

<b>Index No.</b>
<b>37-FA24281K-G01-01</b>

**Maheshkhali Power Station Project**

**Feasibility Study**

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## **Geotechnical Engineering Investigation Report**



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Index No.	37-FA24281K-G01-01	Rev.	
Name	Geotechnical Engineering Investigation Report	Sheet	
Appendix	Layout of Exploratory Points, Geological Profile etc.	Sheet	

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## **CONTENTS**

1	Introduction .....	1
1.1	Project Information .....	1
1.2	Geographical conditions of the site .....	1
1.3	Technical Standards .....	2
1.4	Investigation Scheme .....	2
1.5	Investigation Methods .....	3
1.6	Quantity of Completed Works .....	6
1.7	Explanation Concerned.....	6
2	Regional Stability Evaluation .....	7
2.1	Regional Active Faults .....	7
2.2	Historical Earthquake.....	9
2.3	Ground motion.....	11
2.4	Stability Evaluation of The Proposed Site .....	12
3	Geological Engineering Condition .....	13
3.1	Geomorphology .....	13
3.2	Sub-soils.....	14
3.3	Surface Water .....	16
3.4	Frozen Ground.....	16
4	Geotechnical Engineering Evaluation.....	17
4.1	Physical and Mechanical Property Index of Subsoil .....	17
4.2	Determination of Allowable Bearing Capacity of Subsoils .....	24
4.3	Evaluation on Compressibility of Subsoil.....	25
4.4	Recommended Geotechnical Parameters of sub-soils .....	25
4.5	Evaluation on Corrosivity of Water.....	27
5	Seismic Effect .....	30
5.1	wave velocity test .....	30
5.2	Identification of construction site type and Classification of Soil .	34
5.3	Liquefaction .....	34
5.4	Classification of Seismic Fortification .....	40
6	Subgrade Evaluation of LNG Power Plant.....	41
6.1	Evaluation of Shallow Foundation .....	41
6.2	Pretreatment of backfill.....	41
6.3	Pile foundation evaluation of LNG Power Plant.....	41

7	Photovoltaic plant area.....	45
7.1	Geomorphology .....	45
7.2	Sub-soils.....	45
7.3	Surface Water and it's corrosivity.....	46
7.4	Subsoils Bearing Capacity of Photovoltaic plant area .....	46
7.5	Estimation of stability of Photovoltaic plant area.....	46
7.6	Liquefaction .....	46
7.7	Subgrade Evaluation of Photovoltaic plant area.....	46
8	Foundation pit excavation and dewatering.....	48
8.1	Foundation pit excavation.....	48
8.2	Pit dewatering.....	48
9	Adverse geological activities.....	49
10	Conclusions and Recommendations .....	50

### **Appendix**

No.	Name of Drawings	Sheet	Number of Drawings
1	Layout of Exploratory Points	1	37-FA24281K -G01-02
2	Geological Profile	6	37-FA24281K -G01-03~08
3	Catalogue of Exploratory Points	1	37-FA24281K -G01-09
4	Results of Laboratory Soil Tests	1	37-FA24281K -G01-10

# 1 Introduction

## 1.1 Project Information

Project Name: Maheshkhali Power Station Project

Installed Capacity: One LNG power plant with 2 sets of H-class gas-steam combined cycle units will be constructed in Block1 and Block2 of energy park in Maheshkhali island. Furthermore, a 160MWac photovoltaic project will be constructed in Block1 and Block2 near the LNG power plant.

Employer: Bay of Bengal Power Company (pvt.) Limited (hereafter referred as BBPCL). BBPCL is a joint venture company of Bangladesh Power Development Board (hereafter referred as BPDB) and China Huadian Hong Kong Company Limited (hereafter referred as CHDHK).

Project Site: West side of Maheshkhali island, Chattogram District, South-eastern of Bangladesh.

## 1.2 Geographical conditions of the site

The project will be in the block 1 and block 2 on the north of the energy island park which is on the south bank of Kuhelia River, Maheshkhali Upazila, Cox's Bazar, Chattogram. It is on the west of the Z1004 Road. Chattogram city is at a distance of 100 km in the NNW. Cox's Bazar Airport is in 23km southeast of the site.



Fig 1.2 location of the power plant site

### 1.3 Technical Standards

The main codes implemented in carrying out the investigation work are listed as follows:

*Code for geotechnical investigation of fossil fuel power plant* (GB/T51031-2014);

*Code for investigation of geotechnical engineering* (GB50021-2001);

*Code for seismic design of buildings* (GB50011-2010);

*Code for design of building foundation* (G50007-2011);

*Technical code for ground treatment of buildings* (JGJ79-2012);

*Technical code for building pile foundations* (JGJ94-2008);

*Occupational safety code for geotechnical investigation* (GB50585-2010);

*Ground treatment technical code for fossil fuel power plant* (DL/T5024-2005);

*Technical regulation for engineering geological boring of electrical power engineering* (DL / T5096-2008);

*Safety technical regulations for survey and exploration of electrical power engineering* (DL/T5334-2006);

*Technical code for geophysical exploration electric power engineering* (DL/T 5159-2012);

*Standard for soil test method* (GB/T 50123-1999);

*Technical code for rock and soil description of electric power engineering* (DL/T5160-2015);

*Technical code for data processing for investigation of electric power geotechnical engineering* (DL/T5093-2016) etc.

### 1.4 Investigation Scheme

According to specifications in chapter 3.1 of *Code for investigation of geotechnical engineering* (GB50021-2001), importance grade of the project is **Grade One**, site grade is **Grade Two** (moderate complex site),

ground grade is *Grade Two* (moderate complex ground), and geotechnical engineering investigation grade is *Grade One*.

#### 1.4.1 Principles of Exploratory Points Arrangement

Reasonable workload was arranged in accordance with the design requirements, related code and regulations. Correct investigation methods were adopted in the investigation work to ensure the quality of the investigation result. According to the above principles, and combined with the actual geological conditions of the site and the requirement of *Prior Guidance for Geotechnical Investigation*, the exploratory points are arranged as grid considering the layout of the plant, and the existing geotechnical investigation data was fully used.

#### 1.4.2 Number, Type, Spacing and Depth of Exploratory Points

##### 1.4.2.1 LNG Power Plant

Total 4 controlled boreholes with the depth from 70.26 to 80.28m were arranged in power station area and 4 general boreholes with the depth from 60.00 to 60.35m were arranged in LNG Power Plant.

##### 1.4.2.2 Photovoltaic Project Plant

Total 3 controlled boreholes with the depth from 30.40 to 30.45m were arranged in power station area and 3 general boreholes with the depth from 25.30 to 25.45m were arranged in Photovoltaic Project Plant.

### **1.5 Investigation Methods**

#### 1.5.1 Field works

Field investigation methods include drilling, extraction of disturbed and undisturbed soil samples and SPT. The description of each method is as follows:

##### (a) Drilling

Rotary drilling machine which's essential components include the drilling rig with rotary power, hoisting systems, casing, rods, core barrels, and pumps with circulating system were used in this project. In the process of drilling, the borehole wall was protected with bentonite slurry or casing. Initial and standing groundwater levels were measured and

recorded during the process.

(b) Extraction of Undisturbed Soil Sample (UDS)

Undisturbed Soil Samples were collected in boreholes with thin-wall sampler after reaching the planned sampling depth and clearing the unembedded soil (thickness of unembedded soil on the bottom should be less than 10cm).

The sampler should be fall smoothly and steadily and not impact the bottom of the hole. Check the depth of borehole and sampling, length of rigs to ensure thickness of residual drift soil is less than 10cm, otherwise, lift sampler and clear hole again.

Penetration depth of sampler was less 90% of its total length, measure and record sampling depth after each collecting.

(c) Extraction of Disturbed Soil Sample (DS)

Disturbed soil samples were collected from penetrator of SPT, and reserved in plastic bags with tags.

(d) Execution of Standard Penetration Test (SPT)

Standard penetration test is performed during boring/drilling. When boring to 15cm above planned test depth, clear the unembedded soil, then perform the test.

The penetrator shall be driven into the soil by dropping a hammer of 63.5kg mass from a height of 76cm freely. After the penetrator is driven 15cm into the ground at the bottom of the borehole, the blow count for each 10cm penetration should be recorded, and the accumulative total number of blows necessary to achieve a penetration of 30cm is N value.

If blow counts have reached 50, while the penetration depth is not reaching 30cm, record the practical penetration depth of the 50 counts, and calculate blow counts of standard penetration test N equivalent to 30 cm.

(e) Groundwater Level Measurement

Groundwater level was observed during boring and the depth at which ground water was encountered and the standing water levels were

recorded in the borehole descriptions. Whenever ground water was struck, boring was stopped for half an hour to allow the water level to stabilise. The depth of the water table was then measured. And 24 hours after completion of borehole a further reading of water level was taken. The boreholes were then backfilled.

#### 1.5.2 Laboratory Testing

Test items mainly include physical property test, consolidation test, shear test.

##### (a) Physical Property Test

For UDS, the following parameters was tested:

Sand: specific gravity, natural water content, natural density, particle size composition;

Silt: specific gravity, natural water content, natural density, particle size composition, Atterberg limits;

Clay: specific gravity, natural water content, natural density, particle size composition, Atterberg limits.

For DS of silt and sand, particle size composition was tested.

##### (b) Consolidation Test

Consolidation test was carried on proposed UDS, coefficient of compressibility  $a_{1-2}$  and modulus of compressibility  $E_s$  within pressure range 100~200kPa were calculated accordingly.

##### (c) Direct Shear Test

Direct shear test was applied on UDS, the test method was quick shear method, and cohesive strength and internal friction angle of each testing sample was offered.

##### (d) Permeability testing of soil

Permeability test was performed on UDS collected in proposed boreholes, vertical permeability coefficient  $K_v$  and horizontal permeability coefficient  $K_H$  were tested.

##### (e) soil and groundwater corrosivity test

According to Code for investigation of geotechnical engineering

(GB50021-2001), PH value, content of  $\text{Cl}^-$  and  $\text{SO}_4^{2-}$  is measured which are relevant to corrosivity to concrete and steel bar.

