NEPAL ELECTRCITY AUTHORITY

Transmission Directorate

FINAL REPORT

on

Geotechnical Investigation works

At Inaruwa 400kV GIS Substation

(Hetauda – Dhalkebar – Inaruwa 400kV Substation Expansion Project)

Submitted by:

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1. Introduction

The Inaruwa 400kV GIS Substation is located in the Sunsari District of Eastern Nepal.The geotechnical investigation of the substationhas been carried out by Soil, Rock and Concrete Laboratory (SRCL), NEA as per its contract agreement with Inaruwa 400kV substation Improvement Project, Transmission Directorate. The entire work comprised of exploratory core drilling with in-situ tests, 2-D Electrical Resistivity SurveyandLaboratory Tests. All the works have been carried out to meet the technical specification provided in the contract.

2. Location and Accessibility

The Project is located in Sunsari District of Koshi Zone in the Province No.1 of Nepal. The project site can be accessed by East- West highway. Geologically, the project area is located in Indo gangetic plain of Eastern Nepal. The project area lies about 340km east of Kathmandu and is connected with BP highway up to Bardibas and then with East-West Highway upto the project site .

3. Objective

The main objective of the present Geotechnical Investigation work was to assess the sub-surface geological condition at the substation by 2-D Electrical Resistivity Tomography, core drilling, and Laboratory Tests.

4. Scope of Work

The main scope of works that is to be carried out is,

- To carry out Soil exploration with exploratory core drilling with Standard Penetration Test/ Dynamic Cone Penetration Test and Laboratory test of soil samples at four different locations.

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Table 1: Details of Core drilling and tests length

S. No	Bore Hole	Depth as planned (m)	Drilled Depth (m)	In-Situ Test	Purpose
1.	BH-01	15	15	SPT @1.5m interval	To explore soil layers for foundation design.
2.	BH-02	15	15	SPT @1.5m interval	To explore soil layers for foundation design.
3.	BH-03	15	25	SPT/DCPT @1.5m interval	To explore soil layers for foundation design.
4.	BH-04	15	20	SPT/DCPT @1.5m interval	To explore soil layers for foundation design.
5.	BH-05	15	Cancelled	SPT @1.5m interval	To explore soil layers for foundation design.
Total		**75m**	**75m**		

5. Exploratory Drilling and Trial Pits:

As per the recommendations made in the geotechnical investigations of the project area, a contract has been made between Inaruwa 400kV substation Improvement Project, Transmission Directorate and SRCL, NEA to carryout 75m core drilling, in-situ test and laboratory test of core samples.As per the contract, SRCL team mobilized to the site in February, 2018.

Drilling crew, drill rig with all required accessories and camping gears have been transported by truck from SRCL office, Kathmandu to Inaruwa. Platform construction, borehole drilling works were carried out as per requirement.

The core boxes from all sites were logged and photographed by geologist.XY-100 equipped with all required accessories with drilling crewswasdeployed for the drilling operation.

Two numbers of trial pits were dug out upto the depth of 4.0m to investigate the subsurface material and to collect the sample and information regarding the sub surface soil layers. Location of trial pits has been given in figure 1. Details of trial pits has been given in **Annex-4** with photographs.

