Corrigendum -2

Dated 26.11.2024

Miscellaneous Civil infrastructure works at 600 MW Jhansi Solar Park (Tender no. TUSCO/SOLAR/JSPP/CIVILINFRA/06; Tender Id. 2024_THDC_835266_1)	
Bidders Query	**Clarification**
Design and specification for Porta cabin, weather monitoring station, watch tower and toilet is not provided.	Design of these facilities (more or less standard), to be submitted by contractor and approved by TUSCO Ltd.
For work of Greenbelt development details regarding plant and it's number is not provided	Greenbelt development is to be done as per standard specification of CPWD.
In drawing of fencing for wire mesh it says double twisted wire mesh but in BoQ description of item it specify only single twisted wire mesh	The wire mesh specification may be read as "4MM GI SINGLE TWISTED WIRE OF SIZE 100X100".Other related specifications remain same.
As per point no 2.0 of bidding documents - In PQ Criteria 1 or 2 or 3 completed works are required within 7 years but the updation factor which has to be multiplied with the works completed in previous 7 financial years is not mentioned. For example - If we have completed the work in financial year 2021-2022 of Rs. 100, then to evaluate the completed works as on date what updation factor should be multiplied to bring the value of completed works as on date today. Moreover, all the departments like MORTH etc clearly give the table of multiplication factors for the evaluation of completed works as on date.	Provision of bid document shall prevail.
As per point no. 2.0 of bidding documents - In PQ Criteria Definition of Completed works should be more clarified because sometimes works get physically completed and handing over of completed works takes so much time. Moreover, preparation of the final bill and payment is time consuming. So, it should be clarified whether the work is 90 % completed then the work is deemed to be completed. These conditions are clarified in all the Bidding Documents of other departments like MoRTH etc. Hence, clarification in this point is required.	Provision of bid document shall prevail.
Requirement of Soil Testing report	Geotechnical report is attached as Annexure -1.
Details of Cross Section of Internal Road including Specification of Top Layer Bitumen	Design to be provided by the contractor in line with CPWD specifications for approval of TUSCO Ltd.

AGM (C&MM)
TUSCO Limited, 4th Floor UPNEDA Bhawan
Vibhuti Khand Gomti Nagar, Lucknow,
Uttar Pradesh (India),
E-mail: tuscosolar@thdc.co.in

Annexure -1

TUSCO LIMITED

(A JOINT VENTURE OF THDCIL & UPNEDA GOVT. of U.P.)

Final Report

on

Geotechnical Investigation for Proposed 600 MW Solar Power Plant (3000 Acres of Land) at Talbehat Tehsil, Villages-Pawa, Sarkhadi, Piprai, Geura Gundera, Verma Bihar, Shahpur, Kalesra Kala, Jharar, Lalitpur Uttar Pradesh

Project No. 220145N-B

Date: 19th March, 2021

Report Prepared by:
CENGRS GEOTECHNICA PVT. LTD.
SOIL AND FOUNDATION EXPERTS

Cengrs Geotechnica Pvt. Ltd.

Corporate Office: A- 100, Sec- 63, Noida- 201309 (U.P.) India. I CIN- U74899DL1990PTC040605 Phone : +91 120-4206771, E: contact@cengrs.com, W : www.cengrs.com Registered Office : Cengrs House, B- 3/87, Safdarjung Enclave, New Delhi - 110029

19th March 2021

Project No. 220145-B

Tusco Limited
4th Floor, UPNEDA Bhawan,
Vibhuti Khand, Gomti Nagar,
Lucknow- 226 010
Uttar Pradesh

Sub: **Geotechnical Investigation for Proposed 600 MW Solar Power Plant (3000 Acres of Land) at Talbehat Tehsil, Villages-Pawa, Sarkhadi, Piprai, Geura Gundera, Verma Bihar, Shahpur, Kalesra Kala, Jharar, Lalitpur Uttar Pradesh**

Dear Sir,

We have carried out the captioned study in accordance with your Letter of Award No.: TUSCO / SOLAR / LUCKNOW / 94 dated 24TH December, 2020. We thank you for your business, and hope that you are satisfied with our services rendered.

This Final Interpretive Report presents our findings based on the geotechnical investigations conducted by us at the captioned project site. This report presents the field and laboratory test data, along with our geotechnical engineering recommendations, which shall help you in deciding the optimum foundation arrangement for use on site.

We have prepared this report based on our findings on site, as well as our vast experience gained in over 6000 projects completed over the past 30 years. We welcome you to involve us during the detailed foundation design, construction and testing phases, so that we may use our knowledge to serve you better.

We are pleased to have been of service to you on this project and will be glad to consult further with you and your design team.

Yours faithfully,
CENGRS GEOTECHNICA PVT. LTD.

Sanjay Gupta
Managing Director

Ravi Sundaram
Director

EXECUTIVE SUMMARY

Topic	Summary of Results
Project	Geotechnical Investigation for Proposed 600 MW Solar Power Plant Project
Location	Talbehat Tehsil, Villages-Pawa, Sarkhadi, Piprai, Geura Gundera, Verma Bihar, Shahpur, Kalesra Kala, Jharar, Lalitpur Uttar Pradesh
Scope of Work	Drilling of twenty (20) boreholes to 10 m depth in soil or 3 m into rock; Conducting ten (10) electrical resistivity tests; and Conducting twenty (20) field permeabilty tests
Site Straitigraphy	Based on soil strata and depth of rock encountered in borehole locations at site, we have divided the site into two areas. **Area 1**: In this area soil is met from GL to 1.5 m depth underlain by Granitic Gneiss rock to the maximum explored depth of 4.0 m. SPT-N values to 1.5 m are in range of 24-85. Refusal was met at soil rock interface. In rock strata . core recovery of 34-99% with RQD of 0-78 % was met at site.. **Area 2**: In this area the soils at project site primarily consist of silty sand to 7.0 m depth underlain by Granitic Gneiss to the maximum explored depth of 10.0 m. SPT-N values range from 24-56 to about 2.9~7.0 m depth. Refusal was met at soil rock interface. In rock strata. core recovery of 12-80% with RQD of 0-34 % was met at site..
Groundwater	Groundwater was not met to the explored depth (February, 2021)
Liquefaction Susceptibility Assessment	The soils at this site are not likely to liquefy in the event of an earthquake
Foundation Recommendations	Individual / isolated foundation may be used to support the various plant facilities such as Control Rooms, guard houses, etc. Our suggested values of net allowable bearing pressures for foundations bearing at 1.5-2.0 m depth are presented in Section 5.7. Bored cast-in-situ piles may be provided for the proposed PV solar panels in view of the high uplift forces generally anticipated for such structures. Our recommended safe pile capacities for 300 & 350 mm diameter short bored pile and pile socketed into rock are presented in Section 5.6.
Foundation Construction Considerations	Please refer to Section 6.0 for general recommendations on temporary excavations, foundation level preparation, backfilling and chemical attack on foundation concrete.