### 1.6 Quantity of Completed Works

Completed field investigation work load and laboratory tests work load are separately listed in table 1.6-1 and 1.6-2.

Table 1.6-1 Quantity of Field Investigation Works

Type	Quantity	Depth (m)	Total footages (m)	Undisturbed sample (Piece)	Disturbed sample (Piece)	SPT (Time)
Controlled Borehole	7	30.30~80.28	386.97	109	27	244
General Borehole	7	25.31~60.35	321.65	77	19	204
Total	14		708.62	186	46	448

Table 1.6-2 Quantity of Laboratory Testing Works

Items	Quantity (Piece)	Items	Quantity (Piece)
Water Content Test	186	Consolidation Test(50kPa~400kPa)	80
Density Test	186	Consolidation Test(25kPa~3200kPa)	63
Specific Gravity Test	186	Direct Shear Test	84
Particle Size Analysis Test	122	Permeability Test	27
Atterberg Limits Test	124	Chemical Testing of Groundwater	8

### 1.7 Explanation Concerned

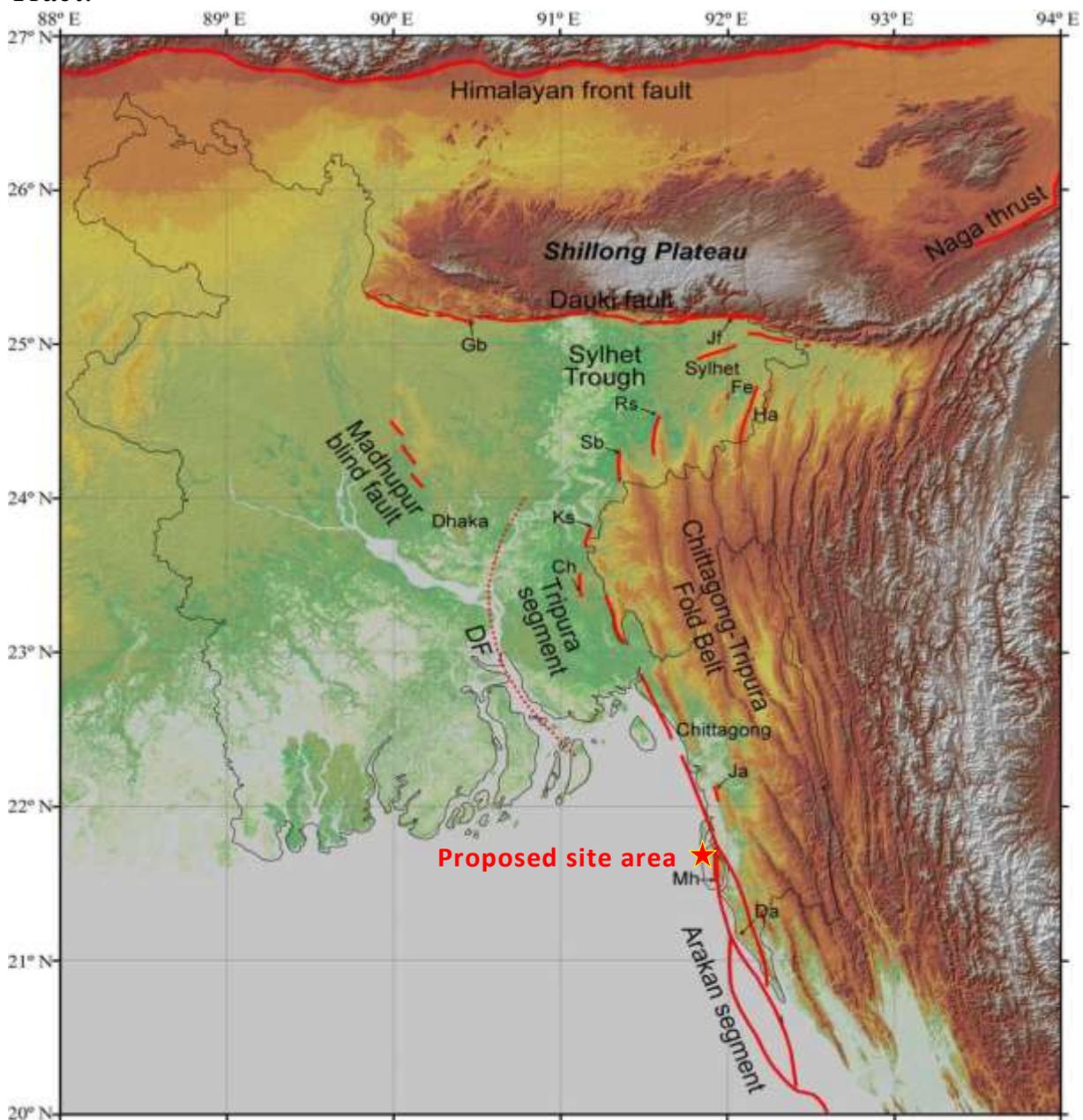
(1) Field investigation works began with June 20th, 2021 and ended in September 15th, 2021.

(2) The coordinate system of the exploration point adopts the UTM coordinate system, and the elevation system is Mean Sea Level.

## 2 Regional Stability Evaluation

### 2.1 Regional Active Faults

Bangladesh stands on the northeastern corner of the Indian plate. There are two major structures relative to the plate boundary, namely the Dauki fault and the Indian-Burman plate boundary fault. The strike of the Dauki fault is parallel to the Himalayan Front fault, so this fault is thought to be related to the Indian-Eurasian plate boundary fault. Several active faults are suggested within the Chittagong-Tripura Fold Belt, and the Madhupur blind fault is inferred on the western margin of the Madhupur Tract.



**Fig.2.1 Distribution Map of Active Faults in Bangladesh**

### (1) Dauki Fault

The Himalayan Front Fault (HFF) as a mega-thrust is developed on the collision boundary between the Indian and Eurasian plates. The Shillong Plateau stands on the south of the HFF, and the Dauki fault, which is a north-dipping reverse fault, passes on the southern margin of the Shillong Plateau (Fig.2.2). The Shillong Plateau is composed of bedrocks of the Indian shield, which was uplifted by the activity of the Dauki fault with 2000 m high. The Dauki fault is thought to be an active fault related to the collision boundary, since its strike is parallel to the HSS, while it is an intra-plate active fault within the Indian plate.

The southern part of the Shillong Plateau shows a positive gravity anomaly over +20 mgal, while the Alluvial Plain on the south of the Shillong Plateau shows a remarkable negative gravity anomaly over -80 mgal. The gravity anomaly contours are closely arranged with E-W direction on the southern margin of the Shillong Plateau. The Shillong Plateau shows a symmetrical topographic profile with the southern steep slope and the northern gentle slope. These gravity anomaly data and geomorphic features strongly suggest the presence of the Dauki fault.

### (2) Active Faults within Chittagong-Tripura Fault Belt

Most of active faults within Chittagong-Tripura Fold Belt (CTFB) is thought to be secondary faults and deformations related to the rupture of the Tripura segment. However, a part of these faults may generate large earthquakes separately from the plate boundary fault like the 1918 Srimongal earthquake.

### (3) Madhupur Blind Fault

The Madhupur blind fault is inferred on the western margin of the Madhupur Tract, northwest of Dhaka (Fig.2.2). The fault is considered as an important structure for the seismic hazard assessment of Dhaka, the capital of Bangladesh.

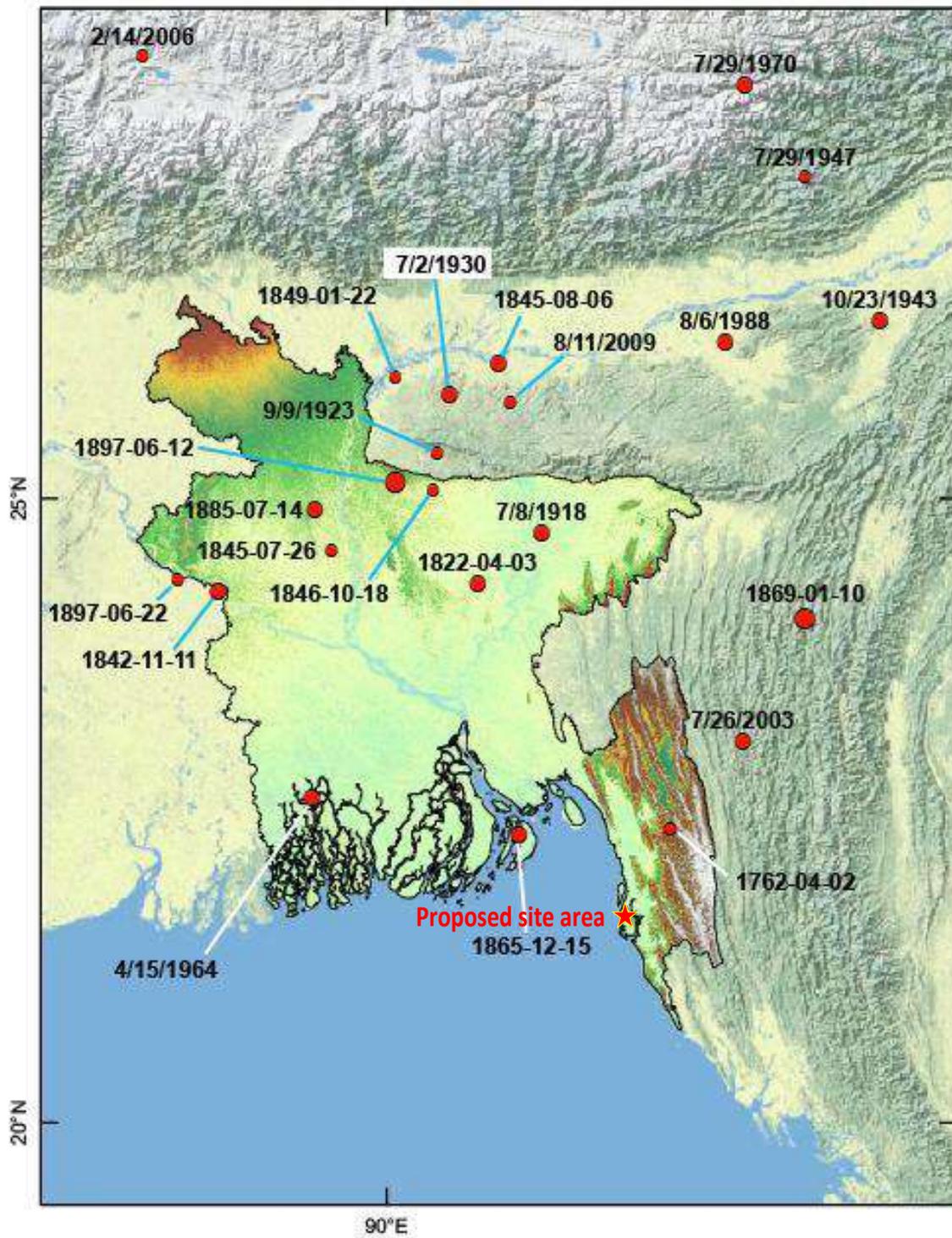
The Madhupur Tract is characterized by red-color residual deposits on the surface. It is thought that the alluvium in the Quaternary time was

