# Geotechnical Investigation Report

**Name of Work :**

Consultancy Services for Carrying out Engineering Studies for Remodelling of Inplant Ukai Thermal Power station, connectivity from Ukai – Songadh Station & GSECL Inplant yard at Ukai Songadh station (11.5 km Approx) in Mumbai Division of the Western Railway.

Gujarat Rail Infrastructure Development Corporation Limited

**Borehole No. :**

Bore-1

**Report No. :**

BIPL/UKAI_Yard/01

**Date of Report :**

15-07-2025

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# List of Annexure

## ABBREVIATION

C:- Cohesion
DS:- Disturbed Sample
UDS:- Undisturbed Sample
SPT:- Standard Penetration Test
WS:- Wash Sample
CR:- Core Recovery
RQD:- Rock Quality Designation
GWT:- Ground Water Table
EGL:- Existing Ground Level
SBC:- Safe Bearing Capacity
BH:- Borehole
FOS:- Factor of Safety
Y:- Density of Soil
LL:- Liquid Limit
PL:- Plastic Limit
SL:- Shrinkage Limit
PI:- Plasticity Index
NP:- Non-Plastic
DST:- Direct Shear Test
TUU:- Triaxial Test (UU Condition)
UCS:- Unconfined Compressive Strength
CT:- Compression Test

## IS CLASSIFICATION

G:- Gravel
S:- Sand
M:- Silt
C:- Clay
GW:- Well Graded Gravels
GP:- Poorly Graded Gravels
GM:- Silty Gravels
GC:- Clayey Gravels
GW-GM:- Well Graded Silty Gravels
GW-GC:- Well Graded Clayey Gravels
GP-GM:- Poorly Graded SiltyGravels
GP-GC:- Poorly Graded Clayey Gravels
SW:- Well Graded Sands
SP:- Poorly Graded Sands
SC:- Clayey Sands
SM:- Silty Sands
SW-SM:- Well Graded Silty Sands
SW-SC:- Well Graded Clayey Sands
SP-SM:- Poorly Graded Silty Sands
SP-SC:- Poorly Graded Clayey Sands
ML:- Inorganic Silt with Non to low Plasticity
CL:- Inorganic Clay with low Plasticity
MI:- Inorganic Silt with Non to Medium Plasticity
CI:- Inorganic Clay with Medium Plasticity
MH:- Inorganic Silt with Non to High Plasticity
CH:- Inorganic Clay with High Plasticity

## REFERENCES

| IS Code | Description |
|---|---|
| IS 1498 | Classification and identification of soils for general engineering purposes |
| IS 1892 | Code of practice for subsurface investigation for foundations |
| IS 1904 | Code of practice for design and construction of foundations in soils: General requirements |
| IS 2131 | Method of standard penetration test for soils |
| IS 2132 | Code of practice for thin-walled tube sampling of soils |
| IS 2720 (P-1) | Methods of test for soils: Part 1 Preparation of dry soil samples for various tests |
| IS 2720 (P-2) | Methods of test for soils: Part 2 Determination of water content |
| IS 2720 (P-3/Sec-1) | Methods of test for soils: Part 3 Determination of specific gravity, Section 1 Fine grained soils |
| IS 2720 (P-3/Sec-2) | Methods of test for soils: Part 3 Determination of specific gravity, Section 2 Fine, medium and coarse-grained soils |
| IS 2720 (P-4) | Methods of test for soils: Part 4 Grain size analysis |
| IS 2720 (P-5) | Methods of test for soils: Part 5 Determination of liquid and plastic limit |
| IS 2720 (P-6) | Methods of test for soils: Part 6 Determination of shrinkage factors |
| IS 2720 (P-10) | Methods of test for soils: Part 10 Determination of unconfined compressive strength |
| IS 2720 (P-11) | Methods of test for soils: Part 11 Determination of the shear strength parameters of a specimen tested in unconsolidated undrained triaxial compression without the measurement of pore water pressure |
| IS 2720 (P-13) | Methods of test for soils: Part 13 Direct shear test |
| IS 2720 (P-14) | Methods of test for soils: Part 14 Determination of density index (relative density) of cohesionless soils |
| IS 2720 (P-15) | Methods of test for soils: Part 15 Determination of consolidation properties |
| IS 2720 (P-40) | Methods of test for soils: Part 40 Determination of free swell index of soils |
| IS 2720 (P-41) | Methods of test for soils: Part 41 Measurement of swelling pressure of soils |
| IS 6403 | Code of practice for determination of bearing capacity of shallow foundations |
| IS 8009 (P-1) | Code of practice for calculation of settlements of foundations: Part 1 Shallow foundations subjected to symmetrical static vertical loads |

# 1. INTRODUCTION

**Gujarat Rail Infrastructure Development Corporation Limited** proposed to conduct "Consultancy Services for Carrying out Engineering Studies for Remodelling of Inplant Ukai Thermal Power station, connectivity from Ukai – Songadh Station & GSECL Inplant yard at Ukai Songadh station (11.5 km Approx) in Mumbai Division of the Western Railway." Accordingly, land soil investigations were planned to assess various soil parameters for engineering analysis and foundation design.
Soil investigations at the proposed Railway Yard.

Broad objectives of the investigation are as follows,

a) To evaluate the parameters of soil at the proposed site.

b) To assess the engineering parameters and to estimate the safe bearing capacity of soil.

# 2. FIELD WORK

The investigation was planned to obtain the subsurface stratification in the proposed project site and collect soil / rock core samples for laboratory testing to determine the engineering properties such as shear strength, along with basic engineering classification of the subsurface stratum.

For geotechnical investigation work, required equipements along with rotary drilling rigs and manpower were mobilized at site to carry out various field activities as per the scope of work. These were shifted from one test location to another location during execution of field work and were demobilized on satisfactory completion of field work.