1.0 INTRODUCTION

1.1 Project Description

M/s. Tusco Limited has been awarded a contract to construct 600 MW Solar Power Plant in Distt. Lalitpur, Uttar Pradesh. The project site covers a large tract of land (3000 acres) that falls in Villages Pawa, Sarkhadi, Piprai, Geura Gundera, Verma Bihar, Shahpur, Kalesra Kala and Jharar.

M/s. Tusco Limited has appointed M/s. Cengrs Geotechnica Pvt. Ltd. (CENGRS) for carrying out the geotechnical investigation for the project site. A sketch indicating the test location of our field investigation is illustrated on Plate 1.

1.2 Scope of Work

The overall purposes of this study are to investigate the stratigraphy at the site and to develop geotechnical recommendations for foundation design of planned facilities. To accomplish these purposes, the study was conducted in the following phases:

(i) drilling twenty (20) exploratory boreholes to about 10 m depth in soil or 3 m into rock, in order to evaluate the stratigraphy and to collect samples for laboratory testing;

(ii) conducting ten (10) electrical resistivity tests (ERT's) to provide data for design of the electrical grounding system;

(iii) conducting twenty (20) field permeability test (FPT) at specified boreholes and at specified depth to assess the in-situ coefficient of permeability of the strata;

(iv) testing selected soil samples in the laboratory to determine pertinent index and engineering properties; and

(v) analyzing all field and laboratory data to develop geotechnical recommendations for foundations of proposed structure.

1.3 Scope of Work covered in this Report

The following table presents details of the tests covered in this report:

- **Exploratory Boreholes**

S. No.	Village	Borehole No.	UTM Coordinates*, m		Termination Depth*, m	Field Permeability Tests
			Easting	Northing		
1	Jharar	BH-1	244324	2779422	4.50	FPT at 3.0 m depth
2	Kadesra Kalan	BH-2	243592	2779056	10.00	FPT at 5.0 m depth
3	Jharar	BH-3	244906	2780573	4.50	FPT at 3.0 m depth
4	Shahpur	BH-4	247018	2780717	7.50	FPT at 5.0 m depth
5	Sarkhandi	BH-5	247689	2779528	4.50	FPT at 4.0 m depth
6		BH-6	247396	2778880	10.00	FPT at 5.0 m depth
7		BH-7	247611	2778657	3.00	FPT at 3.0 m depth
8		BH-8	247001	2778539	6.00	FPT at 6.0 m depth
9	Pawa	BH-9	245981	2778556	3.00	FPT at 3.0 m depth
10		BH-10	247500	2777600	8.00	FPT at 3.0 m depth
11		BH-11	246111	2776961	3.00	FPT at 3.0 m depth
12		BH-12	248569	2776870	4.00	FPT at 4.0 m depth
13		BH-13	248572	2776174	6.00	FPT at 6.0 m depth
14		BH-14	247476	2776020	4.50	FPT at 4.0 m depth
15		BH-15	246554	2776155	3.00	FPT at 3.0 m depth

S. No.	Village	Borehole No.	UTM Coordinates*, m		Termination Depth*, m	Field Permeability Tests
			Easting	Northing		
16	Piprai	BH-16	246554	2777028	4.00	FPT at 3.0 m depth
17	Gloragundera	BH-17	248840	2780372	4.50	FPT at 3.0 m depth
18	Gloragundera	BH-18	248859	2781402	4.50	FPT at 3.0 m depth
19		BH-19	248365	2782034	4.00	FPT at 3.0 m depth
20	Pawa	BH-20	248600	2776303	6.00	FPT at 3.0 m depth

* UTM coordinates taken on site using hand-held Garmin® GPS device

- **Electrical Resistivity Tests**

S. No.	Location	ERT No.	UTM Coordinates*, (Zone 43 Q), m		Electrode Spacing, m
			Easting	Northing	
1	Pawa	ERT-1	246015	2778561	10.0
2		ERT-2	248877	2776313	10.0
3	Piprai	ERT-3	247321	2774652	10.0
4	Sarkhandi	ERT-4	247365	2778883	10.0
5		ERT-5	247742	2778676	10.0
6	Pawa	ERT-6	247464	2777521	10.0
7	Gloragundera	ERT-7	248851	2780367	10.0
8	Shahpur	ERT-8	247019	2780716	10.0
9	Kadesra Kalan	ERT-9	243593	2779066	10.0
10	Jharar	ERT-10	247914	2781763	10.0

- A layout plan indicating the location of our field investigations is indicated on Plate 1.
- The borehole locations were marked on the field by the client representative and recorded by us using a hand-held Global Positioning System (GPS). A satellite image indicating the borehole locations (as recorded by GPS) is presented on Plate 2 .

2.0 FIELD INVESTIGATION

2.1 Rotary Drilling

Drilling was performed using a rotary drill rig. The drill chuck has four jaws to accommodate NW size drill rod.

Drilling and sampling of the rock was performed using an NX size double tube core barrel. A 32–carat diamond impregnated bit was used to drill through rock strata. The bit was attached to the end of a core barrel, which is connected to the machine by a string of NW drill rods and rotated by the drilling machine.

Water was circulated through the drill rods to the bottom of the hole. The water serves the purpose of lubrication, cooling and protection of the diamond drill bit in addition to flushing the cuttings out of the hole. A reciprocating pump was used to circulate the water. While drilling through soft rock that is likely to collapse, NX size casing was installed. The casing has a TC shoe bit to assist it to advance.