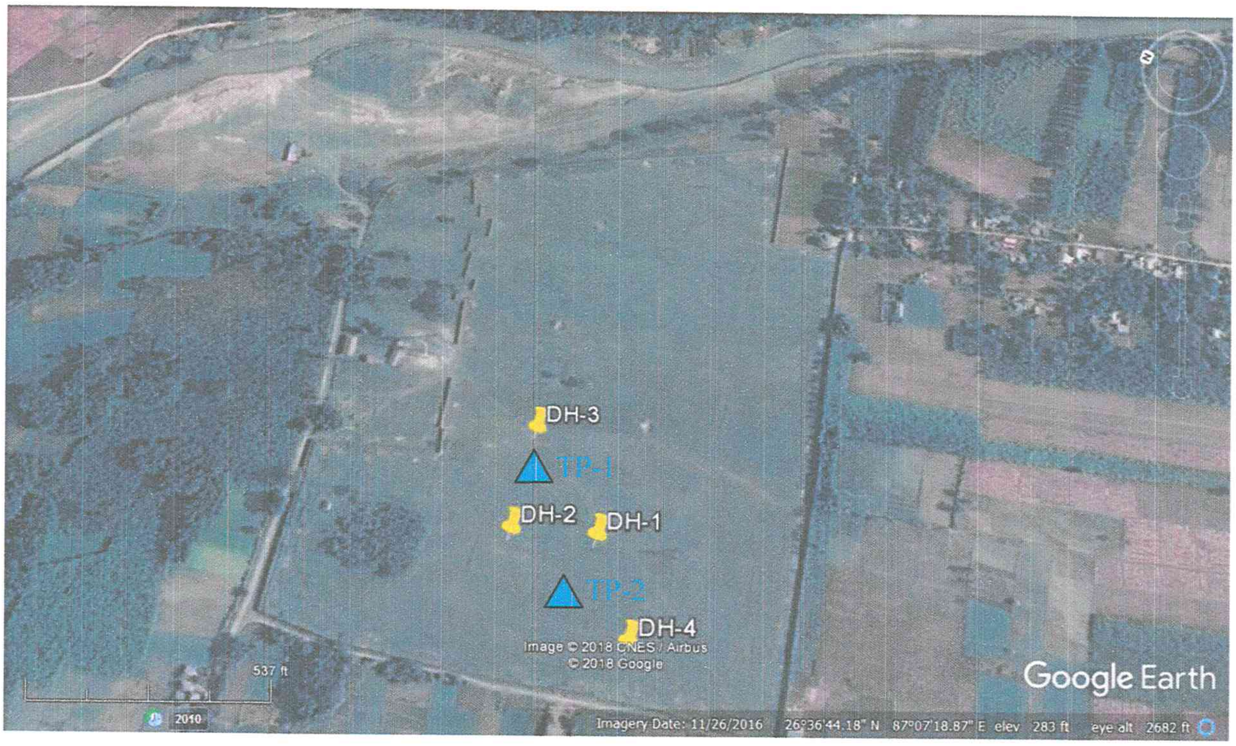

Figure 1: Location of Bore holes and test pits

6. Methodology

6.1 Core Drilling

Core drilling was carried out as per ASTM designation D 2113-83. The core size is NQ size (47 mm) with hole diameter of 76mmand the core barrel is double tube swivel type with retrievable inner tube. The drilling is carried out by using NW casing. All the cores recovered from drilling areplaced safely in wooden box. Geologist hassupervised overall drilling work including in-situ tests. Core logging of drilled cores and core photography arecarried out by geologist in the site. The in-situ tests such as dynamic cone penetration test (DCPT) and standard penetration test (SPT)arecarried out as per internationally accepted standards. The Undisturbed Sample was also taken during core drilling.

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6.2 In-situ Tests

6.2.1 Standard Penetration Test (SPT)

It consists of driving a Split Spoon Sampler with an outside diameter of 50 mm into the soil at the base of borehole. Driving is accomplished by a drop of hammer weighing 63.5 kg falling freely through a height of 750 mm onto the drive head. First of all the spoon is driven 150 mm into the soil at bottom of the borehole. It is then driven further 300 mm and the number of blows (N values) required to drive this distance is recorded.

6.2.2 Dynamic Cone Penetration Test (DCPT)

The test is performed using a 50 mm cone (IS: 4968-part I and II, 1976). The cone is driven with 63.5 kg hammer falling through a height of 75 cm and the number of blows required to penetrate the 300 mm is taken as DCPT values (N-values).

The in-situ test locations/ depths carried out in the drilling holes have been summarized in **Table 2.**

Table 2: Summary of the In-Situ Test performed in the drill holes.

S.N	Borehole No	Water Table(m)	Run Depth(m)	SPT Value (N)	DCPT Value (N)	Remarks
1	BH-1	2.0	0.50-0.95	6		Firm
2			2.00-2.45	9		Stiff
3			3.50-3.95	2		Soft
4			5.00-5.45	13		Medium dense
5			6.55-7.00	9		Loose
6			8.00-8.45	14		Medium dense
7			9.00-9.95	43		Dense
8			11.00-11.45	39		Dense
9			12.50-12.95	25		Medium dense
10			13.50-13.95		15	Medium dense
11			14.55-15.00	35		Dense
12	BH-2	2.2	2.00-2.45	2		Soft
13			3.50-3.95	4		Loose
14			5.00-5.45	10		Firm
15			6.50-6.95	8		Loose
16			8.00-8.45	17		Medium dense

17			9.50-9.95	14		Medium dense
18			11.00-11.45	29		Medium dense
19			12.50-12.95	24		Medium dense
20			14.55-15.00	30		Dense
21	BH-3	2.0	2.55-3.00	6		Firm
22			4.55-5.00	21		Very stiff
23			6.55-7.00	20		Medium dense
24			8.55-9.00	34		Dense
25			10.55-11.00	44		Dense
26			12.55-13.00	34		Dense
27			14.55-15.00	62		Very dense
28			16.55-17.00	>50		Very dense
29			18.55-19.00	73		Very dense
30			20.55-21.00		>50	Very dense
31			22.55-23.00		83	Very dense
32			24.55-25.00		50	Very dense
33	BH-4	2.2	2.55-3.00	4		Soft
34			4.05-4.50	24		Very stiff
35			5.55-6.00	20		Very stiff
36			7.05-7.50	23		Medium dense
37			8.55-9.00	30		Dense
38			10.05-10.50	27		Medium dense
39			11.55-12.60	27		Medium dense
40			13.05-13.50	30		Dense
41			14.55-15.00	31		Dense
42			16.05-16.50	43		Dense
43			17.55-18.00	45		Dense
44			19.55-20.00	45		Dense

6.3 Bearing Capacity Analysis

Standard Penetration Test Results

As mentioned above, total twelve numbers of SPT were carried out in all three test pits. The SPT values obtained in the field were corrected as per the standard practice (presented in detail in bearing capacity calculation by field method). The corrected SPT values have been utilized to calculate the bearing capacity of the soils by field method used mainly as controlling values.

SPT values have been corrected for effective overburden of soil (normalizing factor) and submergence. SPT N_{cor} and allowable bearing capacity values are presented at Annex-4.

7. Findings of the Geotechnical Investigation

The present investigation includes mainly sub-surface investigations such as exploratory core drilling, test pit excavation with laboratory tests. The results obtained from the investigation would be utilized in the design of different structures of the Project.

7.1 Output of core drilling works

Four drill holes in the substation were planned and accordingly drilling works was completed in February 2018.

The soil found at the drill holes is Sand and Silt with gravels. The core drilling was successfully completed as per planned schedule.

7.1.1 Core Logging

The logging of all drill holes has been carried out at site by geologist observing the core boxes and daily drilling report. The standard log sheet has been used for the logging purpose.

The photographs of core boxes have been taken with proper leveling at each core. All core boxes have been stacked properly and stored to site store shown by the Client. The geological logs of all the bore holes are given in **Annex -1,** in-situ-tests in **Annex-2,** and photographs of the core boxes are **Annex-3**.