## 2.1 Boring

### 2.1.1 Soil Boring

In soil, boreholes of 150mm dia. were drilled as per the standard procedure laid in IS: 1892. Wherever rocky strata was encountered before achieving the required depth below E.G.L., bore holes were further extended beyond refusal through soft/ hard rock using Hydraulic drilling Rigs with Nx size double tube core barrel / diamond bits upto required depth below the EGL and were terminated as per the instruction of Engr. in-charge.
Borehole was properly cleaned before taking any sample in soil. Casing was used as per the prevailing soil conditions, to stabilize the borehole. Standard Penetration Tests were conducted in bore holes at regular intervals or at every change of strata as per Technical specification. Undisturbed / disturbed samples were collected wherevery feasible as per the requirements and at specified depths / levels. The same has been discussed in detail in soil characteristics sheets attached with the report. Water table was met in the boreholes noted in borelog sheet.

### 2.1.2 Rock Boring

Core samples of Rock were collected to obtain the engineering properties of rock. The colour of water and water loss during drilling is also observed. Core Recovery and Rock Quality Designation obtained during drilling for each run and drilling note is noted and given in bore log. Details of boring with soil/rock profile, SPT values along with sampling details are given in the respective bore logs in the appendix of the report.
In case of medium to coarse grained, non-cohesive sand or Highly weathered rock where core recovery not possible where sampling is unsuccessful, disturbed sample was taken from the cleaning the borehole. Disturbed samples were also collected in polythene bags with proper labels during drilling for finding index properties of the samples.

## 2.2 Sampling

### 2.2.1 Disturbed Samples

Disturbed samples were collected during the boring and also from the split spoon sampler. The samples recovered were logged, labelled and placed in polythene bags and sent to laboratory for testing.

### 2.2.2 Undisturbed Samples

Undisturbed samples were collected using MS tubes of suitable diameter and length with Area ratio as per clause 4.1.1 (c) of IS: 1892 (latest) fitted to an adopter with ball and socket arrangement. Before taking any sample, sampling tube was properly greased. Immediately after taking on undisturbed sample in a tube, the adopter head was removed along with the disturbed material. The visible ends of the sample were trimmed off any wet disturbed soil. The ends were coated alternately with four layers of just molten wax. More molten wax was added to give a total thickness of min. 25 mm. The samples were carefully labeled and transported to the laboratory for testing. Undisturbed samples wherever slipped during lifting were duly marked in the field logs as well as in the soil profile.

### 2.2.3 Standard Penetration Test

The Standard Penetration Tests (SPT) (IS-2131, 1981) was carried out in the bore hole at predetermined depths. It gives indirect evaluation of strength–deformation characteristics of the sub soil. The test includes driving a split spoon sampler using a 63.5 kg hammer with a free fall of 750mm. The first 15cm is considered as seating drive. The No. of blows required to penetrate next 30 cm is reported as N-value.
Empirical relations are established to correlate N-Value with the shear parameters or bearing capacity of soil. A disturbed soil sample is collected inside the split spoon sampler which can be used to find soil classification and In-situ water content. If the no. of blows exceeds 50 before desired penetration is achieved, it is reported as N-value >50 with the actual achieved.

**Table 1: Soil compactness as per SPT N values (cl. 9.7, table 9.3 & 9.4, page 330_text book of V.N.S. Murthy)**

**Co-relation of SPT (N) value for Cohesionless Soil**

| SPT (N) value | Consistency of Soil |
|---|---|
| 0-4 | Very Loose |
| 4-10 | Loose |
| 10-30 | Medium |
| 30-50 | Dense |
| > 50 | Very Dense |

**Co-relation of SPT (N) value for Cohesive Soil**

| SPT (N) value | Consistency of Soil |
|---|---|
| 0-2 | Very Soft |
| 2-4 | Soft |
| 4-8 | Medium |
| 8-15 | Stiff |
| 15-30 | Very Stiff |
| > 30 | Hard |

The field SPT N values obtained were further corrected as per the guidelines given in IS: 2131 as follows:
(a) **For overburden: -** The N value for cohesionless soil is corrected with the help of fig. 1 given in IS-2131.
(b) **Due to dilatancy :-** Wherever N values observed below water table in fine sand, silty sand or silt was greater than 15, then corrected N values were corrected as under:

N' =15 + ½ (N-15)

### 2.2.4 Rock Drilling and Core Samples

Drilling was advanced by rotary core drilling method using double tube core barrels as per the guidelines of IS: 6926-1996. A double tube core barrel and Nx sized bits are used for drilling and recovering rock cores. Core Samples were extracted by the application of a continuous pressure at one end of the core with the barrel held horizontally without vibration. Immediately after withdrawal from the core barrel, the cores were placed in a tray and transferred into boxes specially prepared for the purpose. The boxes are made of Aluminum. Recovered rock cores were numbered serially as specified in IS: 4078-1980. Rock core recovery and Rock Quality Designation were computed for every run length drilled. The description of the core samples was recorded. Rock core recovered during the NX size rock drilling have been measured, numbered and packed in wooden core boxes.

## 2.3 Field Investigation Data

**Table 2: Borehole Summary**

| | |
|---|---|
| **Start Date** | 22-06-2025 |
| **Terminate Date** | 22+-06-25 |
| **Method of Boring and Drilling rigs** | Machine Drilling Diesel Engine (Kelex) |
| **Termination Depth below EGL(m)** | 7.5 m |
| **Ground Water Table** | 0.5 m |
| **Location** | 21°12'32.4"N 72°32'49.8"E |

**Table 3: Field Borelog Sheet**

| Depth (m) | Sample type | Depth(m) | | SPT N-value | | | | Remarks |
|---|---|---|---|---|---|---|---|---|
| | | from | to | $N_1$ | $N_2$ | $N_3$ | $N = N_2 + N_3$ | |
| **0.0** | DS | --- | 0.0 | --- | --- | --- | --- | --- |
| **1.5** | DS | 0 | 1.5 | --- | --- | --- | --- | --- |
| **3.0** | DS | 1.5 | 3.0 | --- | --- | --- | --- | --- |
| **4.5** | Core | 3 | 4.5 | --- | --- | --- | --- | --- |
| **6.0** | Core | 4.5 | 6.0 | --- | --- | --- | --- | --- |
| **7.5** | Core | 6 | 7.5 | --- | --- | --- | --- | --- |

# 3. LABORATORY WORK

Following laboratory tests are carried out to determine the physical and engineering properties of undisturbed and disturbed soil samples.

