation pressure (P'_c) refers to the maximum effective vertical stress that has acted on the clay in the past and overconsolidation ratio (OCR) is the ratio in between P'_c and vertical effective stress.

The overconsolidation ratio (OCR) is estimated from consolidation tests, liquid index (LI) and CPT results. The preconsolidation pressures are derived from consolidation tests.

The following equation presents the correlation with liquid index (LI),

$$OCR = \left(\frac{P_a}{\sigma'_{v0}} \right) 10^{(1.11 - 1.62LI)}$$

Where,

P_a : Atmospheric pressure equal to 100 kPa,

σ'_{v0} : Vertical effective stress

LI: Liquid Index

The relations presented for CPT by Robertson, 2002 are as below;

$$OCR = k \left(\frac{q_t - \sigma'_{v0}}{\sigma'_{v0}} \right)$$

$$\sigma_p = k \times (q_c - \sigma'_{v0})$$

$$k = 0.33$$

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The (P'_c) and OCR profiles are presented in Figures 44 and 45. Table 15 presents the adopted values for design.

2.6.6. Compressibility Characteristics

The initial void ratios (e_0) from 63 number of consolidation tests are presented in Figure 37. The compression parameters comprising Compression Index ($C_{ce}=C_c/1+e_0$), Recompression Index ($C_{se}=C_r/1+e_0$) have been assessed from and summarized in Figures 38 and 39. Table 11 outlines the adopted values for design.

2.6.7. Coefficient of Volume Compressibility

The coefficient of volume compressibility m_v (MPa^{-1}) may be expressed as below:

$$m_v = \frac{1}{1 + e_0} \left(\frac{e_0 - e_1}{\sigma'_1 - \sigma'_0} \right)$$

The value of m_v for a particular soil is not constant but depends on the stress range over which it is calculated. Figures 40 to 43 are prepared for pressures ranging from 50 kPa to 100 kPa and 100 kPa to 200 kPa and the recommended values are 0.35 and 0.25 (MPa^{-1}) respectively.

2.6.8. Swelling Potential

A total number of 17 swelling tests was carried out. Three groups of swelling tests were carried out to measure swell potential of top 5m of soil, these tests are comprised of i) 12 numbers of free swell, ii) 4 numbers of swell under applied surcharge pressure close to imposed structure pressure of 200 kPa and iii) one test to control swell pressure by inundating the sample and applying pressure to stop swelling. The results are presented in Enclosure Z.

Based on the data from free swelling presented, it is evident that a swelling potential ranges from zero to a maximum of 1.4%. If the average value of 0.2% is considered for a foundation with an embedment depth of 1.5m, then the swelling of 5 mm is likely to occur. The results under 200 kPa load are mainly in the order of 0 to 0.4%, there is one exceptional sample (4% swelling potential) that could be discarded. The swelling potential from the remaining tests also illustrate there is insignificant swelling potential.

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2.6.9. Coefficient of Permeability Characteristics

The coefficients of permeability with depth have been interpreted from series of in-situ tests and laboratory permeability tests.

The following relations were used to derive permeability from in-situ tests,

$$K = 0.37 \frac{Q}{LH} \log\left(\frac{L}{r}\right) \text{ Constant head } K = \left(\frac{C}{t}\right) \log\left(\frac{H}{h}\right) \text{ Falling head}$$

$$C = 2.64 \left(\frac{r^2}{L}\right) \log\left(\frac{L}{r}\right)$$

Where:

K= Permeability

Q= discharge

L=length

H=height

r=radius of borehole

t=elapsed time

Total numbers of 8 permeability laboratory tests have been undertaken, the number of tests (48 No. for Phase I) is less than the requested numbers of tests. The reduction in numbers of permeability tests is due to the fact that the soil profile is homogeneous and consists of clay. The results exhibit the permeability results in the range of 10^{-5} to 10^{-8} cm/sec. The results from laboratory tests are less than in-situ tests as the samples selected for testing in laboratory are uniform, where as in-situ tests are carried out over a certain length with sitly interbedded lenses that are likely to increase the permeability results. Table 11 summarizes our recommended permeability values for the units.