7.1.2 Results and Analyses

The interpretation of each drill hole has been carried out after logging all the drill holes. The results and analysis of each drill hole have been summarized as follows:

BH-1

The drill hole, BH-1 has been drilled at Right Bank of Sunsari Khola. The total length of drill hole is 15 m. The drill hole consists of alluvial deposit. Upper layer from ground surface to the run depth of 4m is grey colored, Sandy Silt with Clay. Lower layer which starts from the run depth 4.00m to

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15.00m is grey colored, fine to medium grained Sand with Silt. The water table has been recorded at a depth of 2.0 m.There is no core recovery during drilling except SPT sample. TenSPT and one DCPT were conducted in the borehole. The results of the in-situ tests showed that upper soil is Softto Stiff and lower soil is Loose to Dense. The core log and in-situ tests of the hole are given in Annex-**1**and Annex-2**.**

BH-2

The drill hole, BH-2 has been drilled at Right Bank of Sunsari Khola. The total length of drill hole is 15 m. The drill hole consists of alluvial deposit. Upper layer from ground surface to the run depth of 6m is dark grey coloredSilt and Clay. Lower layer starts from the run depth 6.00m to 15.00m is grey colored, fine to medium grained Sand with Silt. The water table has been recorded at a depth of 2.0 m.There is no core recovery during drilling except SPT sample and UD sample. Nine SPT were conducted in the borehole. The results of the in-situ tests showed that upper soil is Softto Stiff and lower soil is Loose to Dense. The core log and in-situ tests of the hole are given in Annex-**1**and Annex-2**.**

BH-3

The drill hole, BH-3 has been drilled at Right Bank of SunsariKhola. The total length of drill hole is 25 m. The drill hole consists of alluvial deposit. Upper layer from ground surface to the run depth of 5m is grey colored Silt and Clay with fine grained Sand. The intermediate layer is between 5m and 13m which consists of fine to medium grained Sand with Silt. Lower layer starts from the run depth 13.00m to25.00m which consists of grey colored Sand and gravels with Silt. The water table has been recorded at a depth of 2.0 m.There is no core recovery down to 13.00m during drilling except SPT sampleand UD sample but gravels were retained after 13m. Eight SPT and three DCPT were conducted in the borehole. The results of the insitu tests showed that upper soil is Firm to Very stiff , intermediate layer is Medium dense to Dense and lower soil is Very dense. The core log and insitu tests of the hole are given in Annex-**1**and Annex-2**.**

BH- 4

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The drill hole, BH-4 has been drilled at Right Bank of Sunsari Khola. The total length of drill hole is 20 m. The drill hole consists of alluvial deposit. Upper layer from ground surface to the run depth of 5m is grey colored Silt and Clay with fine grained Sand. Lower layer starts from the run depth 5.00m to 20.00m which consists of grey colored fine to coarse grained Sand with Silt. The water table has been recorded at a depth of 2.2 m.There is no core recovery down to 13.00m during drilling except SPT sample and after 13m gravels were retained during drilling. TwelveSPT were conducted in the borehole. The results of the insitu tests showed that upper soil is Soft to Very stiff and lower soil is Medium dense to Dense. The core log and insitu tests of the hole are given in Annex-**1**and Annex-2**.**

7.2 Drilling Machines and Limitations

Drilling depth for both overburden and bedrock basically depends upon the capacity of the deployed drill rig. XY-100 drill rig (capacity up to 100m deep)was used for drilling. Except few simple mechanical problems, no significant problems took place during the operation of this machine.

7.3 Allowable Bearing Capcity

The bearing capacity for DH-1 ranges from 9.59 t/m^2 to 26.31 t/m^2, for DH-2 it ranges from 10.11 t/m^2 to 19.39 t/m^2, for DH-3 it ranges from 13.19 t/m^2 to 30.84 t/m^2 and for DH-4 it ranges from 11.5 t/m^2 to 22.16 t/m^2.

7.4 Liquefaction Potential

For DH-1, there is no liquefaction for the magnitude of 6, whereas liquefaction has been observed for the magnitude of 7 upto 8m from the surface. For magnitude of 8, the soil layer from 0-8m and 12-14m is found to be liquefiable.

For DH-2, there is no liquefaction for the magnitude of 6 but the soil layer upto 9.5m is liquefiable for the magnitude of 7 and 8.

For DH-3, there is no liquefaction for 6 magnitude of earthquake. The lower layer below 3.0 m is compacted enough as suggested by the SPT values. The upper layer of 3.0m is potential for liquefaction for the magnitude of 7 and 8. The lower layer below 3m is not liquefiable.

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For DH-4, there is no liquefaction for 6 magnitude of earthquake. The lower layer below 3.0 m is compacted enough as suggested by the SPT values. The upper layer of 3.0m is potential for liquefaction for the magnitude of 7 and 8. The lower layer below 3m is not liquefiable. Calculation of the liquefaction potential has been presented in Annex-6.

7.5 Electrical Resistivity Imaging

2Dimensional Electrical Resistivity Tomography (2-D ERT)

A total of 4 Electrical Resistivity Tomography profiles measuring 750.00 m has been proposed by the client at different locations within the proposed location. 2D-ERT profiles depict different soil layers below the measured locations. 2D-ERT has also depicted the saturated and non saturated zones.

Field data were gathered to obtain a continuous coverage of the subsurface along the line of investigation. Multi-function digital DC Electrical Resistivity meter, multiplex electrode converter (Multi electrode switching equipment), multi core cable with each take out at every 5 m (60 take outs in total) have been used for data acquisition in the field. Based on the location and resolution needed, Wenner electrode configuration was employed in the present study.

Figure 2: Multi-function digital DC Electrical Resistivity meter.

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Four profiles were measured within the proposed building premises to determine the resistivity of dfferent soil layers within. The overview of the ERT profiles is outlined in the google image.