**Table 4: Summary of Laboratory test to be carried out on samples of Soil**

| Laboratory Test | IS Code |
|---|---|
| Dry Density and Natural Moisture Content | IS 2720, Part-2, 1973 |
| Particle Size Analysis | IS 2720, Part-4, 1985 |
| Atterberg's Limit | IS 2720, Part-5, 1985 |
| Shrinkage Limit | IS 2720, Part-6, 1972 |
| Free Swell Index | IS 2720, Part-40, 1977 |
| Specific Gravity | IS 2720, Part-3.1, 1980 |
| Direct Shear Parameters | IS 2720, Part-13, 1986 |
| Triaxial Shear Parameters | IS 2720, Part-11, 1993 |
| Consolidation Test of Soil | IS 2720, Part-15, 1965 |

# 4. PHYSICAL PROPERTIES OF SOIL

## 4.1 Natural Moisture Content & Field Dry Density

The weight of undisturbed soil sample with sampler (Shelby tube) is determined after removing paraffin wax and loose soil. The total length of soil sample recovery is determined after deducting empty length from the total length of sampler. The volume of soil mass retained in sampler is thus determined from the known inside diameter of sampler and total length of soil mass. The soil mass is then removed and the average moisture content is determined by keeping the soil sample along with crucible in oven at 105°C to 110°C for 24 hours. The empty weight of the sampler is then found out. From the total weight of sampler with soil mass, the weight of empty sampler is deducted.

## 4.2 Particle Size Analysis

The sieve analysis is carried out in accordance with IS-2720, Part-IV, 1985. The results are presented in the form of Grain size distribution curve.

❖ **Soil fraction passing 4.75 mm IS Sieve**

The portion of the soil passing 4.75 mm ISS is oven dried at 105°C to 110°C. The portion is coned & quartered to obtain required representative quantity of the material. The material is weighed and. placed in tray/bucket filled with water for soaking and loosening the adhered cohesive materials. The soaked soil specimen is then washed on 75 microns IS Sieve until the water passing the sieve is almost clear. The material retained on 75 microns IS Sieve is then transferred in a tray, dried in oven.

Sieve analysis is then conducted on a nest of sieves (viz. 2 mm, 425- and 75-micron ISS) either by hand or by using mechanical sieve shaker. The fraction retained on each of the sieves is weighed separately and masses recorded. Cumulative mass of soil fraction retained on each sieve is then calculated. The weights are then converted into cumulative percentage retained and passing on the basis of the mass of the sample passing 4.75 ISS taken. The combined gradation on the basis of the total sample taken for analysis is finally calculated.

❖ **Indian Standard Soil Classification (as per IS:1498)**

The classification of soil by particle size involves separating soil particles into different groups based on their diameters. The main categories are:

**Gravel:** Gravel consists of coarse particles with diameters greater than 4.75 millimeters (mm). These particles are typically formed from rock fragments and are further subdivided into coarse gravel (particles ranging from 20 mm to 80 mm) and fine gravel (particles ranging from 4.75 mm to 20 mm).

**Sand:** Sand particles have diameters ranging from 0.075 mm to 4.75 mm. Sand is further divided into coarse sand (particles ranging from 2 mm to 4.75 mm), medium sand (particles ranging from 0.425 mm to 2 mm), and fine sand (particles ranging from 0.075 mm to 0.425 mm).

**Silt:** Silt particles are finer than sand, with diameters ranging from 0.002 mm to 0.075 mm. Silt feels smooth to the touch and retains moisture better than sand.

**Clay:** Clay particles are the finest soil particles, with diameters less than 0.002 mm. Clay has high plasticity and cohesive properties, making it sticky when wet and hard when dry.

**Table 5: Classification of Soils according to Indian Standard**

| Description | | | Size |
|---|---|---|---|
| Very Coarse Grain Soils | Boulder | | > 300 mm |
| | Cobble | | 80 - 300 mm |
| Coarse Grain Soils | Gravel (G) | Coarse | 20 - 80 mm |
| | | Fine | 4.75 - 20 mm |
| | Sand (S) | Coarse | 2 - 4.75 mm |
| | | Medium | 0.425 - 2 mm |
| | | Fine | 0.075 - 0.425 mm |
| Fine Grain Soils | Silt (M) | | 0.002 - 0.075 mm |
| | Clay (C) | | < 0.002 mm |

## 4.3 Atterberg's Limit

Atterberg limits provide insights into the water content-dependent characteristics of fine-grained soils such as clay and silt. These assessments, comprising Plastic Limit (PL) and Liquid Limit (LL) tests, identify the moisture thresholds at which the soil transitions from a solid to plastic (PL) state and from plastic to liquid (LL) state, respectively. The Plasticity Index (PI), derived from the difference between LL and PL, indicates the moisture range within which the soil displays plastic behavior. Atterberg limits play a pivotal role in soil classification, anticipating potential issues like shrinkage and swelling, and informing appropriate construction practices tailored to various soil compositions.