The percent core recovery and Rock Quality Designation (RQD) was measured for each core run. The percent core recovery is defined as the percent ratio of the cumulative length of core sample recovered to the total length of the core run. The Rock Quality Designation (RQD) is defined as the ratio of the cumulative length of core pieces 10 cm or longer to the total length of the core run, expressed as percentage. The Rock mass Rating (RMR), an engineering parameter that assists in assessing the rock quality and behavior is also presented on the individual rock profiles.

Details of rock samples collected and their respective core recovery, RQD and RMR values are presented on the borelogs at various depths. The color of return water and the extent of water loss while drilling the borehole recorded on the boring logs may be used for an assessment of the nature of rock, water-tightness of joints and possible presence of interconnected channels / cavities.

2.2 Standard Penetration Tests

Standard Penetration Test (SPT) was conducted in the boreholes at 1.5 m depth interval. The test was conducted by connecting a split spoon sampler to 'A' rods and driving it by 45 cm using an automatic trip hammer of weight 63.5 kg hammer falling freely from a height of 75 cm. The tests were conducted in accordance with IS: 2131-1981 RA 2002.

The SPT 'N'-values are described as follows:

1. The number of blows for each 15 cm of penetration of the split spoon sampler was recorded.

2. The blows required to penetrate the initial 15 cm of the split spoon for seating the sampler were ignored due to the possible presence of loose materials or cuttings from the drilling operation.

3. The cumulative number of blows required to penetrate the balance 30 cm of the 45 cm split spoon sampler was termed the SPT value or the 'N' value. For example, a SPT value reported as "20" means that 20 blows were imparted to penetrate the split spoon sampler by the last 30 cm.

4. Where the number of blows required to penetrate the balance 30 cm of the split spoon sampler exceeds 100, the number of blows was presented along with the corresponding penetration. For example, an SPT value reported as "101/5 cm" means that 101 blows were imparted to penetrate the split spoon sampler by 5 cm after the first 15 cm initial (seating) penetration.

5. Where refusal (N>100) to further penetration of the split spoon sampler was encountered in the first 15 cm of seating penetration itself, SPT test could not be completed and "*Ref*" was indicated in the bore logs, along with the penetration achieved. For example, an SPT value reported as "Ref/5 cm" means that more than 100 blows were imparted to penetrate the split spoon sampler by a total of 5 cm only and the 15 cm seating penetration could not be achieved.

Disturbed samples were collected from the split spoon after conducting SPT. The samples were preserved in transparent polythene bags. Undisturbed soil samples were collected by attaching 100 mm diameter thin walled 'Shelby' tubes and driving the sampler by light-hammering using a 63.5 kg hammer in accordance with IS: 2132-1986 RA 2002. The tubes were sealed with wax at both ends. All samples were transported to our NABL-accredited laboratory at Noida for further examination and testing.

2.2 Field Permeability Test

Field permeability tests in were done by falling head method. Total 20 number of tests were carried out at site in accordance with IS: 5529 (Part-I) -1985 RA 1995. The tests were conducted in selected boreholes at the specified depths. The field coefficient of permeability was computed using the following equation:

$$k = \frac{d^2}{8L} \ln\left(\frac{L}{r}\right) \frac{\ln(h_1/h_2)}{(t_2 - t_1)}$$

where

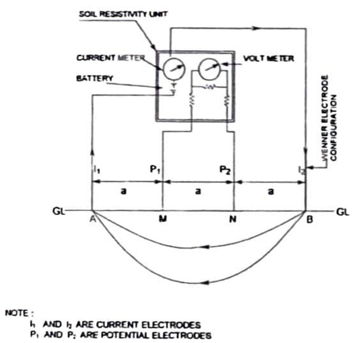
k	=	mean coefficient of permeability
d	=	diameter of intake pipe (stand pipe)
r	=	radius of hole
L	=	length of test section
h_1	=	head at time t_1
h_2	=	head at time t_2

2.3 Electrical Resistivity Tests (ERT)

Electrical resistivity of the soils at the site was determined at specified locations. The electrical resistivity test is used for shallow subsurface exploration by means of electrical measures made at the ground surface. Resistivity measurements are made by driving four electrodes about 10 to 15 cm in to the ground at pre-selected electrode spacing. We used the Wenner electrode configuration for this study.

The four electrodes were spaced at equal distance along a line. The test procedure is in accordance with IS: 3043:1987 RA 2006.

The schematic arrangement of electrodes is shown below:



NOTE:

I_1 and I_2 are current electrodes
P_1 and P_2 are potential electrodes
a: Electrode spacing

Measurements are made by causing a current, 'I', to pass through the earth and distribute within a relatively large hemispherical earth mass. The portion of the current that flows along the surface produces a voltage drop, 'V'. The resistance 'R', ratio of voltage drop 'V' to current 'I' is directly measured by Digital Earth Resistance Tester. The resistivity is determined from the following equation:

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

where:

ρ = apparent resistivity, ohm-m
a = spacing between the electrodes, meter
R = resistance, ohms

Results are presented as semi-logarithmic plot of apparent resistivity versus electrode spacing, as well as in the form of polar curves, as specified by IS: 3043:1987 RA 2006.

2.4 Groundwater

Groundwater level was measured in the boreholes after drilling and sampling was completed. The measured water levels are recorded on the individual borelog.

3.0 LABORATORY TESTS

The laboratory testing has been carried out in our NABL accredited laboratory. The quality procedure in our laboratory conforms to ISO/IEC-17025-2017.

Laboratory tests were conducted on selected soil and rock samples to determine their physical and engineering properties. The testing procedures were in accordance with current applicable IS specifications.