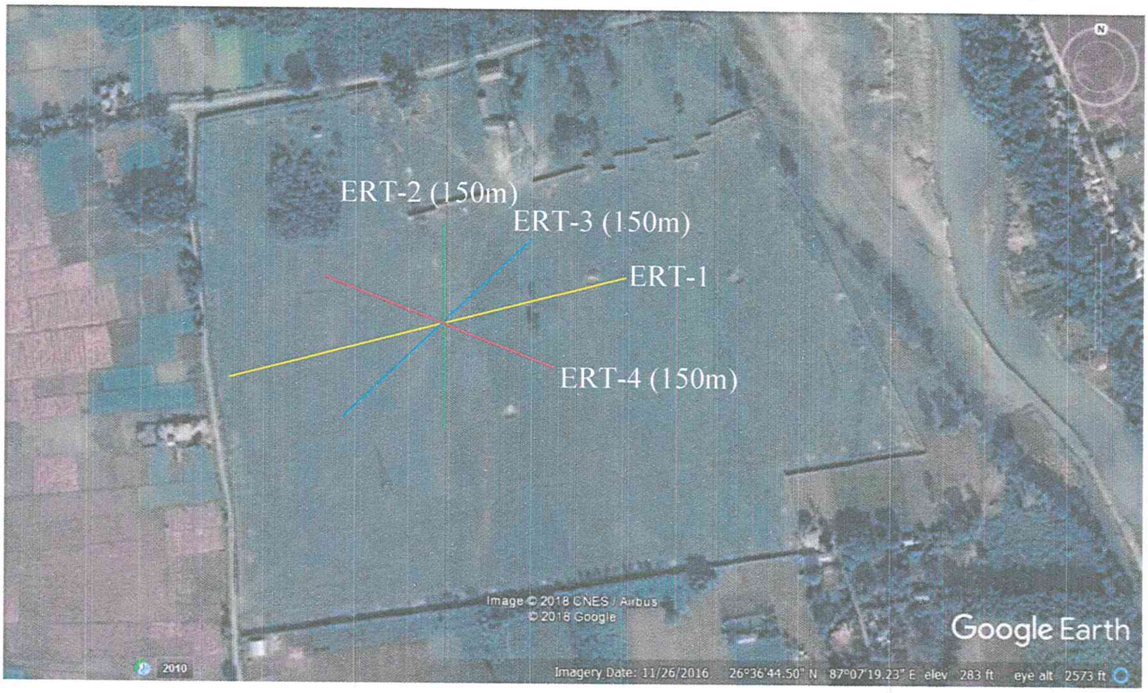
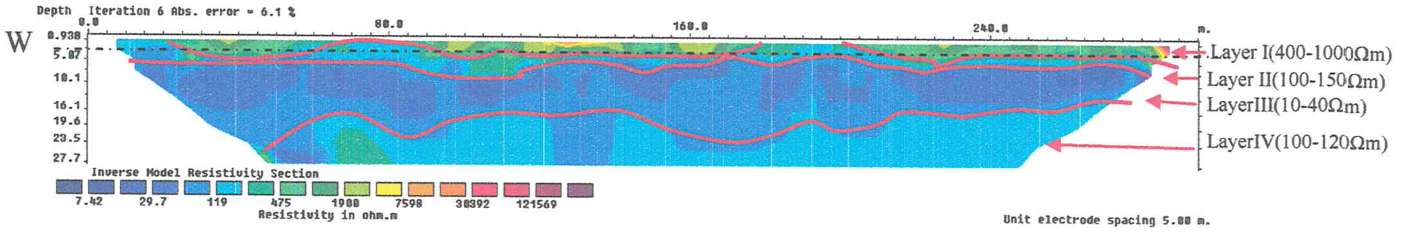


Figure 3: ERT profiles measured in the field.

The acquired data in the field were processed by the use of RES2DINV software and were analyzed on the basis of observed resistivity values. Different layers of soil layers have been outlined by the solid curved lines whereas the dashed line indicate the water level (WL) in each layer. The resistivity value of each layer has been mentioned in each profile at the right margin of each figure.

ERT-1 (300.00m)



ERT-2 (150.00m)

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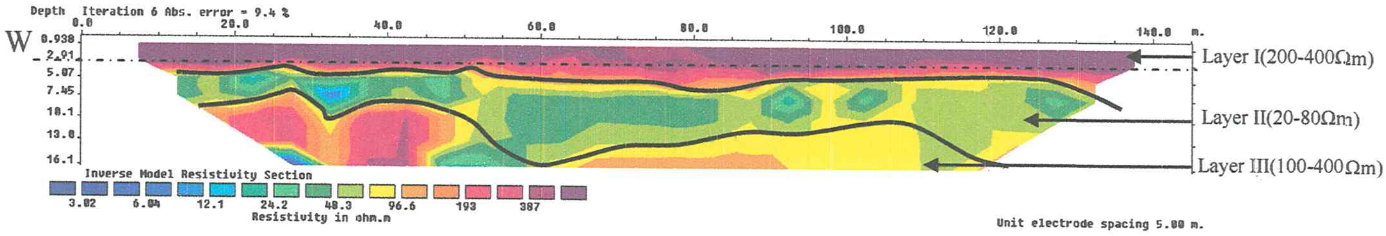
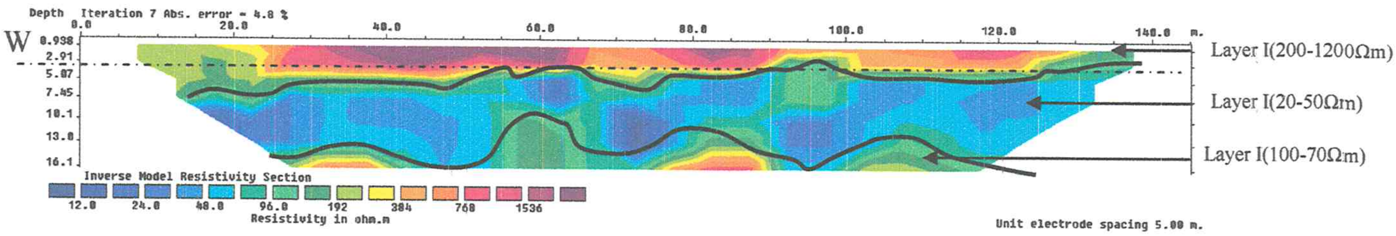


ERT-3 (150.00m)



ERT-4 (150.00m)

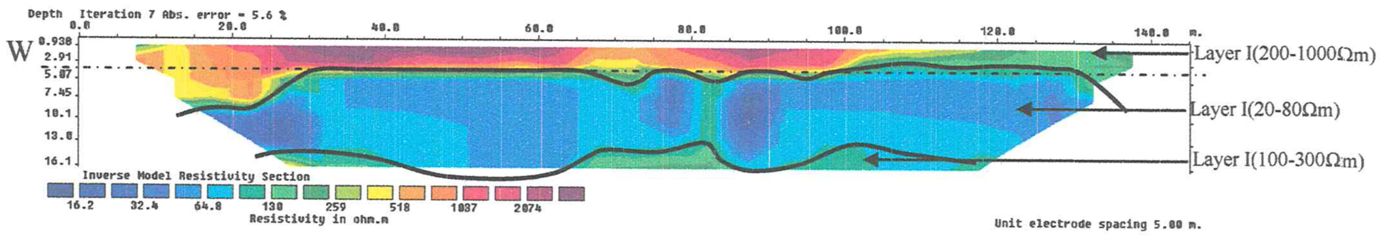


Figure 4: Interpretative images of ERT profiles.