2.7. Summary of Design Geotechnical Parameters

Table 11 summarises the geotechnical parameters presented and discussed above.

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Table 11 - ADOPTED SOIL PARAMETERS FOR PHASE I

Soil Unit	γ kN/m ³	e_0	Strength Parameters			Compression Parameters		Permeability	E (MPa)	OCR (-)
			ϕ degs	C' (kPa)	S_u (kPa)	C_{CE} (-)	C_{SE} (-)	K_y (cm/s)		
Unit I	20	-	15	30	-	0.1	0.016	10^{-6}	45	2.5
Unit II	20	0.715	-	-	30	0.1	0.016	5×10^{-7}	21	1.5
Unit III	20	0.685	-	-	50	0.09	0.016	10^{-7}	40	1
Unit VI	20	0.7	-	-	75	0.09	0.016	10^{-7}	60	1

The soil symbols and acronyms used in Table 11 are as follows:

- γ – Bulk Unit Weight
- S_u - Undrained Shear Strength
- C_{CE} - Compression Ratio = $C_c / (1+e_0)$; C_c - Compression Index
- C_{SE} - Recompression Ratio = $C_s / (1+e_0)$; C_s - Recompression Index
- OCR - Overconsolidation Ratio
- K_y - Vertical permeability

2.8. Plate Loading Test Results

With reference to test results included in Enclosure J, correcting the curves for initial kinks, ultimate and allowable bearing capacities (by applying a safety factor of 2) were determined and using the following equation, modulus of sub-grade reaction was obtained:

$$k_s = \left(\frac{q}{\delta}\right).I$$

Where:

k_s = modulus of subgrade reaction for standard 30^{cm} plate

q =allowable pressure

δ = deformation (settlement) for q

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I = test condition factor = 0.5

For different diameters the relation $(\frac{k_s}{k_{s'}}) = (\frac{b'}{b})$ is used, where:

b'= plate diameter

$k_{s'}$ = modulus of sub grade reaction for plate with diameter, b'

b=standard 30^{cm} plate diameter

Based on the above discussion, average $k_s = 40 \frac{\text{MN}}{\text{m}^3}$ is obtained.

2.9. Shear Wave Test Results

For determination of compression wave velocity, seismic source of P-wave (vertical blow by hammer on the plate) is placed at 3.0 m from borehole and then down-hole geophone surveys all of the borehole depth. For determination of shear wave velocity, S-wave seismic source (horizontal blow by hammer on the plate) is placed at 3.0 m from borehole and then down-hole geophone surveys all of the borehole depth. Results of seismic test are presenting in Enclosure L. Dynamic parameters included compression velocity (V_p), shear velocity (V_s), elasticity modulus (E), shear modulus (G) and bulk modulus (K). Further analyses of data are presented in Annex II.

Iranian Building Code No. 2800 is used for determination of soil profile type. The Code uses $V_{s(30)}$ as mean shear wave velocity to 30 m depth based on the following equation:

$$V_{s(30)} = \frac{\sum_{i=1}^u T_i}{\sum_{i=1}^u \frac{T_i}{V_{si}}}$$

Where T_i in thickness of layer i and V_{si} is shear wave velocity in that same layer

The mean shear wave velocities for boreholes BH-109, BH-145, BH-202 and BH-279 up to 30m depth have been calculated and found to be 305, 295, 318 and 309 m/sec, respectively. Therefore, soil profile of the site is classified as type III based on the mentioned code as shown in Table 12.