### 4.3.1 Liquid Limit

The cylindrical cup of cone penetrometer ensuring that no air is trapped in this process. Finally, the wet soil is levelled up to the top of the cup and placed on the base of the cone penetrometer apparatus. The penetrometer shall be adjusted that the cone point just touches the surface of the soil paste in the cup clamped obtained 200 gm in of soil sample shall be worked well into a paste with addition of distil water. In the case of highly clayey soils, to ensure uniform moisture distribution, it is recommended that the soil in the mixed state is left for sufficient time (24 hours) in an air-tight container. The wet soil paste shall then be transferred in to in this position. The initial reading is either adjusted to zero or noted down as is shown on the graduated scale. The vertical clamp is then released allowing the cone to penetrate in to the soil paste under its own weight.

The penetration of the cone after 5 sec. shall be noted to the nearest millimetres. If the difference in penetration lies between 16 and 26 mm, the test is repeated with suitable adjustment to moisture either by addition of more water or exposure of the spread paste on a glass plate for reduction in moisture content. The exact moisture content of each trial shall be determined.

The liquid limit water content is computed from any one of the following relationships:

$$W_L = W_W / [0.65 + (0.0175 \times D)]$$

where,

$W_w$ = Water Content of the soil sample

D = depth of Penetration

### 4.3.2 Plastic Limit

For determination of plastic limit, a soil sample weighing at least 20 gm from the soil sample passing 425 microns IS sieve is thoroughly mixed with water such that it can be easily moulded with fingers. A ball is formed with about 8 to 10 gm of this soil & is rolled between the fingers and the glass plate with just sufficient pressure to roll the mass into a thread of uniform diameter of 3mm throughout its length. The soil is then kneaded together to a uniform mass and rolled again. The process is continued until the thread crumbles. The pieces of crumbled soil thread are collected and moisture content is determined and reported as plastic limit.

## 4.4 Specific Gravity

The specific gravity of soil solids is determined by a 50ml density bottle. The weight (W1) of the empty dry bottle is taken first. A sample of oven-dried soil about 10-20 gm cooled in a desiccator, is put in the bottle, and weight (W2) of the bottle and the soil is taken. The bottle is then filled with distilled water gradually removing the entrapped air either by applying vacuum or by shaking the bottle. The weight (W3) of the bottle, soil and water (full up to the top) is then taken. Finally, the bottle is emptied completely and thoroughly washed and clean water is filled to the top and the weight (W4) is taken,

$$\text{Specific Gravity (G)} = (W_2-W_1) / [(W_2-W_1) -(W_3-W_4)]$$

# 5. SHEAR PROPERTIES OF SOIL

The shear test is a common geotechnical test used to measure the shear strength of a soil sample by applying a transverse load. This test helps evaluate crucial soil properties like stability, slope stability, and foundation bearing capacity, impacting various construction projects. Data collected throughout the test allows engineers to calculate the soil's shear strength and other important properties. Understanding soil shear strength is vital for designing and constructing safe and stable structures.

In general, shear tests were conducted using three methods.

### 5.1 Triaxial Shear Test

Triaxial shear (Quick) tests are conducted to determine the shear parameters of clayey samples. For unconsolidated undrained (Quick) Triaxial compression test, the specimen having diameter 38mm and height to diameter ration of 2 is prepared and placed on the pedestal of the triaxial cell after enclosing it in rubber membrane. The cell is then assembled with the loading ram and then placed in the loading machine. The fluid is admitted to the cell and the pressure is raised to the desired value. Initial reading of the gauge specimen is recorded. The test is then commenced and sufficient number of simultaneous readings of load and compression measuring gauge being taken. The test is continued until the maximum value of the stress has been reached or until an axial strain of 20 percent has been attained. The test is carried out at confining stress of 0.5, 1.0 and 1.5 kg/cm2.

### 5.2 Direct Shear Test

Test procedure shall be as per IS: 2720 (Part 13). In this test the soil specimen is confined in a square metal box split into two halves horizontally. Perforated metal plates and pores stones are placed above and below the specimen to allow free drainage. After applying vertical load the soil is gradually sheared by applying horizontal force, which makes the two halves of the box move relative to each other. The shear is applied at a constant strain rate and measured in a proving ring. Vertical deformation is measured with dial gauge. Minimum of three specimens are tested and the graph drawn with normal stress in ‘X’ axis and shear stress in ‘Y’ axis. From the straight line plot, values of C and Φ are measured.

## 6. PHYSICAL PROPERTIES OF ROCK

Rock classification in terms of weathering and state of fractures and strength is carried out in the following manner. Tabulations given in below explain it briefly.

### 6.1 Scale of Weathering Grades of Rock Mass (IS: 4464)

As per IS: 4464, It should be understood that all grades of weathering may not be seen in a given rock mass and that in some cases a particular grade may be present to a very small extent. Distribution of the various weathering grades of rock material in the rock mass may be related to the porosity of the rock material and the presence of open discontinuities of all types in the rock mass.

**Table 6: Weathering Grades of Rock Mass as per Geologist Interpretation**

| Terms | Description | Grade | Geologist Interpretation |
|---|---|---|---|
| Fresh | No visible sign of rock material weathering ; perhaps slight coloration on major discontinuity surfaces | I | CR > 90 % |
| Slightly Weathered | Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering. | II | CR between 70 % to 90 % |
| Moderately Weathered | Less than half of the rock material is decomposed or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as core-stones. | III | CR between 51 % to 70 % |
| Highly Weathered | More than half of the rock material is decomposed or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as core-stones. | IV | CR between 11 % to 50 % |
| Completely weathered | All rock material is decomposed and /or disintegrated to soil. The original mass structure is still largely intact. | V | CR between zero to 10 % |
| Residual Soil | All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported | VI | CR = 0 % But N > 50 |

## 6.2 Relation between RQD and In-Situ Rock Quality, IS: 13365 (Part-1), Annex-B

Relation between RQD and In-situ Rock quality is further measured by frequency of natural joints in rock mass. Rock Quality Designation (RQD) is used to define state of fractures or massiveness of rock. Following table defines the quality of rock mass.