Resistivity value measured in the field has been presented in the Annexes.

8. Bearing Capacity

At the project site, types of soil strata are not found uniform in which the dominant stratifications are cohesive and semi-cohesion less soils.

The calculation of the bearing capacity for isolated(1.5x1.5,2.0x2.0,2.5x2.5,3.0x3.0)m^2 or mat/raft(6.0morgreater)foundations in the borehole locations could be considered for the safe bearing capacity analysis respectively.

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Standardize Field Penetration Value:

$$N_{60} = \frac{N_{rec}\eta_H\eta_B\eta_S\eta_R}{60}$$

Where,

N_{60} = Standard penetration number, corrected for field conditions to an average energy ratio of 60%

N_{rec} = measured penetration number

η_H = hammer efficiency (%)

= 60%

η_B = correction for borehole diameter

= 1.0

η_S = sampler correction

= 1.0

η_R = correction for rod length

= 0.0 – 4.0m	-	0.75
4.0 – 6.0m	-	0.85
6.0 – 10.0	-	0.90
>10.0m	-	1.0

Dilatancy Correction:

Terzaghi and Peck (1967)

If $N_r \leq 15$ Use $N = N_r$

If $N_r \geq 15$ then,

$N_c = 15 + 1/2\ (N-15)$

Where,

N_c = corrected value of N_r

η_R = correction for rod length

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Correction for overburden pressure,

From Peck, Hansen and Thornburn (1974)

N_{corr} = 0.77 Nr log (2000/σ')

Where,

N_r = SPT value from field after dilatancy correction

σ' = effective overburden pressure in KN/m^2

Unit Weight of the Soil Layers (γ) KN/m^3

The unit weight of the soil layers are directly found from the retrieved soil samples through the SPT Tubes in the field or as per the observed N value from the field test. The ultimate design of the foundation is found for the worst condition, i.g. submerged condition. So, the saturated unit weight of the soil layers were found considering the above mostly adopted assumptions.

γ_{sat} = Saturated Unit weight of the soil (KN/m^3)

= 16.0, 17.0, 18.0, 19.0 and 20.0 KN/m^3 (assumed as per observed (N) values)

If, $N \leq 10$ (γ_{sat}= 16 KN/m^3)

$10 < N \leq 15$ (γ_{sat}= 17 KN/m^3)

$15 < N \leq 20$ (γ_{sat}= 18 KN/m^3)

$20 < N \leq 30$ (γ_{sat}= 19 KN/m^3)

$N > 30$ (γ_{sat}= 20 KN/m^3)

CALCULATION OF BEARING CAPACITY

A) For Spread Footing

Assuming a typical having (1.5 x 1.5, 2.0 x 2.0, 2.5 x 2.5, 3.0 x 3.0) m^2 Square Open Isolated Shallow Foundationfor light to medium load bearing structures.

(i) Terzaghi's Relation (1943)

From Terzaghi's equation

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q_{ult} = $1.3\ CNc + \gamma_{sat}\ D_f (N_q-1) + 0.4\ \gamma_{sat}\ BN_\gamma R_{w2}$ ....(I)

q_{safe} = $(q_{ult} + \gamma_{sat} D_f) / F.S.$

Where,

γ_{sat} = Saturated Unit weight of the soil (KN/m^3)

Nc, Nγ& Nq are bearing capacity factors

B = Width of foundation (m)

D_f = Depth of foundation (m)

C = Cohesion (KN/m^2)

F.S. = Factor of safety i.e., 3

R_{w2} = Water correction factor

If soils have loose to medium denseness and soft to medium stiff consistency then the foundation fails according as the local shear failure (LSF) otherwise fails in general shear failure (GSF) criterion.

(ii) Using Meyerhof's (1956, 1974) Correlation

For 25mm Settlement

q_{safe} = $8.1\ N_{60}\ K_{D2}\ ((B+0.3)/B)^2\ R_{w2}$ KN/m^2 for B >1.2m ....(II)

Where,

K_{D1} = $1+0.33\ (D/B) \leq 1.33$

B & D = Breadth and depth of foundation

R_{w2} = Water correction factors = 0.5

B) For Mat Foundation

Considering a typical (6.0 x 6.0) m^2 or (8.0 x 8.0) m^2greater Mat Foundation for the heavy loaded structures.

(i) From Terzaghi's equation

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$$q_{ult} = 1.3\ CN_c + \gamma_{sat}\ D_f (N_q - 1) + 0.4\ \gamma_{sat}\ BN_\gamma R_{w2}\ \ldots\ldots\ldots(I)$$

$$q_{safe} = (q_{ult} + \gamma_{sat} D_f) / F.S.$$

Where,

γ_{sat} = Saturated Unit weight of the soil (KN/m^3)

Nc, Nγ& Nq are bearing capacity factors

B = Width of foundation (m)

D_f = Depth of foundation (m)

C = Cohesion (KN/m^2)

F.S. = Factor of safety i.e., 3

R_{w2} = Water correction factor

If soils have loose to medium denseness and soft to medium stiff consistency then the foundation fails according as the local shear failure (LSF) otherwise fails in general shear failure (GSF) criterion.