The following tests were conducted on selected soil and rock samples recovered from the boreholes:

For Soil:

Laboratory Test		IS Code Referred
Specific Gravity		IS: 2720 (Part-3)-1980, RA-2007
Grain size analysis		IS: 2720 (Part-4)-1985, RA-2010
Liquid Limit and Plastic Limit		IS: 2720 (Part-5)-1985, RA-2010
Modified Proctor Test		IS: 2720 (Part-8) -1983, RA-2006
Laboratory CBR		IS: 2720 (Part-16)-1987, RA-2002
Free Swell Index Test		IS: 2720 (Part-40)-1977, RA-2007
Chemical Analysis of soil*	pH value	IS: 2720 (Part 26)-1987, RA-2007
	Sulphates	IS: 2720 (Part-27)-1977, RA-2010
	Chlorides	IS: 3025 (Part-32)-1988, RA-2009

* Outside NABL Scope

For Rock:

Laboratory Test	IS Code Referred
Porosity and Void ratio	By calculations
Density & Water absorption	IS :13030-1991, RA-2001
Specific Gravity	IS : 2720 (Part-3)-1980, RA-2007
Unconfined compressive strength	IS : 9143-1979, RA-2006

*Outside NABL Scope

Engineering terms used for describing soils are explained on Plate 4. Engineering terms used for describing rock are explained on Plate 5. Rock mass rating system using Bieniawaski is presented on Plate 6. A note on our NABL accreditation together with uncertainty estimates in laboratory measurements is presented on Plate 7.

4.0 GENERAL SITE CONDITIONS

4.1 Site Description

The project site is located at Distt. Lalitpur, Uttar Pradesh and is lies at latitude 25° 6'30.37" N and longitude 78°29'45.67" E. The site is located to about 2 km north-east from NH-26. The site is an open and agriculture land.

4.2 Site Stratigraphy

Based on soil strata and depth of rock encountered in borehole locations at site, we have divided the site into two areas.

Area 1 (BH-1, 3, 5, 7, 9, 11, 12, 14, 15, 16, 17, 18 and 19):

In this area Silty sand is met to 0.0~1.5 m depth underlain by Granitic Gneiss rock to the maximum explored depth of 4.0 m.

SPT-N values to 1.5 m are in range of 24-85. Refusal was met at soil rock interface. In rock strata . core recovery of 34-99% with RQD of 0-78 % was met at site.

The detailed site stratigraphy of **AREA-1** with SPT-N value , core recovery and RQD are tabulated below:

BH. No.	Depth, m		Soil type	Field SPT N value	CR,%	RQD	RMR
	From	To					
BH-1	0.0	1.5	Clayey silt, medium plastic	27	-	-	-
	1.5	4.5	Moderately weak grey GRANITIC GNEISS, severely weathered	Ref	39-45	0-7	20-25
BH-3	0.0	1.5	Silty fine sand	85	-	-	-
	1.5	4.5	Moderately strong grey GRANITIC GNEISS, moderately weathered	Ref	36-49	19-39	25-30
BH-5	0.0	1.5	Silty fine sand	28	-	-	-
	1.5	4.5	Moderately weak to moderately strong grey GRANITIC GNEISS, severely to moderately weathered	Ref	34-40	0-31	20-30
BH-7	0.0	0.5	Very weak grey GRANITIC GNEISS, very severely weathered	100	-	-	-
	0.5	3.0	Moderately strong grey GRANITIC GNEISS, moderately weathered	-	61-67	61-67	40
BH-9	0.0	3.0	Moderately strong grey GRANITIC GNEISS, moderately weathered	-	56-66	38-53	40
BH-11	0.0	3.0	Moderately strong grey GRANITIC GNEISS, moderately weathered	-	50-69	12-46	25-35
BH-12	0.0	1.0	Silty fine sand	17	-	-	-
	1.0	4.0	Moderately weak to moderately strong grey GRANITIC GNEISS, severely to moderately weathered	-	20-46	0-22	18-25
BH-14	0.0	0.5	Fine sand	Refusal	-	-	-
	0.5	4.5	Moderately weak to moderately strong grey GRANITIC GNEISS, severely to moderately weathered	-	25-85	0-74	18-45
BH-15	0.0	30.0	Moderately strong grey GRANITIC GNEISS, moderately weathered	-	32-61	21-29	28-35
BH-16	0.0	1.0	Silty fine sand	57	-	-	-
	1.0	4.0	Moderately strong grey GRANITIC GNEISS, moderately weathered	-	25-99	21-78	22-48
BH-17	0.0	1.5	Silty fine sand	24	-	-	-
	1.5	4.5	Moderately weak to moderately strong grey GRANITIC GNEISS, severely to moderately weathered	Ref	33-74	0-57	20-45
BH-18	0.0	1.5	Sandy silt, low plastic	13	-	-	-
	1.5	4.5	Moderately weak to moderately strong grey GRANITIC GNEISS, severely to moderately weathered	Ref	39-46	0-18	20-25
BH-19	0.0	1.0	Silty fine sand	64	-	-	-
	1.0	4.0	Weak to moderately strong grey GRANITIC GNEISS, severely to moderately weathered	Ref	14-45	0-31	17-30

Area 2 (BH-2, 4, 6, 8, 10, 13, 20):

In this area the soils at project site primarily consist of Silty sand to 7.0 m depth underlain by Granitic Gneiss to the maximum explored depth of 10.0 m.

SPT-N values range from 24-56 to about 2.9~7.0 m depth. Refusal was met at soil rock interface. In rock strata. core recovery of 12-80% with RQD of 0-34 % was met at site..

The detailed site stratigraphy of **AREA-2** with SPT-N value , core recovery and RQD are tabulated below :

BH. No.	Depth, m		Soil type	Field SPT N value	CR,%	RQD	RMR
	From	To					
BH-2	0.0	7.0	Silty fine sand	35-56	-	-	-
	7.0	10.0	Moderately strong grey GRANITIC GNEISS, moderately weathered	Ref	22-80	22-34	25-40
BH-4	0.0	3.0	Silty fine sand	34-Ref	-	-	-
	3.0	7.5	Weak to moderately strong grey GRANITIC GNEISS, severely weathered	Ref	8-35	0	17-20
BH-6	0.0	4.5	Silty fine sand	31-62	-	-	-
	4.5	10.0	Moderately weak grey GRANITIC GNEISS, severely weathered	Ref	12-25	0-9	17-20
BH-8	0.0	2.9	Silty fine sand	24-78	-	-	-
	2.9	6.0	Moderately weak grey GRANITIC GNEISS, severely weathered	Ref	14-34	0	17-20
BH-10	0.0	4.5	Silty fine sand	13-25	-	-	-
	4.5	8.0	Weak to moderately strong grey GRANITIC GNEISS, severely to moderately weathered	Ref	9-29	0-23	17-25
BH-13	0.0	3.0	Fine sand	Ref	-	-	-
	3.0	6.0	Moderately strong grey GRANITIC GNEISS, moderately weathered	Ref	31-49	21-23	25
BH-20	0.0	3.0	Silty fine sand	Ref	-	-	-
	3.0	6.0	Moderately weak grey GRANITIC GNEISS, severely weathered	Ref	7-27	0	17-20

Detailed description of the soil encountered at the site is presented on Plates 8 to 47. A pictorial summary of the borehole profiles is presented in Plate 48 to 52. Contours of depth to rock are presented on Plate 3.