uplifted and the soil on the surface was changed into red-color clay by weathering. The surface of Madhupur Tract is tilted to the east. The straight low scarps with NW-SE direction are recognized on the western margin of the Madhupur Tract (Fig.2.2). The low scarps seem to be erosional. However, it is sure that the Madhupur Tract is uplifted and tilted to the east by the faulting. The flexural scarps, which have been uplifted by the activity of the Madhupur blind fault, may be eroded and shifted to the east.

## **2.2 Historical Earthquake**

Bangladesh and the north eastern Indian states have long been one of the seismically active regions of the world, and have experienced numerous large earthquakes during the past 250 years. Large historical earthquakes ( $M > 6$ ) since the 1762 earthquake in and around Bangladesh are shown in Fig. 2.2.

According to the information from the web site of department of disaster management ministry of disaster management and relief, the record of approximately 150 years shows that Bangladesh and the surrounding regions experienced seven major earthquakes (with  $M_b > 7$ ) shown in table 2.3. In the recent past, several tremors of moderate to severe intensity had already taken place in and around Bangladesh. The Sylhet Earthquake ( $M_b=5.6$ ) of May 8, 1997, the Bandarban Earthquake ( $M_b=6.0$ ) of November 21, 1997, the Moheshkhali Earthquake ( $M_b=5.1$ ) of July 22, 1999, and the Barkal (Rangamati) Earthquake ( $M_b=5.5$ ) of July 27, 2003 may be cited as examples (Source: Choudhury, 2005).



**Fig.2.2 Distribution Map of Major Historical Earthquakes in and Around Bangladesh(M>6)**

**Table 2.2 List of Major Earthquakes Affecting Bangladesh**

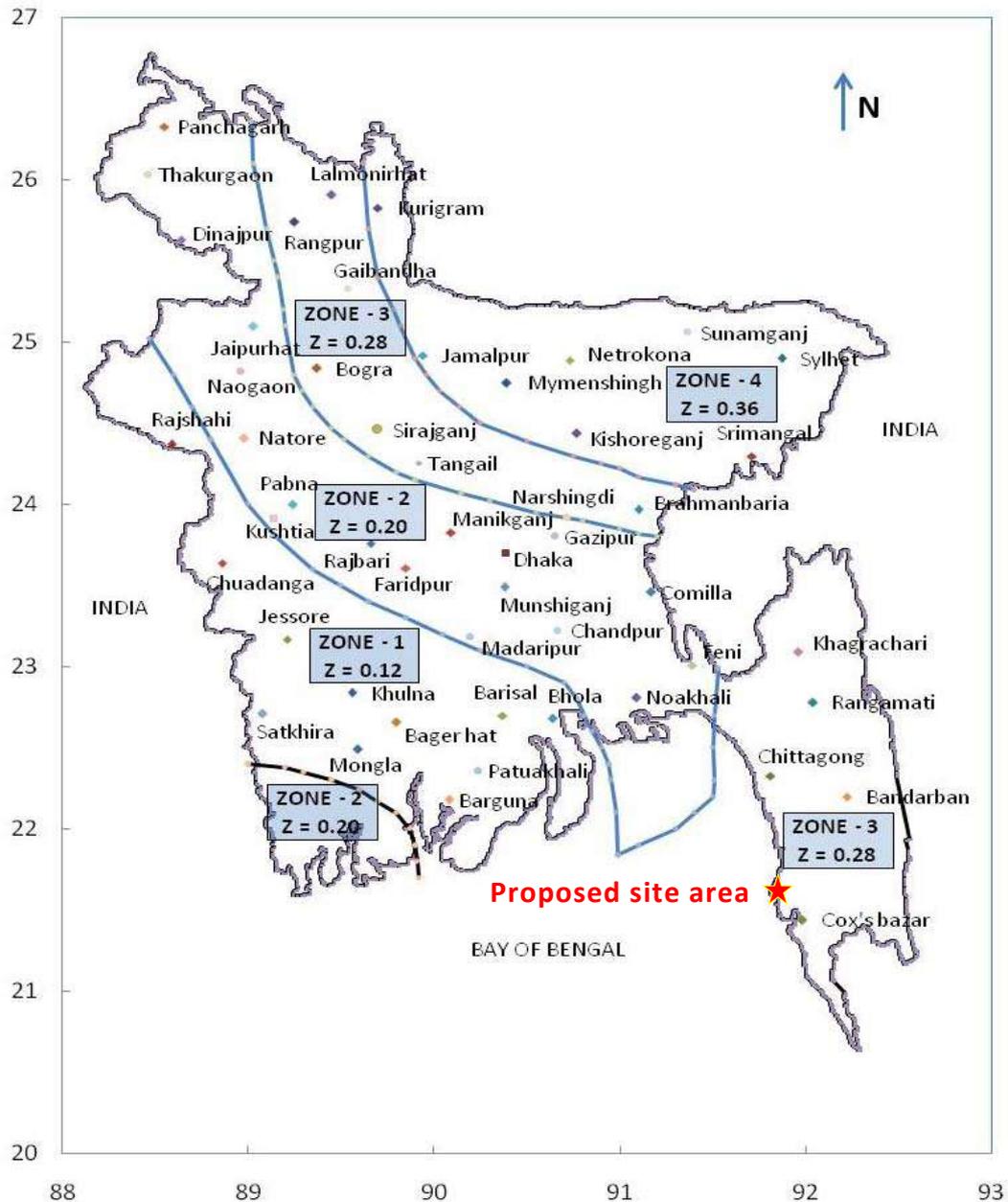
No	Date	Name	Magnitude (Richter)	Epicentral Distance from Dhaka (km)	Epicentral Distance from Sylhet City (km)	Epicentral Distance from Chittagong (km)
1	1869.01.10	Cachar Earthquake	7.5	250	70	280
2	1885.07.14	Bengal Earthquake	7.0	170	220	350
3	1897.06.12	Great Indian Earthquake	8.7	230	80	340
4	1918.07.08	Srimongal Earthquake	7.6	150	60	200
5	1930.07.02	Dhubri Earthquake	7.1	250	275	415
6	1934.01.15	Bihar-Nepal Earthquake	8.3	510	530	580
7	1950.08.15	Assam Earthquake	8.5	780	580	540

According to the record data of historical earthquake in Bangladesh and the surrounding regions, there is no earthquake with magnitude greater than 7 experienced near the proposed site. The site is in an area with relatively small impact of earthquake. The earthquake impact on this project is mainly distant earthquake impact outside the region.

### 2.3 Ground motion

According to Seismic Zoning Map of Bangladesh, Part6 Fig. 2.5.1 in Bangladesh National Building Code 2010, shown in Fig. 错误!未找到引用源。., the proposed site is in zone-3 with seismic zone coefficient  $Z=0.28$ , which represents the ground motion peak acceleration with 2% exceeding probability during the 50 years reference period is 0.28 gon very stiff soil/rock ( $V_s > 800\text{m/s}$ ).

Converted in accordance with Code for seismic design of buildings (GB50011-2010), the ground motion peak acceleration of the proposed site with 10% exceeding probability during the 50-years reference period should be 0.224 in case of Class IV. The seismic precautionary intensity should be 8 degree and the suggested design earthquake group is Group IV.



**Fig.2.4 Seismic Zoning Map of Bangladesh**

### **2.4 Stability Evaluation of The Proposed Site**

The distance between the site and active fault is about 2km, and meets the requirement of Code for geotechnical investigation of fossil fuel power plant (GB/T 51031 - 2014). Based on the analysis of regional geology, fracture structure, historical earthquake and ground motion parameters in the area where the proposed site is located, it is considered that the proposed site is in a relatively stable area and suitable for large-scale engineering construction.