**Table 7: Rock Classification as per RQD**

| RQD classification | RQD % |
|---|---|
| Excellent | 91-100 |
| Good | 76-90 |
| Fair | 51-75 |
| Poor | 25-50 |
| Very Poor | 0-25 |

**Table 8: Rock Compressive Strength from Type of Rock**

| Rock type | Unconfined compressive strength in Mpa |
|---|---|
| Extremely strong | >200 |
| Very Strong | 100-200 |
| Strong | 50-100 |
| Moderately Strong | 12.5-50 |
| Moderately Weak | 5-12.5 |
| Weak | 1.25-5 |
| Very weak | <1.25 |

# 7. COMPUTATION OF SOIL BEARING CAPACITY

## 7.1 Safe Bearing Capacity Based on Shear Criteria:

For Shear Criteria IS-6403 Ultimate Bearing Capacity Equation is used based on laboratory shear parameters. A factor of safety (F) = 2.5 against shear failure.

For soils, containing both coarse grained (gravels & sands) and fine grained (clays), **c** and ϕ are used to determine the soil strength. In case of predominantly fine grained soils, **c** and ϕ are determined by the Triaxial Compression test as per IS: 2720 pt XI. For predominantly coarse grained soils, **c** and ϕ are determined by Direct Shear test as per IS: 2720 pt XIII. These **c** and ϕ values were used for determining the SBC of soil as per shear failure criteria.

Bearing Capacity Equation: Shear Criteria (IS 6403 – 1981)

The ultimate net bearing capacity in case of general shear failure is given by (from IS 6403 : 1981, page No. 8)

$$\mathbf{q_d} = c\, N_c\, s_c\, d_c\, i_c + q\,(N_q\text{-}1)\, s_q\, d_q\, i_q + (1/2)\,\gamma\, B\, N_\gamma\, s_\gamma\, d_\gamma\, i_\gamma\, w']$$

The ultimate net bearing capacity in case of local shear failure is given by (from IS 6403 : 1981, page No. 8)

$$\mathbf{q'_d} = (2/3)\, c\, N'_c\, s_c\, d_c\, i_c + q\,(N'_q\text{-}1)\, s_q\, d_q\, i_q + (1/2)\,\gamma\, B\, N'_\gamma\, s_\gamma\, d_\gamma\, i_\gamma\, w'$$

**Note:** The type of failure used for bearing capacity analysis depends upon the value of void ratio (see IS 6403 : 1981, Page No. 9, Table No. 3).

Where,

$q_d$ & $q'_d$ = Ultimate net bearing capacity

Nc, Nq, Nγ = Bearing Capacity factors based on General Shear Failure for c,

N'c, N'q, N'γ = Bearing Capacity factors based on Local Shear Failure for c',

B = Width of footing,

D = Depth of footing,

γ = unit weight of soil,

w' = 0.5 {1 + (Dw –D) / B} & if Dw < Df, w' = 0.5 & if Dw > (D + B), w' =1.0 = 0.50 for GWT at and above Footing Level

Dw = depth of GWT from Ground Level

Sc, Sq, Sγ = Shape factors

ic, iq, iγ = inclination factors =1 for vertical loads.

dc = 1 +0.2 Df / B * √Nq

dq and dγ = 1 for ϕ < 10°

dq, dγ = 1 +0.1 Df / B * √Nq for ϕ >10°

## 8. EFFECT OF WATER TABLE FLUCTUATION

The basic theory of bearing capacity is derived by assuming the water table to be at great depth below and not interfering with the foundation. However, the presence of water table at foundation depth affects the strength of soil. Further, the unit weight of soil to be considered in the presence of water table is submerged density and not dry density. Hence, the reduction coefficients RW1 and RW2 are used in second and third terms of bearing capacity equation to consider the effects of water table.

## 9. FACTORS INFLUENCING BEARING CAPACITY

Bearing capacity of soil depends on many factors. The following are some important ones.

1. Type of soil
2. Unit weight of soil
3. Surcharge load
4. Depth of foundation
5. Mode of failure
6. Size of footing
7. Shape of footing
8. Depth of water table
9. Eccentricity in footing load
10. Inclination of footing load
11. Inclination of ground
12. Inclination of base of foundation

## 10. UNCERTAINTY CLAUSE

1) The Soil exploration and Geotechnical Investigation was carried out at described site to identify and evaluate the soil parameters.

2) Site condition and Soil strata may differ and variate during the execution of actual work due to uncertainty. This ambiguity may exist due to the difference between location of Soil Exploration and Construction Site. Moreover, It may due to the weather condition, selection of sampling method, Methods of soil exploration, field test parameter and Test conducted.

# Summary of Allowable Safe Bearing Capacity

**Table 9: Summary of Suggested Allowable Bearing Capacity of Soil**

| Size of Footing | Depth Below Ground Level (m) | Safe Bearing Capacity based on Shear Criteria C and ϕ value ($t/m^2$) (1) | Safe Bearing Capacity based on Settlement Criteria ($t/m^2$) (2) | Allowable Bearing Capacity suggested for 25mm settlement (Minimum Value of 1 and 2) ($t/m^2$) |
|---|---|---|---|---|
| Square footing (2m x 2m) | 1.50 | 13.9 | 20.1 | 13.9 |
| | 3.00 | 28.1 | 29.7 | 28.1 |
| Square footing (3m x 3m) | 1.50 | 14.6 | 16.9 | 14.6 |
| | 3.00 | 27.5 | 28.2 | 27.5 |
| Square footing (4m x 4m) | 1.50 | 15.5 | 15.0 | 15.0 |
| | 3.00 | 27.8 | 25.1 | 25.1 |

**Table 10: Summary of Safe bearing capacity of Rock Core (Abutment 1)**

| Sr No. | Depth of Rock Sample (m) | Core Recovery (%) | Rock Quality Designation (%) | Safe bearing Pressure from Rock Core strength (T/m2) |
|---|---|---|---|---|
| 1 | 4.50 | 5.33 | Nil | 63.73 |
| 2 | 6.00 | 8.00 | Nil | 79.54 |
| 3 | 7.50 | 24.33 | 24.33 | 114.72 |

# CONCLUSION & RECOMMENDATION

1). The borelog indicates that the top layer predominantly consists of non-plastic, coarse-grained soils, primarily composed of sand and gravel. Below this layer, rock strata are encountered, indicating a transition from granular soil to bedrock conditions with depth.