4.3 Groundwater

Based on our measurements in the completed boreholes at site, groundwater was not met to the final explored depth of 10.45 m at site during the period of our field investigation (February, 2021). Fluctuations may occur in measured water levels due to seasonal variation in rainfall and surface evaporation rates.

4.4 Field Permeability Test Results

Field permeability tests have been conducted in the boreholes at specified depths by falling head method. The test results are summarized below:

BH No.	Test Designation	Test Depth, m	Strata Classification	Coefficient of Permeability (k), cm/s	Presentation of Results
BH-1	FPT-1	3.00	Weathered rock	6.8×10^{-7}	Plate 73 to 112
BH-2	FPT-2	5.00	Fine Sand	1.0×10^{-4}	
BH-3	FPT-3	3.00	Weathered rock	6.1×10^{-7}	

BH No.	Test Designation	Test Depth, m	Strata Classification	Coefficient of Permeability (k), cm/s	Presentation of Results
BH-4	FPT-4	5.00	Weathered rock	4.1×10^{-7}	Plate 73 to 112
BH-5	FPT-5	4.00	Weathered rock	6.9×10^{-7}	
BH-6	FPT-6	5.00	Weathered rock	1.2×10^{-6}	
BH-7	FPT-7	3.00	Weathered rock	6.8×10^{-7}	
BH-8	FPT-8	6.00	Weathered rock	$6.8 \times 10-^{7}$	
BH-9	FPT-9	3.00	Weathered rock	6.8×10^{-7}	
BH-10	FPT-10	3.00	Silty sand	1.2×10^{-4}	
BH-11	FPT-11	3.00	Weathered rock	2.1×10^{-6}	
BH-12	FPT-12	4.00	Weathered rock	1.2×10^{-6}	
BH-13	FPT-13	6.00	Weathered rock	6.1×10^{-7}	
BH-14	FPT-14	4.00	Weathered rock	9.6×10^{-7}	
BH-15	FPT-15	3.00	Weathered rock	$7.5 \times 10-^{7}$	
BH-16	FPT-16	3.00	Weathered rock	1.4×10^{-6}	
BH-17	FPT-17	3.00	Weathered rock	2.0×10^{-7}	
BH-18	FPT-18	3.00	Weathered rock	6.2×10^{-7}	
BH-19	FPT-19	3.00	Weathered rock	8.2×10^{-7}	
BH-20	FPT-20	3.00	Silty sand	1.0×10^{-4}	

4.5 Electrical Resistivity Tests

Ten (10) electrical resistivity tests were conducted at site as per IS: 3043:1987 [RA 2007]. The test was conducted using the Wenner configuration. The apparent resistivity value obtained was analyzed to generate the polar curve.

Mean resistivity values of the ten (10) electrical resistivity tests (ERT) results up to 10 m electrode spacing are summarized in the table below:

Sr. No.	Location	Test Designation	Mean Resistivity, ohm-m	Presentation of Results
1	Pawa	ERT-1	13.0	Plate 53 to 72
2	Pawa	ERT-2	13.0	
3	Piprai	ERT-3	13.0	
4	Sarkhandi	ERT-4	5.0	
5	Sarkhandi	ERT-5	5.0	
6	Pawa	ERT-6	11.0	
7	Gloragundera	ERT-7	5.0	
8	Shahpur	ERT-8	5.0	
9	Kadesra Kalan	ERT-9	5.0	
10	Jharar	ERT-10	5.0	

The above values may be used for design of the electrical grounding system. The data may also be used to assess the corrosion potential for buried utility lines as per the guideline given in IS 3043-1987-RA 2006.

4.6 Laboratory Proctor and CBR Test Results

Modified Proctor Tests were conducted as per IS: 2720 (Part-8) -1983, RA-2006, on the bulk samples collected from the location of the respective Field CBR tests. The Modified Proctor test results are tabulated below:

Sample Location	Maximum Dry Density (MDD), g/cc	Optimum Moisture Content (OMC), %	Presentation of Results
LCBR-1	2.03	8.8	Plate 130 to 139
LCBR-2	2.07	9.4	
LCBR-3	1.96	10.5	
LCBR-4	2.03	9.9	
LCBR-5	2.01	8.9	
LCBR-6	2.03	9.6	
LCBR-7	2.03	9.2	
LCBR-8	1.97	9.7	
LCBR-9	1.94	9.2	
LCBR-10	2.02	8.3	

Laboratory CBR tests (soaked & unsoaked) were conducted as per IS: 2720 (Part-16) -1987 RA-2007 on samples collected from the site. The samples were remoulded to 98 % maximum dry density (MDD) and 100 % optimum moisture content (OMC). For conducting the test under soaked condition, the samples were soaked for 96 hours under the influence of the surcharge load.

Test results on specimens compacted to the required density are illustrated on Plates 154 to 163. A summary of the CBR test results is also tabulated below for easy reference:

Sample Location	Compacted Dry Density, g/cc	Initial Moisture Content, %	Laboratory CBR Value, %		Presentation of Results
			Unsoaked	Soaked	
LCBR-1	1.98	9.0	38.9	18.8	Plate 140 to 149
LCBR-2	2.04	9.0	35.1	18.5	
LCBR-3	1.91	11.0	35.4	17.0	
LCBR-4	1.99	9.5	28.9	17.0	
LCBR-5	1.98	8.8	29.6	17.3	
LCBR-6	1.99	9.6	28.5	15.9	
LCBR-7	1.98	9.3	35.6	20.2	
LCBR-8	1.93	9.8	31.4	18.5	
LCBR-9	1.90	9.1	33.0	17.9	
LCBR-10	1.97	8.4	36.7	21.7	

We suggest that a soaked CBR value of 10 be used for the design of the internal roads.