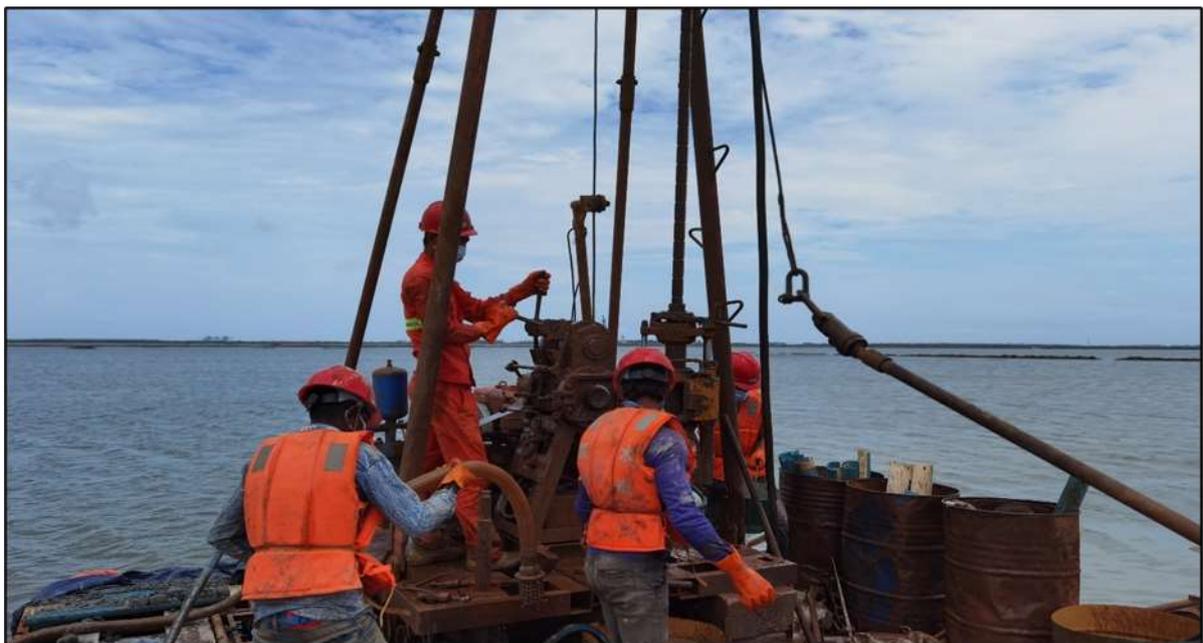
### **3 Geological Engineering Condition**

#### **3.1 Geomorphology**

The geomorphic origin of the proposed plant site is coastal plain, rice field in the east, shrimp pond in the west, open terrain, flat terrain, and generally 0.71-1.44m above ground. Seawater is poured into the plant area during high tide, water in the plant area is poured into the river channel during ebb tide, and the water depth in the plant area is about 0.5-1.5 m during the survey period. The google map and photography of the proposed site take during the field work are shown as Fig3.1-1 ~ 3.1-2



**Fig.3.1-1 Google Map of the Proposed Site**



**Fig.3.2-2 Photography of the Proposed Site During Field Work**

### 3.2 Sub-soils

The sub-soils as revealed from investigations so far indicate that the constituent material of sub-soil comprises clay, silty clay, silty fine sand and silt clay with fine sand which belong to quaternary alluvium. The descriptions of each soil layer from new to old and top to bottom are listed as below:

① plain fill: The constituent material mainly comprises grey or brownish grey, very soft, saturated clay and silty clay.

② clay: The constituent material mainly comprises soft, saturated, taupe or yellowish-brown mucky silty clay. This layer is widely distributed in the site.

③ silty clay: The constituent material mainly comprises soft, saturated, grey silty clay. This layer is widely distributed in the site.

③-1 silty fine sand: The constituent material mainly comprises very loose, saturated, grey silty fine sand. This layer is partially distributed in the site.

③-2 silty sand: The constituent material mainly comprises medium dense, saturated, grey silty fine sand. This layer is widely distributed in the site.

④ silty sand: The constituent material mainly comprises medium dense / dense, saturated, grey silty fine sand. This layer is widely distributed in the site.

④-1 silty clay with silty sand interbedded: The constituent material mainly comprises soft, saturated, brownish grey or yellowish-brown silty clay with thickness varies from 2mm to 20mm and interbedded with loose dense, saturated, grey or yellowish-brown silty sand with thickness varies from 2mm to 18mm. This layer is partially distributed in the site.

⑤ silt clay with fine sand: The constituent material mainly comprises hard, saturated, grey silty clay with fine sand which exist in the form of intercalation. This layer is widely distributed in the site.

⑥ silty fine sand: The constituent material mainly comprises very dense, saturated, grey silty fine sand. This layer is widely distributed in the site.

⑥-1 silty clay: The constituent material mainly comprises hard, saturated grey silty clay. This layer is partially distributed in the site.

⑦ silt clay with fine sand: The constituent material mainly comprises hard, saturated, grey silty clay with fine sand which exist in the form of intercalation. This layer is widely distributed in the site.

⑦-1 silty fine sand: The constituent material mainly comprises very dense, saturated, grey silty fine sand. This layer is partially distributed in the site.

⑧ silty fine sand: The constituent material mainly comprises very dense, saturated, grey silty fine sand. This layer is widely distributed in the site.

⑧-1 silt clay with fine sand: The constituent material mainly comprises hard, saturated, grey silty clay with fine sand which exist in the form of intercalation. This layer is partially distributed in the site.

⑨ silt clay with fine sand: The constituent material mainly comprises hard, saturated, grey silty clay with fine sand which exist in the form of intercalation. This layer is widely distributed in the site.

⑨-1 silty fine sand: The constituent material mainly comprises very dense, saturated, grey silty fine sand. This layer is widely distributed in the site.

Table 3.2 The Distribution of Subsoil

Stratum	Thickness (m)			Depth of the bottom (m)			Elevation of the bottom (m)		
	Minimum	Maximum	Mean	Minimum	Maximum	Mean	Minimum	Maximum	Mean
①	0.40	0.90	0.67	0.40	0.90	0.67	-0.09	0.81	0.38
②	1.90	4.80	3.29	1.90	5.50	3.80	-4.29	-0.92	-2.76
③	0.55	7.30	2.64	5.50	13.10	7.89	-12.12	-4.48	-6.81
③-1	0.70	6.20	3.17	3.60	11.70	7.87	-10.49	-2.58	-6.85
③-2	1.95	6.90	4.33	7.60	13.10	10.80	-12.39	-6.46	-9.73
④	0.60	8.20	3.97	13.10	22.40	17.33	-20.98	-12.24	-16.30
④-1	2.00	7.00	3.56	12.60	18.10	15.17	-16.79	-11.49	-14.08
⑤	1.35	11.10	5.29	18.20	29.10	22.53	-27.89	-17.18	-21.43
⑥	3.40	9.40	6.46	26.30	37.00	30.31	-35.56	-25.26	-29.25
⑥-1	0.90	5.10	2.89	25.10	30.70	28.15	-29.49	-24.05	-26.94
⑦	1.85	13.50	7.94	32.40	47.40	38.76	-45.96	-31.46	-37.70
⑦-1	4.05	4.05	4.05	31.40	31.40	31.40	-30.69	-30.69	-30.69
⑧	1.50	11.90	5.25	37.70	49.20	44.14	-47.76	-36.66	-43.08
⑧-1	1.00	1.00	1.00	35.40	35.40	35.40	-34.69	-34.69	-34.69
⑨	3.00	17.80	11.48	51.70	65.40	57.02	-64.26	-50.68	-55.95
⑨-1	6.30	11.50	8.90	59.50	65.30	62.40	-64.09	-58.64	-61.37

### **3.3 Surface Water**

The site area is shrimp pond. Due to the blocking effect of surrounding dams and the backflow of tidal water during high tide, water accumulated easily in rainy season, which can only be discharged by several small rivers during evaporation and ebb tide. The perennial accumulated water depth is about 0.5-1.5m, the highest accumulated water level is 2.4m, and the highest flood level affected by storm surge is 5.45m.

### **3.4 Frozen Ground**

According to the meteorological data of Bangladesh, the annual minimum temperature of the project site is higher than 0°C, so the influence of frozen ground on foundation design can be not taken into account.

## **4 Geotechnical Engineering Evaluation**

### **4.1 Physical and Mechanical Property Index of Subsoil**

#### 4.1.1 Statistical Analysis on Soil Test results

Total 122 undisturbed soil samples and 186 disturbed soil samples were collected in the investigation work. Based on results of soil tests, statistic of physical and mechanical property index of foundation soil is proceed as per corresponding statistic calculation principles and methods in *Code for design of building foundation (G50007-2012)* and *Technical code for data processing for investigation of electric power geotechnical engineering(DL/T5093-2016)*.The statistical Analysis on the physical and mechanical properties of subsoil are shown in table 4.1.1.

Table 4.1.1

Statistical analysis on the Physical and Mechanical Properties of Subsoil

Stratum	Statistic Result item	Moisture Content W (%)	Unit Weight $\gamma$ (kN/m <sup>3</sup> )	Natural Void Ratio $e_0$	Saturation Sr (%)	liquid limit $W_L$ %	plastic limit $W_P$ %	Plasticity index $I_p$ -	liquid index IL -	Direct shear test		Compressibility Coefficient $a_{1-2}$ (MPa <sup>-1</sup> )	Compressive Modulus $E_s$ (MPa)	vertical Permeability $K_v$ (cm/s)
										cohesion c (kPa)	friction $\phi$ (degree)			
②clay	Frequency N	9	9	9	9	9	9	9	9	7	7	7	7	4
	Arithmetical mean $\mu$	45.0	17.4	1.292	95.9	47.9	28.2	19.7	0.86	17	8.7	0.85	2.8	8.43E-06
	Maximum value $U_{max}$	51.3	18.2	1.486	100.0	54.1	31.5	24.1	0.96	21	13.3	1.02	4.1	2.31E-05
	Minimum value $U_{min}$	39.5	1.67	1.120	92.0	41.5	23.8	16.3	0.76	15	6.2	0.53	2.3	1.26E-06
	Standard deviation $\sigma$	4.2	0.05	0.124					0.07	1.74	2.28	0.20	0.7	
	Coefficient of variation $\delta$	0.09	0.03	0.10					0.09	0.10	0.26	0.24	0.25	
	Coefficient of correction $\gamma$	1.06	0.98	1.06					1.05	0.93	0.81	1.17	0.82	
	Standard value	47.7	17.1	1.370					0.90	16	7.0	1.00	2.3	
③silty clay	Frequency N	8	8	8	8	8	8	8	8	3	3	8	8	4
	Arithmetical mean $\mu$	41.1	17.6	1.202	93.5	42.8	26.2	16.6	0.88	18	10.3	0.91	2.7	4.5E-06
	Maximum value $U_{max}$	52.7	18.9	1.456	100.0	53.9	32.2	24.1	1.01	19	14.8	1.31	4.0	1.47E-05
	Minimum value $U_{min}$	27.2	16.4	0.831	88.0	30.7	18.2	11.4	0.72	16	7.2	0.46	1.9	3.05E-08
	Standard deviation $\sigma$	9.5	0.09	0.263					0.09			0.34	0.8	
	Coefficient of variation $\delta$	0.23	0.05	0.22					0.10			0.38	0.30	
	Coefficient of correction $\gamma$	1.16	0.97	1.15					1.07			1.25	0.80	
	Standard value	47.5	17.0	1.380					0.95			1.14	2.1	