2). The S.B.C. are worked out for various size of footings at various Depths are suggested in Summary of Soil Safe Bearing Capacity.

3). The results of the Laboratory tests are incorporated in the form of table at the later part of the Report.

4). S.B.C. is Recommended by considering all the parameters affecting to soil At the field as well as in the Laboratory

5). After completion of work, Extra care should be taken to prevent water percolation below footing by plinth coverage.

**Note:-**

**1)** The Soil exploration and Geotechnical Investigation was carried out at described site to identify and evaluate the soil parameters.

**2)** Site condition and Soil strata may differ and variate during the execution of actual work due to uncertainty. This ambiguity may exist due to the difference between location of Soil Exploration and Construction Site. Moreover, It may due to the weather condition, selection of sampling method, Methods of soil exploration, field test parameter and Test conducted.

Therefore, it is recommended that the SBC values to be used appropriately according to the type of structures, Application of loading conditions, As per foundation depth, and the characteristics of the soil strata at that depth.

## ANNEXURE 1: BORELOG DATA SHEET

Name of Work: Consultancy Services for Carrying out Engineering Studies for Remodelling of Inplant Ukai Thermal Power station, connectivity from Ukai – Songadh Station & GSECL Inplant yard at Ukai Songadh station (11.5 km Approx) in Mumbai Division of the Western Railway.

| Gujarat Rail Infrastructure Development | Report No.: UKAI_Yard/01 | Water Table: 0.50 m | Type of Boring: Machine Drilling |
|---|---|---|---|

| Borehole No: Bore-1 | Termination Depth: 7.5 m | SPT: N Value | Core Recovery (%) | RQD (%) |
|---|---|---|---|---|

| Depth from EGL (m) | Description of Sample | Symbol/ Hatching | Thickness of Strata(m) | Sampling Type | Sample Depth (m) | $N_1$ | $N_2$ | $N_3$ | N = $N_2$ + $N_3$ | Core Recovery (%) | RQD (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | Filled up Material | | **0.50** | DS | 0.00 | --- | --- | --- | --- | --- | --- |
| GWT 1.00 | Brown, Poorly Graded Sand with Gravel, Coarse-Grained Soil |  **SP** | **3.00** | DS | 1.50 | --- | --- | --- | --- | --- | --- |
| 2.00 | | | | DS | 3.00 | --- | --- | --- | --- | --- | --- |
| 3.00 | | | | | | | | | | | |
| 4.00 | Brownish to Black, Rock Fragments of Gravel Size, Soft to Medium Hard Completely weathered and broken rock pieces |  **ROCK** | **4.50** | Core | 4.50 | --- | --- | --- | --- | 5.33 | Nil |
| 5.00 | | | | Core | 6.00 | --- | --- | --- | --- | 8.00 | Nil |
| 6.00 | | | | | | | | | | | |
| 7.00 | Dark Gray, Highly Weathered, Moderately Hard Rock | | | Core | 7.50 | --- | --- | --- | --- | 24.33 | 24.33 |
| 7.50 | | | | | | | | | | | |

**Abbreviation**: **EGL**-Existing Ground Level, **SPT**-Standar Penetration Test, **DS**-Disturbed Sample, **UDS**-Undisturbed Sample, **WS**-Wash Sample, **NA**-Not Applicable

**Note**:- 1) The Soil exploration and Geotechnical Investigation was carried out at described site to identify and evaluate the soil parameters. 2) Site condition and Soil strata may differ and variate during the execution of actual work due to uncertainty. This ambiguity may exist due to the difference between location of Soil Exploration and Construction Site. Moreover, It may due to the weather condition, selection of sampling method, Methods of soil exploration, field test parameter and Test conducted.

# ANNEXURE 2: LABORATORY TEST RESULTS

Name of Work: Consultancy Services for Carrying out Engineering Studies for Remodelling of Inplant Ukai Thermal Power station, connectivity from Ukai – Songadh Station & GSECL Inplant yard at Ukai Songadh station (11.5 km Approx) in Mumbai Division of the Western Railway.

| Gujarat Rail Infrastructure Development Corporation Limited | Borehole No: Bore-1 | Termination Depth: 7.5 m | Type of Boring: Machine Drilling | Water Table: 0.50 m |
|---|---|---|---|---|