**Table 12- SOIL PROFILE TYPES BASED ON
IRANIAN BUILDING CODE NO. 2800**

DESCRIPTION	$V_{s(30)}$ m/sec	PROFILE TYPE
a) Igneous rocks, stiff sedimentary and metamorphic rocks and conglomerate b) Stiff soils (compact gravel and sand, very stiff clay) with a thickness less than 30 m above bedrock	>750 375-750	I
a) Loose Igneous rocks, stiff sedimentary and metamorphic rocks and conglomerate b) Stiff soils (compact gravel and sand, very stiff clay) having a thickness more than 30 m above bedrock	375-750	II
a) Rocks that are disintegrated due to weathering b) Soils with medium compaction, layers of sand and gravel with medium inter granular	175-375	III
a) Soft deposits with high moisture content due to high level of water table b) Any soil profile containing clay with a minimum thickness of 6 m and a plastic index and water content exceeding 20 and 40 percent respectively	<175	IV

2.10. California Bearing Ratio (CBR)

A total of 7 tests were performed. The results range from about 11% to almost 50%. The tests were carried out during dry season of May to June 2015, and therefore the values obtained are considered to be more than the values in wet season.

For clayey material (CL), it is expected to have CBR values to be in the range of 5-15%. Consequently, for wet condition (80% saturation), a few more tests are planned and the results will be presented in Phase II report.

2.11. Electrical Resistivity Test Results

Electrical resistivity of soil is measured in Ohm-m units from the relationship

$$\rho = 2 \pi .a.R$$

Where:

ρ = electrical resistivity in Ohm-m

a = electrode spacing in m

R = electrical resistivity in Ohm

E = potential difference in mv

I = current in mA

The calculated results given in Enclosure I are presented in Figure 51 in graphical form showing variations at depth.

With reference to electrical resistivity test results, classification of corrosiveness by the National Iranian Gas Company (NIGC, Clause No. 925) is as per Table 13,

**Table 13- CORROSIVENESS CLASSIFICATION
BASED ON ELECTRICAL RESISTIVITY TESTS (NIGC)**

ELECTRICAL RESISTIVITY (Ω - M)	CORROSIVENESS
0-5	Very Corrosive
5-10	Corrosive
10-20	Moderately Corrosive
20-100	Mildly Corrosive
More than 100	Progressively Less Corrosive

BS CP 1021 Standard proposes a wider range per following Table 14.

**Table 14- CORROSIVENESS CLASSIFICATION BASED ON ELECTRICAL RESISTIVITY TESTS
(BS CP 1021)**

ELECTRICAL RESISTIVITY (Ω - M)	CORROSIVENESS
Up to 10	Very Corrosive
10-100	Relatively Corrosive
More than 100	Less Corrosive

2.12. Chemical Soil & Water Characteristics

With reference to Enclosures S, it can be seen that the soil and water have pH values above 7 which indicate alkalinity and hence harmless environment for concrete works from this aspect.

Maximum Sulphate (SO_4^{2-}) (aqueous 2:1 water extract) and chloride ion contents for soil are 0.67% and 1.36% respectively (See Figure 52 to 55).

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Maximum sulphate and chloride ion contents for water were originally measured to be 7,373 PPM and 41,654 PPM respectively. It was noted that sulphate ion content of water samples was very high. This was attributed to sampling condition, because samples were taken about 10 days after drilling works were completed. Therefore, water test results were not considered final and fresh water samples were taken and results are presented in this report (Figure 56). The maximum SO_4^{2-} in water is measured to be 3848 PPM. It seems that water sampling is very sensitive in this project and results are affected tremendously. Therefore, we suggest that these results are not conclusive and more fresh water samples from trial pits shall be collected. The trial pits will be dug to about 50 cm below watertable level. We expect to finalise this issue in Phase II of this project.

2.13. Liquefaction Potential

The purpose of this section of the report is to evaluate the potential of cyclic failure of the subsurface layers during earthquakes.