Table 4.1.1

Statistical analysis on the Physical and Mechanical Properties of Subsoil

Stratum	Statistic Result item	Moisture Content W (%)	Unit Weight $\gamma$ (kN/m <sup>3</sup> )	Natural Void Ratio $e_0$	Saturation Sr (%)	liquid limit W <sub>L</sub> %	plastic limit W <sub>P</sub> %	Plasticity index Ip -	liquid index IL -	Direct shear test		Compressio Coefficient a <sub>1-2</sub> (MPa <sup>-1</sup> )	Compressio Modulus Es (MPa)	vertical Permeability Kv (cm/s)
										cohesion c (kPa)	friction $\phi$ (degree)			
④-silty clay	Frequency N	14	13	13	13	13	13	13	13	6	6	12	12	3
	Arithmetical mean $\mu$	31.8	18.8	0.920	93.4	39.5	24.6	14.9	0.50	31	14.5	0.33	7.1	1.76E-06
	Maximum value U <sub>max</sub>	49.1	199	1.426	100.0	55.6	32.7	22.9	0.93	75	20.2	0.62	11.8	4.87E-06
	Minimum value U <sub>min</sub>	23.2	16.9	0.699	85.0	30.1	18.9	10.4	0.02	14	7.9	0.15	3.7	5.67E-08
	Standard deviation $\sigma$	8.8	0.09	0.239					0.35	23.4	4.38	0.18	3.1	
	Coefficient of variation $\delta$	0.28	0.05	0.26					0.70	0.75	0.30	0.54	0.43	
	Coefficient of correction $\gamma$	1.13	0.97	1.13					1.33	0.38	0.75	1.28	0.78	
	Standard value	36.0	18.4	1.040					0.67	12	10.9	0.42	5.5	
⑤silt clay with fine sand	Frequency N	23	23	23	23	22	22	22	22	16	16	22	22	4
	Arithmetical mean $\mu$	27.5	19.2	0.806	92.7	35.5	21.5	14.1	0.42	36	16.1	0.28	6.9	4.04E-06
	Maximum value U <sub>max</sub>	38.1	19.7	1.087	99.0	46.8	26.6	20.3	0.72	70	23.4	0.45	10.3	9.75E-06
	Minimum value U <sub>min</sub>	22.6	18.2	0.687	83.0	29.1	18.8	10.2	0.13	22	7.5	0.17	4.1	1.07E-06
	Standard deviation $\sigma$	3.9	0.04	0.097					0.15	12.59	4.41	0.08	1.7	
	Coefficient of variation $\delta$	0.14	0.02	0.12					0.37	0.35	0.27	0.29	0.25	
	Coefficient of correction $\gamma$	1.05	0.99	1.04					1.14	0.85	0.88	1.11	0.91	
	Standard value	28.9	19.1	0.841					0.48	31	14.2	0.31	6.3	

Table 4.1.1

Statistical analysis on the Physical and Mechanical Properties of Subsoil

Stratum	Statistic Result item	Moisture Content W (%)	Unit Weight $\gamma$ (kN/m <sup>3</sup> )	Natural Void Ratio $e_0$	Saturation Sr (%)	liquid limit $W_L$ %	plastic limit $W_P$ %	Plasticity index $I_p$ -	liquid index IL -	Direct shear test		Compression Coefficient $a_{1-2}$ (MPa <sup>-1</sup> )	Compression Modulus $E_s$ (MPa)
										cohesion c (kPa)	friction $\phi$ (degree)		
⑥silt clay	Frequency N	5	5	5	5	5	5	5	5	4	4	2	2
	Arithmetical mean $\mu$	25.8	19.7	0.743	94.6	35.0	21.9	13.1	0.33	41	17.6	0.24	7.4
	Maximum value $U_{max}$	26.6	20.1	0.785	97.0	40.6	24.4	16.2	0.55	66	20.3	0.26	8.0
	Minimum value $U_{min}$	23.8	19.2	0.681	91.0	31.2	20.8	10.4	0.10	25	13.5	0.22	6.9
⑦silt clay with fine sand	Frequency N	13	13	13	13	13	13	13	13	7	7	11	11
	Arithmetical mean $\mu$	22.5	20.0	0.669	91.2	36.8	22.7	14.1	-0.01	65	20.4	0.15	11.3
	Maximum value $U_{max}$	27.7	20.4	0.779	99.0	44.7	26.1	19.2	0.16	93	23.5	0.18	14.5
	Minimum value $U_{min}$	18.3	19.5	0.589	85.0	31.6	19.1	11.0	-0.31	41	16.6	0.11	9.3
	Standard deviation $\sigma$	2.9	0.03	0.059					0.14	16.67	2.75	0.03	2.1
	Coefficient of variation $\delta$	0.13	0.01	0.09					-12.02	0.25	0.13	0.19	0.18
	Coefficient of correction $\gamma_s$	1.06	0.99	1.04					5.01	0.81	0.90	1.10	0.90
	Standard value	23.9	19.9	0.699					-0.06	53	18.3	0.17	10.2
⑨silt clay with fine sand	Frequency N	19	18	18	18	19	19	19	19	9	9	17	17
	Arithmetical mean $\mu$	22.7	19.9	0.684	91.6	38.4	23.1	15.2	-0.01	70	19.3	0.16	10.9
	Maximum value $U_{max}$	25.9	20.8	0.762	100.0	48.3	27.4	21.3	0.35	123	26.3	0.21	15.1
	Minimum value $U_{min}$	17.8	19.3	0.535	84.0	29.3	18.8	10.2	-0.25	30	15.4	0.11	8.1
	Standard deviation $\sigma$	2.0	0.05	0.063					0.15	29.62	3.38	0.03	1.9
	Coefficient of variation $\delta$	0.09	0.02	0.09					-15.28	0.42	0.18	0.18	0.17
	Coefficient of correction $\gamma_s$	1.04	0.99	1.04					5.17	0.74	0.89	1.08	0.93
	Standard value	23.6	19.7	0.710					-0.05	52	17.2	0.17	10.1

#### 4.1.2 Statical Analysis on Results of SPT

According to *Code for investigation of geotechnical engineering* (GB50021-2001) (2009 Edition), data of standard penetration test are counted layer by layer, and statistic results are listed in table 3.1.2 in which the rod length is modified according to table 3-3-2 in *Handbook of geotechnical engineering* (Fifth Edition).

Table 4.1.2 Statistical Analysis on the Results of SPT

No.	Stratum	Statistic item	Statistic results	
			Measured(Blow)	Corrected (Blow)
②	clay	Frequency N	24	24
		Arithmetical mean $\mu$	2.1	2.0
		Standard deviation $\sigma$	0.9	0.9
		Coefficient of variation $\delta$	0.45	0.44
		Coefficient of correction $\gamma_s$	0.84	0.85
		Standard value	1.8	1.7
③	silty clay	Frequency N	20	20
		Arithmetical mean $\mu$	6.3	5.4
		Standard deviation $\sigma$	2.0	1.6
		Coefficient of variation $\delta$	0.33	0.29
		Coefficient of correction $\gamma_s$	0.87	0.89
		Standard value	5.4	4.7
③-1	silty fine sand	Frequency N	20	20
		Arithmetical mean $\mu$	8.3	7.1
		Standard deviation $\sigma$	2.6	2.1
		Coefficient of variation $\delta$	0.31	0.29
		Coefficient of correction $\gamma_s$	0.88	0.88
		Standard value	7.2	6.2
③-2	silty sand	Frequency N	26	26
		Arithmetical mean $\mu$	20.8	17.3
		Standard deviation $\sigma$	4.0	3.1
		Coefficient of variation $\delta$	0.19	0.18
		Coefficient of correction $\gamma_s$	0.94	0.94
		Standard value	19.5	16.3

Table 4.1.2 Statistical Analysis on the Results of SPT

No.	Stratum	Statistic item	Statistic results	
			Measured(Blow)	Corrected (Blow)
④	silty sand	Frequency N	36	36
		Arithmetical mean $\mu$	31.3	22.9
		Standard deviation $\sigma$	7.6	5.3
		Coefficient of variation $\delta$	0.24	0.23
		Coefficient of correction $\gamma_s$	0.93	0.93
		Standard value	29.1	21.3
④-1	silty clay with silty sand interbedded	Frequency N	23	23
		Arithmetical mean $\mu$	12.1	9.2
		Standard deviation $\sigma$	3.6	2.6
		Coefficient of variation $\delta$	0.29	0.29
		Coefficient of correction $\gamma_s$	0.89	0.90
		Standard value	10.8	8.2
⑤	silt clay with fine sand	Frequency N	51	51
		Arithmetical mean $\mu$	31.0	21.0
		Standard deviation $\sigma$	7.7	4.9
		Coefficient of variation $\delta$	0.25	0.23
		Coefficient of correction $\gamma_s$	0.94	0.94
		Standard value	29.2	19.8
⑥	silty fine sand	Frequency N	43	43
		Arithmetical mean $\mu$	63.3	40.0
		Standard deviation $\sigma$	17.1	9.8
		Coefficient of variation $\delta$	0.27	0.24
		Coefficient of correction $\gamma_s$	0.93	0.94
		Standard value	58.8	37.5
⑥-1	silty clay	Frequency N	12	12
		Arithmetical mean $\mu$	48.1	30.3
		Standard deviation $\sigma$	10.7	6.1
		Coefficient of variation $\delta$	0.22	0.20
		Coefficient of correction $\gamma_s$	0.88	0.89
		Standard value	42.5	27.1