4.7 Area Distribution of Soil Conditions

Based on soil strata and depth of rock encountered in borehole locations at site, we have divided the site into two areas. Area 1 covers most of the site where rock is met at less than 1.5 m depth. Area 2 covers of the site where the depth to rock exceeds 1.5 m. The following table presents the distribution of the boreholes in the two areas:

Area	Boreholes	Soil Classification	SPT-N Value Criteria
Area-1	BH-1, 3, 5, 7, 9, 11, 12, 14, 15, 16, 17, 18 and 19	0-0.5~1.5m: Silty sand intermixed with gravel > 1.5 m: Weathered rock	<1.5 m- N>24-85
Area-2	BH-2, 4, 6, 8, 10, 13, 20	0-2.9~7.0 m: Silty sand intermixed with gravel Below 2.9~7.0 m: Weathered rock	<7.0 m- N= 24-56

5.0 FOUNDATION ANALYSIS AND RECOMMENDATIONS

5.1 General

A suitable foundation for any structure should have an adequate factor of safety against exceeding the bearing capacity of the supporting soils. Also the vertical movements due to compression of the soils should be within tolerable limits for the structure. We consider that foundation designed in accordance with the recommendations given herein will satisfy these criteria.

5.2 Liquefaction Susceptibility Assessment

Liquefaction is defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore-water pressure and reduced effective stress [Marcuson, 1978[(1)]]. Increased pore pressure may be induced by the tendency of granular materials to compact when subjected to cyclic shear deformation, such as in the event of an earthquake.

As per IS: 1893 (Part 1) - 2016, liquefaction is likely in loose fine sand (SP) below water table. The following points are highlighted with regard to liquefaction susceptibility of the soils during earthquakes:

1. As discussed in Section 4.2, silty sand is encountered underlain by Granitic Gneiss at the site to the maximum explored depth of 10.0 m.

2. As discussed in Section 4.3, groundwater was not met to the final explored depth of 10.0 m at site during the period of our field investigations (February, 2021).

On review of site strata conditions, SPT values and groundwater condition, we are of the opinion that the soils at site are not likely to liquefy in the event of an earthquake.

According to Fig.1 of IS: 1893 (Part-1)-2016 showing seismic zones, the proposed site falls under Zone-II. The design for seismic forces should be done considering the project in Zone-II.

5.3 Foundation Type and Depth

Short piles may be used for the solar panels. We recommend that RCC bored cast in-situ piles be provided to support the structural loads. Piles bearing on soil as well as piles socketed into rock may be used depending on the loads coming on the piles. Our recommended safe capacities for 300 and 350 mm diameter bored piles are presented in Section 5.6.

The control room and other buildings may be supported on isolated foundations. Our recommended net bearing pressures are given in Section 5.7.

5.4 Concepts for Foundation Analysis

5.4.1 RCC Bored Cast In-situ Piles

The axial capacity for bored piles has been computed as per IS 2911 (Part-I/Section-2) - 2010 based on static analysis using c-ϕvalues as interpreted from the site stratigraphy, SPT values and laboratory test results.

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(1) Marcuson, W.F. (III) (1978), "***Definition of terms related to liquefaction***", J. Geotech Engg. Div. ASCE, 104(9), 1197-1200.

$$Q_{ult} = \left[\sum_{i=1}^{n} f_s A_s L_i\right] + q_u A_p$$

$$= \left[\sum_{i=1}^{n} (\alpha c_i + p_i k \tan \delta) A_s L_i\right] + \left[c_p N_c + q_p N_q + \tfrac{1}{2}\gamma D N_\gamma\right] A_p$$

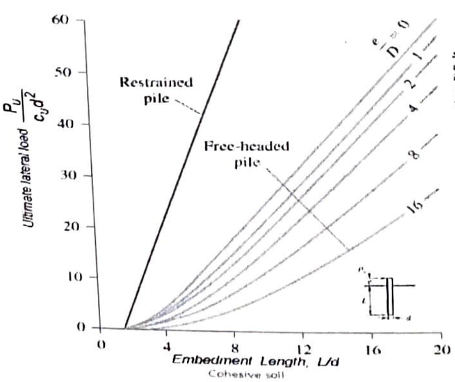
where:

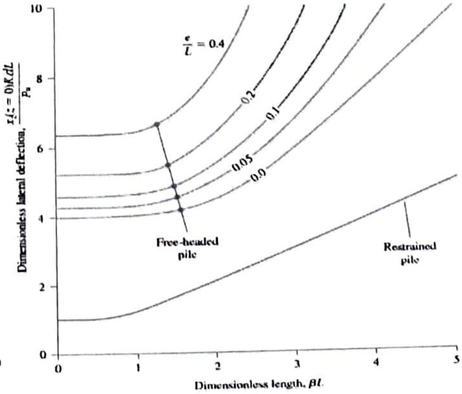
Q_{ult}	=	ultimate pile capacity
f_s	=	unit skin friction
α	=	adhesion factor
c_i	=	cohesion intercept in i^{th} layer
p_i	=	overburden pressure at centre of i^{th} layer
k	=	coefficient of lateral earth pressure
δ_i	=	angle of friction between soil and pile (taken as equal to ϕ_i) for the i^{th} layer
A_s	=	surface area of pile per m length
L_i	=	length of pile section in i^{th} layer
c_p	=	cohesion intercept in bearing strata
q_u	=	unit end bearing
q_p	=	overburden pressure in bearing strata
N_c, N_q	=	bearing capacity factors, which are a function of ϕ in the bearing strata
A_p	=	pile cross sectional area

The overburden pressure is assumed to become constant below depth of 15 times pile diameters. A factor of safety of 2.5 has been used on the ultimate pile capacity.

The uplift / pullout resistance has been computed from the static formula by ignoring the end bearing component and adding the effective weight of the pile to the skin friction component. A factor of safety of 3.0 has been applied to the ultimate pile pullout resistance to obtain the safe pullout capacity.

The lateral load carrying capacity of the short piles has been computed based on the Brom's Method[2]. The simplified solution is based on the assumption of shear failure in soil. Broms's solution for the ultimate/safe load resistance (P_u) and corresponding deflections for *short piles in cohesive soil* is computed as per the figures shown below:





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(2) Tomlinson, M.J. (1994), "***Pile Design & Construction Practice***" E&FN Spon, London, 4th edition, pp 223-235.