(ii) Using Meyerhof's (1965) & Bowles (1977) Correlation

$$q_{safe} = 11.98\ N_{60} \left(\frac{3.28B+1}{3.28B}\right)^2 f_d \left(\frac{S}{25}\right) x\ R_{w2} \quad KN/m^2\ \ldots\ldots(II)$$

Where,

N = Standard Penetration Value

B = width (m)

S = Settlement (mm)

f_d = $1+0.33\ (D/B) \leq 1.33$

R_{w2} = water correction factor = 0.5

TheB.C.valuesfromfieldandlabtestresultsforSpreadandMat/RaftFoundations are presentedasatabulatedformin the*annexes*withinpermissiblesettlements.

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Settlement:-

As ground strata are dominated by plastic silty soil and silty sand sands and layers just below the probable foundation depth. The strata are generally compressible for general loading condition thus; settlement analysis should be considered for the project site. So, the settlement of the foundation could be checked for maximum permissible values of 65/100mm for cohesive layers and 40/50 mm for semi cohesionless soil layers respectively.

For Cohesive Layer:

For heavier and important structures consolidation settlement should be predicted by the following equation;

SO_c = $Hi*Cc/(1+e_o)\log\{(P'o+P)/P'o\}$

Where, SO_c = Long term settlement, cm

H_i = Thickness of each layer

P'_o = Effective overburden pressure at the middle of each layer

C_c = Compression index

e_o = Initial void ratio

P = the excess pressure at the middle of each layer due to superposition of load.

So, total settlement for heavy structure in clayey soil is equal to the consolidation settlement derived above.

S_t = So_c

This total amount of settlement that will takes place continuously for hundred of years and should be lie within the ranges of permissible value (65/100mm).

For Semi-Cohesion and Cohesion less Layer:

$\Delta=2.84q/N[B/(B+0.3)]^2$ for B>1.25m Where, q is N/m^2 and B in meters.

The B.C. Values are found within the permissible values (40/50mm).

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Subgrade Modulus:-

The modulus of sub grade reaction is a conceptual relationship between pressure and deflection. It is defined as the ratio between the soil pressure and the corresponding settlement mathematically.

$$Ks = qn/Sv$$

Different researchers have suggested empirical approaches to get Ks.
Bowles method,

$$Ks = qnu/0.025$$
$$= 40qnu KN/m^3$$

The values of Ks are presented in the ***annexes***.

9. Seismicity

Peak Ground Acceleration:-

For the study, scenario of three earthquakes of different magnitude and setting are selected. Based on the seismic, seismo tectonic and geological condition, scenarios of these three earthquakes are compared with the large Bihar state of India - Nepal earthquake of 1934(Ms-8.4).The scenarios considered are Mid Nepal earthquake(Ms-8.0),North Bagmati earthquake (Ms-6.0) and KV Local earthquake (Ms-5.7).

Assuming Peak Ground Acceleration to Different Scales of Earthquakes

Name of Earthquake	Mid Nepal Earthquake	North Bagmati Earthquake	KV Local Earthquake	Bihar Nepal Earthquake
PGA(gal)	200-300	100-200	200-300	200-300
Ms	8.0	6.0	5.7	8.4

10. Liquefaction

Soil liquefaction is the major cause of damage to the foundation during an earthquake. Liquefaction potential depends upon factors, like the nature of shaking intensity, duration and

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material susceptibility to liquefaction. Liquefaction potential assessment is carried out in the following steps.

Estimation of liquefaction resistance of soil deposit, Estimation of maximum or equivalent cyclic shear stress is likely to be induced in the soil deposit during an earthquake.

The liquefaction potential of sand layer subjected to earthquake load is evaluated using the following equation by Seed et al. (1971)

$$\frac{\tau_{cyc}}{\sigma'_{v0}} \equiv 0.65 \frac{a_{\max}}{g} \frac{\sigma_{v0}}{\sigma'_{v0}} r_d$$

where τ_{cyc} = average cyclic shear stress developed in horizontal sand layer due to earthquake

σ_{vo} = effective overburden stress at a depth under consideration

σ_{vo}' = total overburden pressure

a_{max} = peak horizontal ground acceleration by the earthquake at ground surface

g = acceleration due to gravity

r_d = stress reduction factor (function of depth and rigidity of soil column)

The estimation of cyclic strength of soil deposit is based on the empirical correlations with Standard Penetration Test value. N value is corrected for effective overburden pressure of 1 ton/ft^2 and for further correlation to energy ratio of 60%, the following equation is used

$$(N_1)_{60} \equiv N_m C_N \frac{E_m}{0.60 E_{ff}}$$

N_m = measured SPT N value

C_N = overburden correction factor

E_m = actual hammer energy

E_{ff} = theoretical free fall hammer energy

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Based on the cyclic loading imposed by an earthquake, and liquefaction characteristics of soil, the liquefaction potential is evaluated. Liquefaction at any depth is expected where the loading by earthquake exceeds the resisting capacity of soil to liquefaction. The factor of safety against liquefaction is expressed as the ratio of cyclic shear stress required to cause liquefaction and equivalent cyclic shear stress induced by earthquake.

$$FS_l = \frac{\tau_{cyc,L}}{\tau_{cyc}} = \frac{CSR_L}{CSR}$$

In the present study for project site, the computation of the liquefaction susceptibility could be seen due to the presence of non-plastic sandy silts, loose to medium dense silty sands and sands etc. in upper layers. FSL indicates the factor of safety for liquefaction at corresponding depth; liquefaction is expected when FSL is less than 1.0. In this case the FSL values are lower than 1 upto 7.0m depth in BH-1 & BH-2 whereas 3.0m depth in BH-3 & BH-4 respectively. So, the sub surface layers of the project site could be liquefy up to 7.0 m depth.

11. Seismic Design for Foundation

The following relation expresses the seismic inertia force.