| Sample Depth | Sampling Type | SPT Value Number | Specific Gravity (G) | Field Data | | | Grain Size Analysis ( IS:2720, P-4 ) | | | Atterberg's Limits ( IS:2720, P-5 ) | | | I.S. Soil Classification ( IS:1498) | Shear Parameter of Soil | | | Rock Parameter | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Bulk Density ($Y_b$) | Dry Density ($Y_d$) | Moisture Content | Gravel (G) | Sand (S) | Silt (M) & Clay (C) | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) | | Type Of Shear Test | Cohesion (C) | Angle Of Int. Friction, (Φ) | Core Recovery | Rock Quality Designation | Grade | Water Absorption Test | Density of Rock | Uniaxial Compressive Strength |
| (m) | - | - | - | (gm/cm$^3$) | | (%) | (%) | (%) | (%) | (%) | (%) | (%) | - | - | (Kg/cm$^2$) | ( $^0$ ) | (%) | (%) | - | (%) | (gm/cm$^3$) | (N/mm$^2$) |
| 1.50 | DS | --- | 2.65 | --- | --- | 24.4 | 35.5 | 64.5 | 0.0 | --- | NP | NP | SP | DST | 0.00 | 32 | --- | --- | --- | --- | --- | --- |
| 3.00 | DS | --- | 2.67 | --- | --- | 25.7 | 42.8 | 57.2 | 0.0 | --- | NP | NP | SP | | --- | --- | --- | --- | --- | --- | --- | --- |
| 4.50 | Core | --- | 2.72 | --- | --- | --- | --- | --- | --- | --- | --- | --- | Rock | UCS | --- | --- | 5.33 | Nil | V | 0.25 | 2.44 | 6.25 |
| 6.00 | Core | --- | 2.75 | --- | --- | --- | --- | --- | --- | --- | --- | --- | Rock | | --- | --- | 8.00 | Nil | V | 0.18 | 2.46 | 7.80 |
| 7.50 | Core | --- | 2.76 | --- | --- | --- | --- | --- | --- | --- | --- | --- | Rock | | --- | --- | 24.33 | 24.33 | IV | 0.19 | 2.60 | 11.25 |

# ANNEXURE 3: SAFE BEARING CAPACITY OF SOIL

## Calculation of Net Safe Bearing Capacity based on shear parameters as per IS 6403(I)-1981

| | | | | | | |
|---|---|---|---|---|---|---|
| Shape of Footing | | Square | Angle of Internal friction | $\phi$ = 32.00 ° | | |
| Depth of Footing | D | = 1.50 m | ( from **Note)** | $\phi'$ = 22.73 ° | | |
| Width of Footing | B | = 3.00 m | Correction Factor for Water Table | W' = 0.50 | | |
| Length of Footing | L | = 3.00 m | Bulk Unit Weight of foundation soil | $\gamma$ = 1.85 | gm/cm$^3$ | |
| Cohesion | C | = 0.00 kg/cm$^2$ | | = 0.0019 | kg/cm$^3$ | |
| Factor of Safety | F | = 2.50 | | | | |
| void ratio | e | = 0.83 | | | | |

**Note:** The type of failure used for bearing capacity analysis depends upon the value of void ratio (see IS 6403 : 1981, Page No. 9, Table No. 3).
from above void ration value, > 0.75 then Local shear failure.

The ultimate net bearing capacity in case of general shear failure is given by (from IS 6403 : 1981, page No. 8)

$$\mathbf{q_d} = c\, N_c\, s_c\, d_c\, i_c + q\,(N_q\text{-}1)\, s_q\, d_q\, i_q + (1/2)\,\gamma\, B\, N_\gamma\, s_\gamma\, d_\gamma\, i_\gamma\, w']$$

The ultimate net bearing capacity in case of local shear failure is given by (from IS 6403 : 1981, page No. 8)

$$\mathbf{q'_d} = (2/3)\, c\, N'_c\, s_c\, d_c\, i_c + q\,(N'_q\text{-}1)\, s_q\, d_q\, i_q + (1/2)\,\gamma\, B\, N'_\gamma\, s_\gamma\, d_\gamma\, i_\gamma\, w'$$

For, Intermediate shear failure as per IS 6403:1981, page no. 9, Table no. 3, results are interpolated values of bearing capacity obtained from general and local shear failure criteria.

Bearing Capacity Parameters (IS 6403:1981, page no. 8, Table-1),

| For General Shear Failure | For Local Shear Failure |
|---|---|
| $N_c$ = 35.49 | $N_c'$ = 17.72 |
| Nq = 23.18 | Nq' = 8.42 |
| $N_\gamma$ = 30.21 | $N_\gamma'$ = 7.90 |

**Note:-** As per IS 6403:1981 Table-1, for Local Shear Failure taken $\phi'$ in place of $\phi$.
**$\phi' = \tan^{-1}(0.67 \tan\phi)$**

Shape Factor (IS 6403:1981, page no. 8, Table-2),

$S_c$ = 1.30
$S_q$ = 1.20
$S_\gamma$ = 0.80

Inclination Factors (IS 6403:1981, page no. 9),

$i_c$ = 1.00
$i_q$ = 1.00
$i_\gamma$ = 1.00

Depth Factor (IS 6403:1981, page no. 9),

$d_c$ = 1.18
$d_q$ = 1.09
$d_\gamma$ = 1.09

The ultimate net bearing capacity in case of general shear failure is given by (from IS 6403 : 1981, page No. 8),

$$q_d = c\,N_c\,s_c\,d_c\,i_c + q\,(N_q-1)\,s_q\,d_q\,i_q + (1/2)\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma\,w'$$

$= 11.71$ kg/cm$^2$

$= \mathbf{117.07}$ **T/m$^2$**

Safe Bearing Capacity = $q_d$ / FOS

$= \mathbf{46.83}$ **T/m$^2$**

The ultimate net bearing capacity in case of local shear failure is given by (from IS 6403 : 1981, page No. 8),

$$q'_d = (2/3)\,c\,N'_c\,s_c\,d_c\,i_c + q\,(N'_q-1)\,s_q\,d_q\,i_q + (1/2)\,\gamma\,B\,N'_\gamma\,s_\gamma\,d_\gamma\,i_\gamma\,w'$$

$= 3.65$ kg/cm$^2$

$= \mathbf{36.51}$ **T/m$^2$**

Safe Bearing Capacity = $q'_d$ / FOS

$= \mathbf{14.60}$ **T/m$^2$**

For, Intermediate shear failure from above values,

Safe Bearing Capacity = **---** **T/m$^2$**

Thus,

Recommended Soil safe bearing capacity based on Shear criteria is **14.60** **T/m$^2$.**

**NOTE: The type of failure used for bearing capacity analysis depends upon the value of void ratio (see IS 6403 : 1981, Page No. 9, Table No. 3).**