Das, B.M. (2010), "***Principles of Foundation Engineering***" Cengage Learning, Stamford, 7th edition, pp 599-602 where

P_u	: ultimate lateral load
c_u	: Undrained cohesion
d	: diameter of the pile
x	: deflection at the pile top
K	: modulus of subgrade reaction (as per IS 2911: Part 1(Section 2)-2010, Table 4)
L	: length of the pile
βL	: dimensionless length, where $\beta = (Kd/4EI)^{1/4}$
E	: modulus of elasticity of the pile material
I	: moment of inertia of the pile

Safe lateral pile capacity, $P_{safe} = P_u/3$

The safe load is computed corresponding to a pile head deflection of 5 mm.

5.4.2 RCC Bored Cast In-situ piles Socketed into Rock

The safe capacity of piles socketed into rock has been calculated as per IS 14593-1998 as the sum of the pile tip resistance and the friction along the socket. The following equation has been used:

$$Q_{safe} = \frac{1}{F}\left[C_u N_c \frac{\pi d^2}{4} + \alpha C_s \pi d \ell_s \right]$$

where :

Q_{safe} =	Safe axial compressive capacity of pile
C_u =	Shear strength of rock below base of pile
N_c =	Bearing capacity factor, taken as 9
α =	A multiplying constant, taken as equal to 0.9 (recommended value)
C_s =	Average shear strength of rock adjacent to shaft in the socketed length
l_s =	length of socket
D =	diameter of pile
F =	Factor of safety, usually taken as 6

5.4.3 Concept of Analysis – Shallow Foundations

Bearing capacity analysis for open / raft foundations has been done in general accordance with IS: 6403-1981 RA 2002. The bearing capacity equation used is as follows:

$$q_{net\ safe} = \frac{1}{F}\left[cN_c\zeta_c d_c + q(N_q-1)\zeta_q d_q + 0.5 B\gamma N_\gamma \zeta_\gamma d_\gamma R_w\right]$$

where:

$q_{net\ safe}$	=	safe net bearing capacity of soil based on the shear failure criterion.
q	=	overburden pressure
R_w	=	water table correction factor
F	=	Factor of safety, taken as equal to 2.5
$\zeta_c, \zeta_q, \zeta_\gamma$	=	Shape factors. For Strip footings, $\zeta_c = \zeta_q = \zeta_\gamma = 1$ For Square footing, $\zeta_c = 1.3$, $\zeta_q = 1.2$, $\zeta_\gamma = 0.6$
d_c, d_q, d_γ	=	Depth factors

For $\phi \leq 10$, $d_c = 1 + 0.2 \tan(45+\phi/2)\ D/B$, $d_q = d_\gamma = 1$

For $\phi > 10$, $d_q = d_\gamma = 1 + 0.1 \tan(45+\phi/2)\ D/B$

Appropriate values have been substituted into the bearing capacity equation given above to compute the safe net bearing capacity. The values have been checked to determine the settlement of the foundation under the safe bearing pressure. The allowable bearing pressure has been taken as the lower of the two values computed from the bearing capacity shear failure criterion as well as that computed from the tolerable settlement criterion.

Settlement analysis has been performed based on the SPT values in accordance with Clause 9.1.4 of IS 8009 (Part 1)-1976 RA 2003 Fig.9. The values have been cross-checked with settlement computed by the classical theory as sum of the immediate settlement and consolidation settlement. Since water table is not met to the final explored depth hence consolidation settlement is expected to be negligible.

The elastic settlement has been computed using the following equation [Clause 9.2.3 of IS 8009 Part 1-1976 RA 2003][(3)]:

$$S_i = \frac{qB'(1-\mu^2)}{E} I d_f d_r$$

where:

S_i	=	*immediate (elastic) settlement*
B	=	*foundation width, B' = B/2*
μ	=	*Poisson's ratio*
q	=	*applied bearing pressure*
E	=	*modulus of elasticity*
d_f	=	*depth factor*
d_r	=	*rigidity factor*
I	=	*influence factor at corner of rectangular loaded area (B'x L')*

5.5 Design Soil Parameters

Reviewing the soil characteristics, the following soil parameters have been selected for foundation analysis:

Borehole	Depth below EGL, m		Soil Classification	c, T/m^2	ϕ, degrees	γ, T/m^3
	From	To				
Area -1	0.0	1.5	Silty sand with gravel	0	33	20.0
	1.5	4.5	Weathered rock	0	36	22.0
Area -2	0.0	3.0	Silty sand with gravel	0	33	20.0
	3.0	10.0	Weathered rock	0	36	22.0

* Disintegrated weathered rock considered as Coarse sand for analysis.

where:

γ	=	bulk density
c	=	cohesion intercept
ϕ	=	angle of internal friction

5.6 Recommended Safe Pile Capacities

We recommend the following values of safe pile capacities for 300 mm & 350 mm RCC bored cast-in-situ piles with cut-off level (COL) at ground level.

(3) Bowles, J.E. (1996), "***Foundation Analysis and Design***", International Edition, pp. 303-317.

Area	Pile Diameter (D), mm	Pile length below cut-off level, m	Length of rock socket, m	Safe Compressive Pile Capacity, Tonnes	Safe Pullout capacity, Tonnes	Lateral Pile capacities*, Tonnes
Area-1	300	2.0	0.5	19.0	1.24	1.26
		2.2	0.7	22.0	1.82	1.36
		2.4	0.9	24.0	2.45	1.45
	350	2.0	0.5	26.0	1.45	1.28
		2.2	0.7	28.0	2.13	1.53
		2.4	0.9	31.0	2.86	1.65
Area-2	300	1.8	-	4.54	1.12	0.54
		2.0	-	5.13	1.35	0.65
		2.2	-	5.73	1.60	0.77
		2.4	-	6.37	1.86	0.90
		2.6	-	7.02	2.15	0.95
	350	1.8	-	6.07	1.37	0.55
		2.0	-	6.82	1.64	0.67
		2.2	-	7.60	1.94	0.79
		2.4	-	8.41	2.25	0.92
		2.6	-	9.25	2.59	1.03

* Lateral Pile Capacities are applicable at cut-off level and correspond to 5 mm deflection for free head pile. Pile concrete grade is taken as M25.