S_{if} = α_c M

Where,

a_c = seismic acceleration

M = mass of the body,. We know the weight of the body,

W = Mg

Where, g is the acceleration due to gravity (g = 9810 mm/sec^2)

S_{if} = α_cW/g

Let us introduce a new index characteristics α_o, which is basic horizontal seismic coefficient.

S_{if} = α_cW

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Each mass in a structure is assumed to be subjected to an equivalent force given by α_o times its weight at its own center of gravity in one horizontal direction at a time.

Earthquake Intensity, M	7	8	8.5	9
Seismic Coefficient, α_o	0.025	0.05	0.08	0.1
Max. Ground Accelerations	0.37 g	0.20.0 g	0.20.0 g	0.20.0 g

It shows that seismic acceleration is a fraction of acceleration due to gravity. To arrive at design coefficients, the basic values given above are multiplied by suitable factors β to take care of foundation conditions and importance of structures. The factor β for different soil conditions are given below.

Values of β for

Type of soils	**Isolated RCC footing without tie beams or unreinforced strip Foundations**	**For Mat Foundation**
Type I – rock or hard soils (N > 30)	1.0	1.0
Type II – Medium soils (10 < N ≤ 30)	1.2	1.0
Type III – Soft/loosesoils (N ≤ 10)	1.5	1.0

Nepal is located on high seismic zone. An earthquake intensity M, not less than 8.5 is to be adopted and accordingly the seismic coefficient for an earthquake structure.

Further, for this project sites, take type I, II and III as per the observed (N) value in different sub-surface layers to get β values for the project site.

12. Conclusion & Recommendation

Based on field and laboratory tests following inferences have been made:

i. Adopt a safe allowable bearing capacity for Isolated Spread or Mat/Raft Foundation at different depths of site location as given in the ***annexes.***

Spread & Mat foundation:

Depth (m)	Recommended Allowable B.C. Values (KN/m^2)					Subgrade Modulus, K_s (KN/m^3)
	2.0 x 2.0	3.0 x 3.0	4.0 x 4.0	6.0 x 6.0	8.0 x 8.0	
1.0	32.77	36.49	40.21	47.66	55.11	3932.29
2.0	56.99	61.65	66.32	75.62	84.94	6838.68
3.0	75.74	73.16	65.54	94.21	97.22	9088.54
4.0	104.96	96.03	91.71	138.96	133.31	12594.76

ii. The allowable B.C. value for different foundation size should be easily fulfilled the design loads for the foundation design.

iii. The Pile foundation is not considered for the design of foundations.

iv. The upper layer about 1.0m depth seems loose or soft soils in all locations. So, any foundation structure would not be considered up to that depth.

v. The most suitability of back filling or refilling soils must be clayey silts with sands (ML) or silty sands with fine gravels (SM).

vi. Those soil types in which the designed foundation pressures are not matched with the allowable B.C. values then adopt either changing the another foundation type or changing the soil from stronger depths up to the foundation level by silty gravels, sandy gravels having stone dust (aggregates, sand and stone dust) in 6:3:1 ratio about 70 - 80% degree of compaction in 200mm thick per layer considering the cost effectiveness.

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vii. The B.C. value is obtained for worst water condition could be found in rainy season.

viii. The soil Type III - Soft Soils is found in BH-1 & BH-2 whereas soil Type II – Medium Soils are seen in BH-3 & BH-4 respectively.

ix. The Sub-grade Modulus Ks (KN/m^3) are found as an average for each locations.

x. The proposed site is unsafe against liquefaction susceptibility upto 7.0m depths in BH-1 & BH-2 whereas 3.0m depths in BH-3 & BH-4 respectively. That can be minimized by providing 700mm dia. 7.0m length gravel pile/stone columns at 2.1m c/c spacing beneath the foundation structure at the sub-station project site.

xi. The Methodology and specification for installation of stone columns is attached herewith.

xii. The sub- soil water/ piezometric surface was found in all bore hole location. So, a common used preservative measures can be applied to the foundation structure.

xiii. All the assumed geotechnical values, relationships etc. are directly used as per requirement from the literature review of different papers, author's books and published journals.

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ANNEX-8

METHODOLOGY FOR INSTALLATION

METHODOLOGY AND SPECIFICATION FOR INSTALLATION OF STONE COLUMNS

1. GENERAL

As per the previous Soil Investigation works carried out at the substation site it is found liquefaction seismic hazards which contribute to the sinking or tilting of heavy structures, settlement of building, lateral spreading and retaining wall failure. Similarly, ground surface settlement, sand boils and post-earthquake stability failures can be developed at level ground surfaces.

Here a short mitigation measures adopting ground improvement techniques of stone columns/gravel piles has been found quite economical and faster in construction. The proposed unit minimizes the buildup of pore water pressure during earthquakes by shortening the drainage paths in a soil deposits. The installation of drains generally involves some degree of densification and the drains themselves may also provide some reinforcement. From the above assessment the site could be improved against liquefaction susceptibility.

2. Objective

The main objective of the installation of stone column/gravel piles are;

i) To dissipate the pore water pressure from the surrounding liquefiable soil mass through vertical drains.

ii) To increase strength (Bearing Capacity) by increasing the densities of the soil mass.

3. Scope of the work

The main scope of the works is:

i) The installation of stone columns/gravel piles by drilling, charging and compaction of crushed granular materials by any appropriate method.

ii) Measurement/ verification of soil improvement by fields and lab test results.

4. Methodology / Construction Techniques

The process of installation of stone column is,

I) Grading of Granular Materials

1) 50mm down manufactured graded coarse aggregate or natural river bed materials (pebbles, gravels, fine gravels and coarse to fine sands etc.) is used as a filling materials as a Gradation Table.

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Table

Size (mm)	% fine by weight
75	90 – 100
50	89 – 90
38	55 – 75
20	10 – 20
12	5 – 13
2	15
Sand	2 – 5 % by weight

2) Filling granular materials would be free from any harmful ingredients (i.e. vegetable matters, clay lumps, cobbles and boulders).