Table 4.1.2 Statistical Analysis on the Results of SPT

No.	Stratum	Statistic item	Statistic results	
			Measured(Blow)	Corrected (Blow)
⑦	silt clay with fine sand	Frequency N	35	35
		Arithmetical mean $\mu$	58.9	33.5
		Standard deviation $\sigma$	13.7	8.1
		Coefficient of variation $\delta$	0.23	0.24
		Coefficient of correction $\gamma_s$	0.93	0.93
		Standard value	54.9	31.2
⑦-1	silty fine sand	Frequency N	2	2
		Arithmetical mean $\mu$	68.0	41.7
		Standard deviation $\sigma$	/	/
		Coefficient of variation $\delta$	/	/
		Coefficient of correction $\gamma_s$	/	/
		Standard value	68.0	41.7
⑧	silty fine sand	Frequency N	23	23
		Arithmetical mean $\mu$	102.0	56.3
		Standard deviation $\sigma$	10.2	5.9
		Coefficient of variation $\delta$	0.10	0.10
		Coefficient of correction $\gamma_s$	0.96	0.96
		Standard value	98.3	54.2
⑧-1	silt clay with fine sand	Frequency N	1	1
		Arithmetical mean $\mu$	28.0	16.2
		Standard deviation $\sigma$	/	/
		Coefficient of variation $\delta$	/	/
		Coefficient of correction $\gamma_s$	/	/
		Standard value	28.0	16.2
⑨	silt clay with fine sand	Frequency N	56	56
		Arithmetical mean $\mu$	74.8	38.7
		Standard deviation $\sigma$	17.7	8.8
		Coefficient of variation $\delta$	0.24	0.23
		Coefficient of correction $\gamma_s$	0.95	0.95
		Standard value	70.8	36.7
⑨-1	silty fine sand	Frequency N	49	49
		Arithmetical mean $\mu$	103.8	53.1
		Standard deviation $\sigma$	15.6	7.9
		Coefficient of variation $\delta$	0.15	0.15
		Coefficient of correction $\gamma_s$	0.96	0.96
		Standard value	100.0	51.1

## 4.2 Determination of Allowable Bearing Capacity of Subsoils

### 4.2.1 $f_{ak}$ Determined by Physical and Mechanical Parameters

According to statistical data and calculated results above, as per table 4-4-8 and 4-4-9 in engineering geological manual, allowable values of silty clay layers bearing capacity are determined shown in table 4.2.1.

Tab 4.2.1 Results Determined by Physical and Mechanical Parameters

Stratum	$f_0$ (kPa)	$\psi_f$	$f_{ak}$ (kPa)
② clay	<90	0.884	<80
③ silty clay	<90	0.874	<80

### 4.2.2 $f_{ak}$ Determined by SPT Value

According to SPT value after rod length corrected, as per table 3-2-35 (suitable for silty clay), table 4-4-7 (suitable for sand) and formula  $f_{ak}=72+9.4N^{1.2}$  (suitable for silt) in engineering geological manual (third edition), allowable values of silt, silty clay, and silty sand bearing capacity are determined as shown in table 4.2.2:

Table 4.2.2 Calculated Results Determined by SPT Values

Stratum	SPT N-value (Blow)	$f_{ak}$ (kPa)
② clay	1.7	< 105
③ silty clay	4.7	139
③-1 silty fine sand	6.2	< 140
③-2 silty sand	16.3	186

### 4.2.3 Comprehensive Determination of $f_{ak}$

Allowable values of bearing capacity from different methods above are listed in table 4.2.3 and recommended allowable values are provided.

Table 4.2.3 Comprehensive Determination of  $f_{ak}$

$f_{ak}$ (kPa) Stratum	Basis		
	physical and mechanical parameters	SPT	Suggested values $f_{ak}$ (kPa)
② clay	<80	< 105	60~80
③ silty clay	<80	139	80~100
③-1 silty fine sand		< 140	80~100
③-2 silty sand		186	140~160

### 4.3 Evaluation on Compressibility of Subsoil

#### 4.3.1 classification of Compressibility

According to the statistical analysis on the physical and mechanical properties of subsoil, the grading of compressibility of each soil layer is classified as shown in table 4.3.1.

Table 4.3.1 Evaluation on Compressibility of Subsoil

Stratum	NaturalVoidRatio $e_0$		Compression Coefficient $a_{1-2}$ (MPa <sup>-1</sup> )		Compression Modulus $E_s$ (MPa)		Grading of Compressibility
	The Range of Values	Mean Value	The Range of Values	Mean Value	The Range of Values	Mean Value	
②clay	1.120~1.486	1.292	0.53~1.02	0.82	2.3~4.1	2.8	High
③silty clay	0.831~1.456	1.202	0.46~1.31	0.91	1.9~4.0	2.7	High
③-1 silty fine sand						5.0	High
③-2 silty sand						9.2	Medium
④silty sand						12.9	Medium
④-1 silty clay	0.699~1.426	0.920	0.15~0.62	0.33	3.7~11.8	5.5	Medium
⑤silt clay with fine sand	0.687~1.087	0.806	0.17~0.45	0.28	4.1~10.3	6.9	Modern
⑥silty fine sand						24.1	Low
⑥-1 silty clay	0.681~0.785	0.743	0.22~0.26	0.24	6.9~8.0	7.4	Medium
⑦silt clay with fine sand	0.589~0.779	0.669	0.11~0.18	0.24	9.3~14.5	11.3	Medium
⑦-1 silty fine sand						25.7	Low
⑧silty fine sand						37.6	Low
⑧-1 silt clay with fine sand						10.9	Medium
⑨silt clay with fine sand	0.538~0.762	0.684	0.11~0.21	0.16	8.1~15.1	10.9	Medium
⑨-1 silty fine sand						38.2	Low

### 4.4 Recommended Geotechnical Parameters of sub-soils

According to statistical data and caculated results above, the recommended geotechnical parameters of subsoils are shown in table 4.4.

Table 4.4 Recommended Geotechnical Parameters of Subsoils

Stratum	Moisture Content	Unit Weigh	Void Ratio	Saturation	Plastic limit	Liquid limit	Plasticity index	liquid index	Direct shear test		Compression Coefficient	Compression Modulus	Vetical Permeability Coefficient	SPT standard value	
	W	$\gamma$	$e_0$	Sr	WL	WP	Ip	I <sub>L</sub>	cohesion	Friction	$a_{1-2}$	E <sub>s</sub>	Kv	Measured	Corrected
	%	kN/m <sup>3</sup>		%	%	%	-	-	c kPa	$\emptyset$ degree	MPa <sup>-1</sup>	MPa	cm/s	N Blow	N Blow
②clay	47.7	17.1	1.370	95.9	47.9	28.2	19.7	0.90	16	7.0	0.85	2.8	8.43E-06	1.8	1.7
③silty clay	47.5	17.0	1.380	93.5	42.8	26.2	16.6	0.95	18	10.3	0.91	2.7	4.50E-06	5.4	4.7
③-1 silty fine sand		17.5							0	25.0		5.0	2.00E-03	7.2	6.2
③-2 silty sand		18.0							0	35.0		9.2	5.00E-03	19.5	16.3
④silty sand		18.5							0	35.0		12.9	5.00E-03	29.1	21.3
④-1 silty clay	36.0	18.4	1.040	93.4	39.5	24.6	14.9	0.67	12	10.9	0.42	5.5	1.76E-06	10.8	8.2
⑤silt clay with fine sand	28.9	19.1	0.841	92.7	35.5	21.5	14.1	0.48	31	14.2	0.28	6.9	4.04E-06	29.2	19.8
⑥silty fine sand		19.5							0	40.0		24.1	2.00E-03	58.8	37.5
⑥-1 silty clay	25.8	19.7	0.743	94.6	35.0	21.9	13.1	0.33	41	17.6	0.24	7.4	9.69E-07	42.5	27.1
⑦silt clay with fine sand	23.9	19.9	0.699	91.2	36.8	22.7	14.1	- 0.06	53	18.3	0.15	11.3	5.61E-07	54.9	31.2
⑦-1 silty fine sand		19.5							0	45.0		25.7	2.00E-03	68.0	41.7
⑧silty fine sand		19.5							0	45.0		37.6	2.00E-03	98.3	54.2
⑧-1 silt clay with fine sand	23.6	19.7	0.710	91.6	38.4	23.1	15.2	- 0.05	52	17.2	0.16	10.9	2.00E-06	28.0	16.2
⑨silt clay with fine sand	23.6	19.7	0.710	91.6	38.4	23.1	15.2	- 0.05	52	17.2	0.16	10.9	1.55E-06	70.8	36.7
⑨-1 silty fine sand		19.5							0	45.0		38.2	2.00E-03	100.0	51.1

## 4.5 Evaluation on Corrosivity of Water

Corrosivity test is performed on 4 groundwater samples collected in boreholes, and the corrosivity of groundwater is valued in accordance with *Code for investigation of geotechnical engineering* (GB50021-2001).

According to Appendix G of *Code for investigation of geotechnical engineering* (GB50021-2001), environment type of the proposed site can be classified as type II. Depth scope of alternation of wetting and drying is suggested to be considered as 3.00 m below the existing ground surface.

### 4.5.1 Corrosivity Appraisal of Groundwater on Concrete Structure

#### (1) Corrosivity Appraisal of Groundwater on Concrete Structure According to Environmental Type

Referring to the site environment type, sulfate content ( $\text{SO}_4^{2-}$ ) and magnesium salt content ( $\text{Mg}^{2+}$ ), corrosivity of groundwater on concrete structure is appraised in accordance with table 12.1.2.1 of *Code for investigation of geotechnical engineering* (GB50021-2001). Values listed in table 12.1.2.1 are applicable to circumstance of alternation of wetting and drying. If without alternation of wetting and drying for II environment type, sulfate content ( $\text{SO}_4^{2-}$ ) listed in the table shall be multiplied by 1.3. Corrosivity appraisal of groundwater on concrete structure is shown in table 4.5.1-1.