The following points are highlighted with regard to the above capacities:

1. The above values are based on IS: 2911(Part-1): Section- 2, RA-2010 and include a factor of safety equal to 2.5 in compression and a safety factor 2.0 under uplift.

2. For piles socketed into rock, the pile capacities include a safety factor of 6 under compressive loading and 2.0 for pullout

3. Compression and pull-out capacities for piles of intermediate lengths may be linearly interpolated between the values given above.

4. Please note that for rock socketed piles, the compression capacity is derived primarily from the rock socket where rock strata are encountered. The pile length shall vary depending upon the depth to rock. The length of rock socket as given in the table above shall be ensured in order to achieve the desired capacities.

5. The pile toe level of drill hole should be cleaned properly so as to ensure that all rock fragments and disintegrated rock are removed. The pile should be seated on the natural undisturbed rock formation.

6. The lateral capacities given in the above table are for free-head pile considering permissible horizontal deflection as 5 mm at the pile head. Concrete grade is taken as M-25.

7. It should be ensured that the bottom of the pile bore is cleaned properly before casting the pile.

8. The capacities given above may be taken as a guideline for initial design. Final pile capacities should be confirmed by conducting initial pile load tests as per IS: 2911-Part-IV (2013). Also, routine load tests should be conducted on sufficient working piles so as to ensure that the working load on the piles is equal to or less than the design pile capacity.

5.7 Isolated Foundations

The following table presents our suggested values of net allowable bearing pressures for isolated foundations:

Area	Type of Foundation	Foundation Embedment Depth below EGL, m	Bearing Foundation Material	Suggested Net Allowable Bearing Pressure, T/m^2	Computed Settlement, mm	Modulus of Subgrade Reaction, k, kN/m^3
Area-1	Isolated Foundation	1.5	Rock	30.0	< 12 mm	25000
Area-2		1.5	Soil	26.0	< 40 mm	6500
		1.5		32.5	< 50 mm	
		2.0		31.0	<40 mm	7750
		2.0		38.7	< 50 mm	

The following points are highlighted with regard to the above recommended net bearing pressures:

1. The above bearing pressures include a bearing capacity safety factor of 2.5.
2. The appropriate values of net bearing pressure may be selected as per the permissible settlement criterion.
3. The soils at foundation level should be compacted thoroughly using a heavy roller. It should be ensured that there are no loose pockets at foundation level.
4. The suggested modulus of sub grade reaction (k) has been estimated as the ratio of the computed net bearing pressure and corresponding total settlement, and is applicable at the centre of the loaded area[(4)].

5.8 Definition of Gross and Net Bearing Pressure

For the purposes of this report, the net allowable bearing pressure should be calculated as the difference between total load on the foundation and the weight of the soil overlying the foundation divided by the effective area of the foundation. The gross bearing pressure is the total pressure at the foundation level including overburden pressure and surcharge load. The following equations may be used -

$$q_{net} = [(P_s + W_f + W_s) / A_f] - S_v$$

$$q_{gross} = q_{net} + S_v = (P_s + W_f + W_s) / A_f$$

where:

q_{net} = net allowable bearing pressure
q_{gross} = gross bearing pressure
P_s = superimposed static load on foundation
W_f = weight of foundation
W_s = weight of soil overlying foundation
A_f = effective area of foundation
S_v = overburden pressure at foundation level prior to excavation for foundation.

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[4] Bowles, J.E. (1996), "**Foundation Analysis and Design Fifth Edition**", The McGraw-Hill Companies Inc., pp. 503

It may please be noted that safe bearing pressures suggested in this report refer to **"net values"**. The gross bearing pressure may be computed by adding the overburden pressure to the net bearing pressure. Fill placed above EGL should be treated as a surcharge load.

6.0 FOUNDATION CONSTRUCTION CONSIDERATIONS

6.1 Excavation

Temporary open-cut excavations to about 1.5 m depth may be cut nearly vertically (1-vertical to 0.1~0.2 horizontal). These slopes are expected to remain stable except during rains. The engineer should monitor the slopes during excavations. In case excessive sloughing or caving occurs, the slope may be flattened further to ensure stability.

6.2 Foundation Level Preparation for Open Foundations

The area shall be excavated up to the foundation level. All loose soils should be removed and the exposed foundation bearing surface should be watered and compacted properly using rammers / rollers.

In case mechanical means like excavators are deployed for excavations, the excavations should be carried out up to 0.5 m below the proposed level. The last 0.5 m depth of excavation should be carried out manually, so that the founding soils are not disturbed / loosened.

The surface should be protected from disturbances due to construction activities so that the foundations may bear on the natural undisturbed ground. We recommend the placement of a 75 to 100 mm thick "blinding layer" of lean concrete to facilitate placement of reinforcing steel and to protect the soils from disturbance.

6.3 Chemical Attack

Results of chemical test on selected soil samples are presented on Plate 154.

The results indicate that the soils contain 0.010-0.11 percent sulphates and 0.01-0.02 percent chlorides. The pH value of soil is 7.7-7.9.

IS: 456-2000 recommends that precautions should be taken against chemical degradation of concrete if

- sulphates content of the soils exceeds 0.2 percent, or
- groundwater contains more than 300 mg /litre of sulphates (SO_3).

Comparing the test results with these specified limits, the sulphate content of the soil is less than the specified limit. Groundwater was not met to the maximum explored depth of 10.0 m. Therefore, strata at the site may be treated in **Class-I** category as described on IS: 456-2000.

We recommend the following measures as a good practice to limit the potential for chemical attack:

1. Foundation concrete should contain minimum cement content of 280 kg/m^3 of cement. Piles should contain at least 400 kg/m^3 of cement.
2. Water cement ratio in foundation concrete should not exceed 0.55.
3. A clear concrete cover over the reinforcement steel of at least 50 mm should be provided for all foundations.

4. Foundation concrete should be densified adequately using a vibrator so as to form a dense impervious mass.

7.0 VARIABILITY IN SUBSURFACE CONDITIONS

Subsurface conditions encountered during construction may vary somewhat from the conditions encountered during the site investigation. In case significant variations are encountered during construction, we request to be notified so that our engineers may review the recommendations in this report in light of these variations.