II) Making a Bore Holes

1) The bore holes are made by Helical Auger Boring or Bailer & Casing Method (IS 15284, Part I, Annex-c) throughout the mentioned 7.0m depth as per requirement.

a) Helical Auger Boring Method:

The bores are made with machine/manually operated augers supported on tripods. Bentonite slurry was provided while drilling whenever required to prevent caving of the bore hole sides. The bores are driven to a depth of 7.0m (as per drawing and instruction of project Engineer).

b) Bailer & Casing Method:

In case augers did not work then boring are done with bailers using a mechanical winch rotating by a powerful engine up to required 7.0m depths through steel casing / lining with bentonite slurry.

Finally, the bore hole should be cleaned/ cleared mechanically drilling before charging the filling granular materials.

III) Filling and Compaction of Granular Materials

1) The bore holes are charged by granular filling materials layer by layer having 30 cm ht. with regular $0.04m^3$ (two buckets) in each interval and then compaction is proceed by 300 kg or above wt. of hammer/rammer about 5 times from 1.5 m ht by mechanical winch fitted machine. The depth of filling shall vary as per type of structure footing to be constructed on it and instruction of project Engineer.
In case of temporary liner of bailer boring, compaction may be done in stages upto bottom of liner and should be raised and further compaction done.

2) The density of the compacted granular materials should be greater than 70% degree of compaction.

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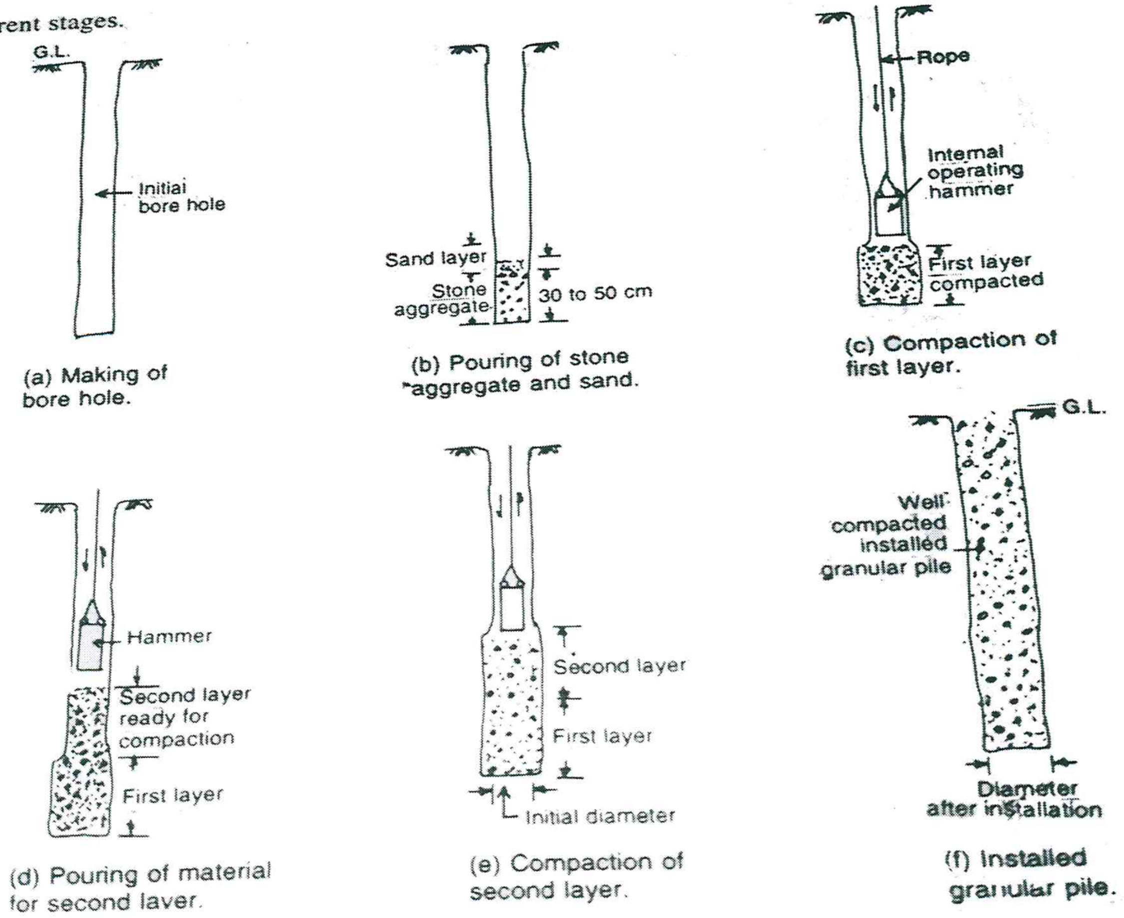
3) The site should be supervised and frequently checked as per standards by a well-known Geo-Technical Engineer during the period of construction.

4) After the installation of stone column the rig is moved to the another point to make a bore hole and that space is left open for the foundation work.

The complete process will be as given below sequentially in a figure:

different stages.



Granular pile installation method

5. Analysis of Installation / Results and Recommendation

Problems:

- From the liquefaction analysis report the liquefaction susceptibility has been seen upto 7.0m depth in sub-station site area.

Mitigation Measures:

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- Stone piling work shall be carried out under the foundations of Towers, Power Transformer, heavy equipment structures and buildings. The spacing of piles shall be @2.1mC/C under heavy structures footing as per foundation layout plan during construction.
- Stone Pile cutoff level shall be 300mm below the founding level. Above stone pile cut off level a compacted layer of 300mm thick graded granular material mixed with local sand shall be provided.

6. Verification of Test Report

The following tests will be carried out in every 250 m^2 in the presence of Consultant/NEA Engineer and later verified accordingly.

- Initial and routine test shall be carried out as per IS: 15284- Part1- clause No: 13.4 & 13.7 (k) after completion of stone piling work.
- Others essential field and laboratory test such as lab test of gradation of granular material, permeability of surrounding soil, compaction test and field test for relative density and SPT/CPT test shall be carried out as per BS/IS or equivalent international standard.

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