Sands and clays have some basic differences in their cyclic behaviour and thus require different approaches to assess their potential to liquefaction. Silt behaviour under seismic condition can be categorised as either clay like or sand like. This difference rises from plastic behaviour of fine grained soils. In this regard experiences show that silts can be categorized based on their plasticity index. However, the boundaries between these categories slightly change with respect to soil classification according to unified soil classification system (USCS). In this project, subsurface layers mostly consist of clay and in some areas layers of silt at various depth and thickness ranging from 1 to 7 m are encountered. First step in liquefaction potential assessment is to determine whether under cyclic loading these silt layers behave more like sand or clay. In this case, silt layers having plasticity index greater than 5 are expected to behave more like clays and thus their plastic index and the water content in respect to their liquid limit play important role in potential to cyclic softening.

Current guidelines of liquefaction susceptibility criteria for fine grained soils proposed by Seed et al. (2003), which are similar to the Chinese method, are used in this report. These criteria have been widely used in engineering practice for liquefaction susceptibility of clays and are based on field observation in China. The guidelines proposed by Seed et al. are included in Figure 1 of Enclosure B.

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These criteria are based on Atterberg limits. In this approach, soils with a water content (w_c) greater than 80% of the LL (Liquid Limit) are considered potentially susceptible to cyclic failure (Zone A in Figure) and points outside zones A and B are considered generally not susceptible to cyclic failure.

For fine grained soils, a lower w_c/LL ratio is equivalent to higher Overconsolidation Ratio (OCR) and thus higher S_u (undrained shear strength). Therefore this parameter can be used as a primary indicator of soil susceptibility to cyclic failure. It is not a robust rule that soils with $PI < 12$ are susceptible to liquefaction and other factors such as soil mineralogy, void ratio (which in clays changes by time), and OCR must be considered.

Data used in this calculation are collected from different test results such as Atterberg limits, consolidation test, and simple shear test. A single test with all the mentioned data are few, therefore data from various samples in each borehole were extracted to represent the data for that borehole. In other words, the highest value of moisture content from different samples of a certain borehole and the lowest LL and PI (Plasticity Index) of the samples are selected in order to conduct calculations. Results for soils which are classified as "CL" based on unified soil classification system are shown in Enclosure B. Although based on this approach, there are some layers that are susceptible to cyclic softening, but not susceptible to liquefaction under design base earthquake. In the event of earthquake, due to plastic behaviour of clays, the expected consequences are insignificant compared to sand liquefaction.

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3. SECTION THREE

3.1. Cement Type for Foundation Works

The soil pH value is more than 7 as presented in Figure 52. With reference to Figure 53 to Figure 55, the amount of sulphate ion is generally below 3.1 gr/lit and hence the site is classified on the basis of SO_3 . The water samples results are also due to be finalized in Phase II. Therefore, the recommendations for type of cements to be used for foundations are given in the following Table 16 .

Table 16 – CEMENT TYPE FOR FOUNDATION IN RELATION TO SOIL AGGRESSIVELY

TYPE OF FOUNDATION	WHOLLY ABOVE WATER TABLE	IN CONTACT WITH WATER TABLE
Massive Concrete	Normal Portland Cement not less than 350 kg/m ³ (w/c) _{max} =0.50 or Sulphate-Resisting Cement not less than 340 kg/m ³ (w/c) _{max} =0.50	Sulphate-Resisting Cement not less than 350 kg/m ³ (w/c) _{max} = 0.50
Precast Concrete Piles	-	Sulphate Resisting Cement not less than 330 kg/m ³ (w/c) _{max} =0.50
Cast-in-place Piles	-	Sulphate-Resisting Cement not less than 370 kg/m ³ (w/c) _{max} =0.50

Based on Table 16, it is recommended to make well compacted concretes.

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3.2. Soil Corrosivity

Results of electrical resistivity tests (Enclosure I) are presented and exhibited in Figure 51.

It can be concluded that:

- Resistivity of ground to about 6^m depth is roughly less than 100 Ohm-m, which is in the range of relatively corrosive.
- Resistivity below 6^m is about 200-500 Ohm-m on average which is less corrosive.

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