Table 4.5.1-1 Corrosivity Appraisal of Groundwater on Concrete Structure According to Environmental Type

Corrosion grade	Corrosive media	(GB50021-2001) Values in code GB50021-2001		Test result (mg/L)	Appraisal result
		Alternation	Non-alternation		
Micro	sulfate content $\text{SO}_4^{2-}$ (mg/L)	< 300	< 390	239~735	weak
Weak		300~1500	390~1950		
Moderate		1500~3000	1950~3900		
Strong		> 3000	> 3900		

#### (2) Corrosivity Appraisal of Groundwater on Concrete Structure According to Penetrability of Stratum

Referring to the penetrability of stratum and PH value of groundwater, corrosivity of groundwater on concrete structure is appraised in accordance with table 12.1.2.2 of *Code for investigation of geotechnical engineering* (GB50021-2001). The soil layers in the

proposed site can be classified as strong permeable layer, and A should be referred. Corrosivity of groundwater on concrete structure is appraised as shown in table4.5.1-2.

Table 4.5.1-2 Corrosivity Appraisal of Groundwater on Concrete Structure According to Penetrability of Stratum

Corrosion grade	Values in code GB50021-2001		Test result pH	Appraisal result
	pH			
Micro	>6.5		7.00~7.40	Micro
Weak	6.5~5.0			
Moderate	5.0~4.0			
Strong	<4.0			

#### 4.5.2 Corrosivity Appraisal of Groundwater on Reinforcing Steel Bar in Reinforced Concrete Structure

Referring to the Cl<sup>-</sup> content in groundwater, corrosivity of groundwater on reinforcing steel bar in reinforced concrete structure is appraised in accordance with table 12.2.4 of *code for investigation of geotechnical engineering* (GB50021-2001). Corrosivity of groundwater on reinforcing steel bar in reinforced concrete structure is appraised as shown in table 4.5.2.

Table4.5.2 Corrosivity Appraisal of Groundwater on Reinforcing Steel Bar in Reinforced Concrete Structure According to Cl<sup>-</sup>Content

Corrosion grade	Cl <sup>-</sup> content in water (mg/L)		Test result Cl <sup>-</sup> (mg/L)	Appraisal result	
	(GB50021-2001)			Alternation	Non-alternation
	Values in code GB50021-2001	Alternation			
Micro	<100	<10000	12916 ~ 48085	Strong	Weak
Weak	100~500	10000 ~ 20000			
Moderate	500~5000	—			
Strong	>5000	—			

According to corrosivity test result of groundwater samples collected in the proposed site, groundwater is weak corrosive on concrete structures; groundwater is Strong corrosive on reinforcing steel bar in concrete in the circumstance of alternation environment and Weak corrosive of non-alternation environment.

## 5 Seismic Effect

### 5.1 wave velocity test

At this stage, three groups of wave velocity tests have been carried out in the proposed plant site area ( Borehole2#,Borehole4#and Borehole7#,Test depth is 20.00m).

According to chapter 4.1.5 in Code for seismic design of buildings (GB50011-2010) (2016 edition),Equivalent shear wave velocity of soil layer shall be calculated according to the following formula:

$$v_{se} = d_0 / t \quad (4.1.5-1)$$

$$t = \sum_{i=1}^n (d_i / v_{si}) \quad (4.1.5-2)$$

where:

$v_{se}$  — Equivalent shear wave velocity of soil (m/s) ;

$d_0$  — Depth of calculation (m) ,minimum value of upper soil depth and 20 m;

$t$  — travel time of shear wave from the ground to the calculated depth;

$d_i$  — Thickness of the  $i$ th soil layer within the calculated depth range (m) ;

$v_{si}$  — Shear wave velocity of the  $i$ -th soil layer in the range of calculated depth (m/s) ;

$n$  — the number of layers of soil within the calculated depth.

The results of equivalent shear wave velocity in this wave velocity test are listed in the table 5.1-1~5.1-3.

Tab 5.1-1 equivalent shear wave velocity test results in borehole 2

test depth H (m)	shear wave velocity(m/s)	travel time/d/v(s)	Shear wave travel time(s)	calculation depth d <sub>0</sub> (m)	equivalent shear wave v <sub>se</sub> (m/s)
1.00	95.7	0.0104	0.1425	20.00	140.3
2.00	101.8	0.0098			
3.00	52.0	0.0192			
4.00	146.4	0.0068			
5.00	136.6	0.0073			
6.00	195.7	0.0051			
7.00	140.5	0.0071			
8.00	117.8	0.0085			
9.00	119.2	0.0084			
10.00	143.9	0.0069			
11.00	112.9	0.0089			
12.00	157.0	0.0064			
13.00	210.8	0.0047			
14.00	252.5	0.0040			
15.00	193.2	0.0052			
16.00	240.6	0.0042			
17.00	274.8	0.0036			
18.00	258.2	0.0039			
19.00	277.0	0.0036			
20.00	259.8	0.0038			

Tab5.1-2 equivalent shear wave velocity test results in borehole 4

test depth H (m)	shear wave velocity(m/s)	travel time/d(v(s))	Shear wave travel time(s)	calculation depth d <sub>0</sub> (m)	equivalent shear wave v <sub>se</sub> (m/s)
1.00	81.0	0.0199	0.1519	20.00	131.7
2.00	86.3	0.0119			
3.00	64.9	0.0080			
4.00	83.2	0.0052			
5.00	65.9	0.0061			
6.00	55.3	0.0064			
7.00	142.9	0.0070			
8.00	118.9	0.0054			
9.00	136.6	0.0046			
10.00	115.9	0.0034			
11.00	129.6	0.0028			
12.00	97.7	0.0037			
13.00	218.6	0.0040			
14.00	111.8	0.0043			
15.00	181.4	0.0037			
16.00	169.6	0.0062			
17.00	234.6	0.0026			
18.00	179.4	0.0037			
19.00	215.9	0.0034			
20.00	215.8	0.0032			

Tab 5.1-3 equivalent shear wave velocity test results in borehole 7

test depth H (m)	shear wave velocity(m/s )	travel time/d(v(s))	Shear wave travel time(s)	calculati on depth d <sub>0</sub> (m)	equivalent shear wave v <sub>sc</sub> (m/s)
1.00	69.4	0.0144	0.1759	20.00	113.7
2.00	71.5	0.0140			
3.00	64.8	0.0154			
4.00	116.0	0.0086			
5.00	87.1	0.0115			
6.00	73.1	0.0137			
7.00	120.0	0.0083			
8.00	133.6	0.0075			
9.00	132.3	0.0076			
10.00	112.1	0.0089			
11.00	139.4	0.0072			
12.00	187.7	0.0053			
13.00	197.2	0.0051			
14.00	166.1	0.0060			
15.00	249.9	0.0040			
16.00	193.6	0.0052			
17.00	238.8	0.0042			
18.00	185.5	0.0054			
19.00	205.0	0.0049			
20.00	216.6	0.0046			

According to the wave velocity test results, the shear wave velocity of the soil layer above 20.00m depth of the proposed site is 52.0-277.0m/s, the average value is 135.3-174.3m/s, and the equivalent shear wave velocity of the soil layer is 113.7~140.3m/s.

## 5.2 Identification of construction site type and Classification of Soil

According to the results of wave velocity test, the site soil type of the proposed plant site is mainly soft soil, medium soft soil, partially medium hard soil, and the construction site type is IV.

### 5.3 Liquefaction

According to Rule 4.3 of Code for seismic design of buildings (GB50011-2010), "For the foundation with saturated sand soil and saturated slit (excluding loess), besides the fortification of 6 degree, it should carry out liquefaction judgment". The seismic precautionary intensity of the proposed site is 8 degree, and saturated sand and saturated slit exist in the site, therefore, liquefaction judgment should be carried out.

#### 5.3.1 Preliminary Judgement

According to Rule 4.3.3 of *Code for seismic design of buildings* (GB50011-2010), saturated sand soil or silt (excluding loess) can be primarily judged as non-liquefaction or ignore the influence of liquefaction when conforming to one of the conditions below:

(1) If the geologic chronology of the saturated sand or silt (excluding loess) is Pleistocene Epoch (Q<sub>3</sub>) of late Quaternary Period, it can be judged as non-liquefaction during 7 and 8 degree

(2) The percent of cosmid (grains with the grain size smaller than 0.005mm) in slit is respectively not smaller than 10 degree, it can be judged as non-liquefaction.

(3) The buildings at natural foundation, if the thickness of the non-liquefaction upper covered soil layer and the depth of ground water level conforming to one of the conditions below, can be judged as ignore the influence of liquefaction:

$$d_u > d_o + d_b - 2 \quad (4.3.3-1)$$

$$d_w > d_o + d_b - 3 \quad (4.3.3-2)$$

$$d_u+d_w>1.5d_0+2d_b-4.5 \quad (4.3.3-3)$$

Where:

$d_w$ —the depth of ground water level (m), which is beneficial to be adopted as the highest annual water level of recent years; It is 0.00m for this project.

$d_u$ —the thickness of non- liquefaction upper covered soil layer (m);

$d_b$ —the embedment depth of foundation (m);

$d_0$ —The characteristic depth of liquid soil (m).

Referring to the geologic chronology of the soil, percent of cosmic and the relationship of the thickness of the non-liquefaction upper covered soil layer and the depth of ground water level, it is indicate that ③-1silty fine sand,③-2silty fine sand,④silty fine sand which is saturated sand and silt exist within depth scope 20m below ground may liquefy, therefore, further liquefaction judgment should be carried out in accordance with the investigation data.

### 5.3.2 Further Liquefaction Judgment

The critical value of standard penetration blow count for liquefaction judgment can be calculated as per formula 4.3.4 of *Code for seismic design of buildings* (GB50011-2010):

$$N_{cr} = N_0\beta[\ln(0.6d_s + 1.5) - 0.1d_w]\sqrt{3/\rho_c} \quad (4.3.4)$$

When standard penetration blow  $N$  (not corrected by rod length) of saturated soil is smaller than the critical value of standard penetration blow count for liquefaction judgment, it should be judged as liquid soil.