## Calculation of Bearing Pressure based on Settlement Criteria from SPT result

Settlement of Footing of width B under unit intensity of pressure resting on dry cohesionless deposit with known standard penetration resistance value N, read from Fig-9 of IS 8009(I) 1976. Water table corrections applied by multipying correction factor.

| | | |
|---|---|---|
| Depth | D | = 1.50 m |
| SPT (N) value | N | = 30 |
| Width of Footing | B | = 3.00 m |
| Correction Factor for Water Table | W' | = 0.50 |

| | | |
|---|---|---|
| Settlement for unit pressure ( 1 kg/cm$^2$ ) | = 8.70 mm | from fig-9 of IS 8009(part-1) |
| Depth Factor | = 0.852 | IS 8009 (Part:- 1) - 1976 |
| rigidity factor | = 1.0 | Clause-9.5, Page-21 |

Now,

for 25 mm permissible settlement Safe Bearing Pressure =((25 X 1) / 8.7 X 0.852 X 1) X 0.5

= 1.686 kg/cm$^2$

**= 16.86 T/m$^2$**

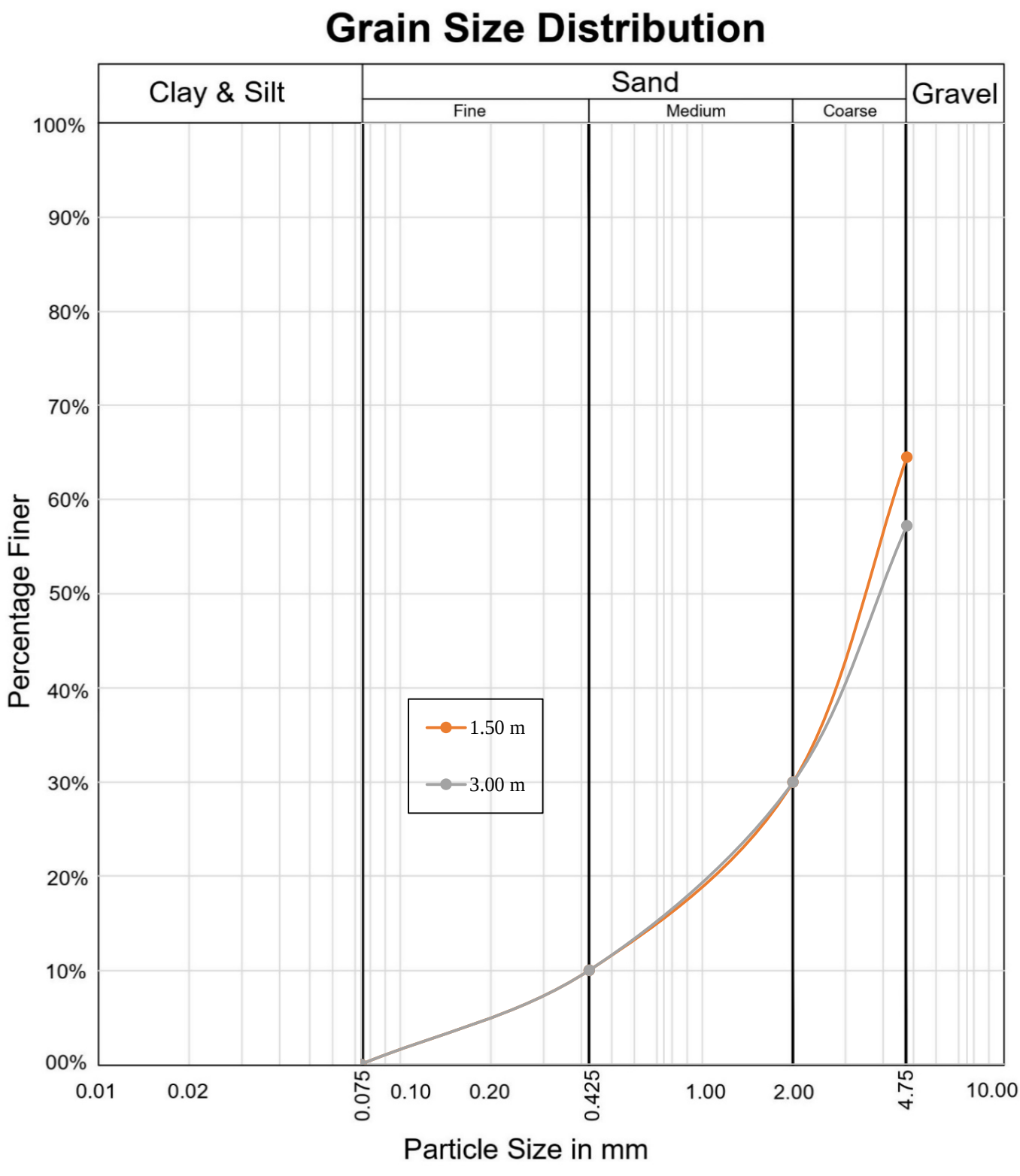
Thus,

Soil Safe Bearing Capacity based on 25mm Settlement is **16.86 T/m$^2$.**

**Then, Recommended Bearing Capacity (minimum Value) is 14.60 T/m$^2$.**

# ANNEXURE 4: PARTICLE SIZE DISTRIBUTION GRAPH

| | |
|---|---|
| Name of Work: Consultancy Services for Carrying out Engineering Studies for Remodelling of Inplant Ukai Thermal Power station, connectivity from Ukai – Songadh Station & GSECL Inplant yard at Ukai Songadh station (11.5 km Approx) in Mumbai Division of the Western Railway. | |
| Gujarat Rail Infrastructure Development Corporation Limited Borehole No.-Bore-1 | Borehole-1<br>Termination Depth: 7.5 m |



# SITE PHOTOGRAPHS

## Borehole No: Bore-1


Bore -1