Where:

$N$ —standard penetration blow of saturated soil;

$N_{cr}$  — the critical value of standard penetration blow count for liquefaction judgment

$N_0$  — the reference value of standard penetration blow count for liquefaction judgment, and it is 12 for this project.

$d_s$ —depth of standard penetration blow count of saturated soil;  
 $\rho_c$ —percent of cosmic, which, if smaller than 3 or is sand soil, should be 3.

$\beta$ —the adjustment coefficient, which shall be taken as 0.80 for design earthquake Group 1, 0.95 for Group 2 and 1.05 for Group 3.

For the foundation with liquid soil layers, it should calculate the liquidity index of each drilling hole as per the formula below:

$$I_{IE} = \sum_{i=1}^n (1 - N_i / N_{cri}) d_i W_i \quad (4.3.5)$$

Where:

$I_{IE}$ —liquidity index;

$n$ —the total number of standard penetration test points of each drilling hole within judging depth scope;

$N_i, N_{cri}$ —respectively refer to the actual measured value and critical value of standard penetration blow count at  $i$  point;

$d_i$ — the thickness of soil layer represented by  $i$  point;

$W_i$ —layer influence weight function value of unit soil layer thickness of  $i$  soil layer (unit:  $m^{-1}$ ). If the depth judged is 15m and when the depth of points of this layer is no larger than 5m, 10 shall be adopted; when the depth is equal to 15m, zero shall be adopted; when between 5 and 15m, the value shall be calculated through linear interpolation method. While if the depth judged is 20m and when the depth of points of this layer is no larger than 5m, 10 shall be adopted; when the depth is equal to 20m, zero shall be adopted; when between 5 and 20m, the value shall be calculated through linear interpolation method.

Tab 5.3.2

Liquefaction Judgment Results According to SPT

BH No.	Stratum No.	Test depth (m)	Stratum	Liquefaction Judgment					Liquidity index		
				Ground Water level (m)	percent of cosmid (%)	Ni (blow)	Ncri (blow)	Judgment	I <sub>LEi</sub>	I <sub>LE</sub>	Grade
K1	3-1	6.15-6.45	fine sand	0.00	3	9.0	21.0	Yes	8.10	11.79	medium
	3-2	7.65-7.95	fine sand	0.00	3	22.0	23.0	Yes	0.35		
	3-2	9.15-9.45	fine sand	0.00	3	21.0	24.7	Yes	1.59		
	3-2	10.65-10.95	fine sand	0.00	3	23.0	26.2	Yes	1.11		
	3-2	12.15-12.45	fine sand	0.00	3	25.0	27.5	Yes	0.64		
	4	16.65-16.95	fine sand	0.00	3	32.0	30.9	No			
	4	18.15-18.45	fine sand	0.00	3	40.0	31.8	No			
	4	19.65-19.95	fine sand	0.00	3	35.0	32.7	No			
K2	3-2	3.15-3.45	fine sand	0.00	3	17.0	15.7	No		2.64	slight
	3-2	6.15-6.45	fine sand	0.00	3	21.0	21.0	No			
	3-2	7.65-7.95	fine sand	0.00	3	24.0	23.0	No			
	3-2	9.15-9.45	fine sand	0.00	3	22.0	24.7	Yes	1.19		
	4	12.15-12.45	fine sand	0.00	3	25.0	27.5	Yes	0.48		
	4	13.65-13.95	fine sand	0.00	3	23.0	28.7	Yes	0.97		
	4	15.15-15.45	fine sand	0.00	3	33.0	29.8	No			
	4	16.65-16.95	fine sand	0.00	3	36.0	30.9	No			
	4	18.15-18.45	fine sand	0.00	3	45.0	31.8	No			
K3	3-1	3.15-3.45	fine sand	0.00	3	4.0	15.7	Yes	9.32	43.99	severe
	3-1	4.65-4.95	fine sand	0.00	3	7.0	18.6	Yes	9.36		
	3-1	6.15-6.45	fine sand	0.00	3	7.0	21.0	Yes	9.12		
	3-1	7.65-7.95	fine sand	0.00	3	10.0	23.0	Yes	8.78		
	3-2	9.15-9.45	fine sand	0.00	3	15.0	24.7	Yes	2.87		
	3-2	10.65-10.95	fine sand	0.00	3	17.0	26.2	Yes	3.12		
	4	13.65-13.95	fine sand	0.00	3	19.0	28.7	Yes	1.42		
	4	15.15-15.45	fine sand	0.00	3	39.0	29.8	No			

Tab 5.3.2

Liquefaction Judgment Results According to SPT

BH No.	Stratum No.	Test depth (m)	Stratum	Liquefaction Judgment					Liquidity index		
				Ground Water level (m)	percent of cosmid (%)	Ni (blow)	Neri (blow)	Judgment	I <sub>LEi</sub>	I <sub>LE</sub>	Grade
K4	3-2	1.65-1.95	fine sand	0.00	3	16.0	11.9	No		2.60	slight
	3-2	3.15-3.45	fine sand	0.00	3	18.0	15.7	No			
	3-2	4.65-4.95	fine sand	0.00	3	24.0	18.6	No			
	3-2	6.15-6.45	fine sand	0.00	3	18.0	21.0	Yes	2.60		
	4	13.65-13.95	fine sand	0.00	3	31.0	28.7	No			
	4	15.15-15.45	fine sand	0.00	3	33.0	29.8	No			
K5	3-1	3.15-3.45	fine sand	0.00	3	6.0	15.7	Yes	4.33	8.15	medium
	3-2	6.15-6.45	fine sand	0.00	3	23.0	21.0	No			
	3-2	7.65-7.95	fine sand	0.00	3	26.0	23.0	No			
	3-2	9.15-9.45	fine sand	0.00	3	24.0	24.7	Yes	0.29		
	3-2	10.65-10.95	fine sand	0.00	3	22.0	26.2	Yes	0.91		
	4	13.65-13.95	fine sand	0.00	3	19.0	28.7	Yes	2.62		
K6	3-2	7.65-7.95	fine sand	0.00	3	16.0	23.0	Yes	2.54	5.07	slight
	3-2	9.15-9.45	fine sand	0.00	3	17.0	24.7	Yes	2.05		
	4	13.65-13.95	fine sand	0.00	3	27.0	28.7	Yes	0.48		
K7	3-1	4.65-4.95	fine sand	0.00	3	4.0	18.6	Yes	5.44	9.04	medium
	4	9.15-9.45	fine sand	0.00	3	21.0	24.7	Yes	1.76		
	4	10.65-10.95	fine sand	0.00	3	23.0	26.2	Yes	1.11		
	4	12.15-12.45	fine sand	0.00	3	25.0	27.5	Yes	0.73		
K8	3-1	7.65-7.95	fine sand	0.00	3	9.0	23.0	Yes	10.21	21.36	severe
	3-2	9.15-9.45	fine sand	0.00	3	11.0	24.7	Yes	8.18		
	4	13.65-13.95	fine sand	0.00	3	24.0	28.7	Yes	1.37		
	4	15.15-15.45	fine sand	0.00	3	22.0	29.8	Yes	1.23		
	4	16.65-16.95	fine sand	0.00	3	27.0	30.9	Yes	0.37		
K9	3-2	7.65-7.95	fine sand	0.00	3	21.0	23.0	Yes	1.37	4.07	slight
	3-2	9.15-9.45	fine sand	0.00	3	28.0	24.7	No			
	3-2	10.65-10.95	fine sand	0.00	3	16.0	26.2	Yes	2.56		
	4	15.15-15.45	fine sand	0.00	3	38.0	29.8	No			
	4	16.65-16.95	fine sand	0.00	3	32.0	30.9	No			
	4	18.15-18.45	fine sand	0.00	3	28.0	31.8	Yes	0.14		

Tab 5.3.2

Liquefaction Judgment Results According to SPT

BH No.	Stratum No.	Test depth (m)	Stratum	Liquefaction Judgment					Liquidity index		
				Ground Water level (m)	percent of cosmid (%)	实测标贯击数 N (击)	临界标贯击数 N <sub>cr</sub> (击)	液化判别	液化指数 I <sub>LEi</sub>	Ground Water level (m)	percent of cosmid (%)
K11	3-1	6.15-6.45	fine sand	0.00	3	5.0	21.0	Yes	5.07	21.09	severe
	3-1	7.65-7.95	fine sand	0.00	3	7.0	23.0	Yes	8.48		
	3-1	9.15-9.45	fine sand	0.00	3	11.0	24.7	Yes	7.54		
	4	10.65-10.95	fine sand	0.00	3	41.0	26.2	No			
	4	12.15-12.45	fine sand	0.00	3	36.0	27.5	No			
	4	13.65-13.95	fine sand	0.00	3	35.0	28.7	No			
	4	15.15-15.45	fine sand	0.00	3	35.0	29.8	No			
	4	16.65-16.95	fine sand	0.00	3	47.0	30.9	No			
	4	18.15-18.45	fine sand	0.00	3	42.0	31.8	No			
K12	3-1	6.15-6.45	fine sand	0.00	3	9.0	21.0	Yes	11.77	23.17	severe
	3-1	7.65-7.95	fine sand	0.00	3	12.0	23.0	Yes	7.42		
	3-2	9.15-9.45	fine sand	0.00	3	15.0	24.7	Yes	2.87		
	3-2	10.65-10.95	fine sand	0.00	3	23.0	26.2	Yes	1.11		
	3-2	12.15-12.45	fine sand	0.00	3	28.0	27.5	No			
	4	18.15-18.45	fine sand	0.00	3	36.0	31.8	No			
	4	19.65-19.95	fine sand	0.00	3	39.0	32.7	No			
K13	3-1	6.15-6.45	fine sand	0.00	3	5.0	21.0	Yes	10.80	30.64	severe
	3-1	7.65-7.95	fine sand	0.00	3	8.0	23.0	Yes	7.94		
	3-1	9.15-9.45	fine sand	0.00	3	9.0	24.7	Yes	3.74		
	3-1	10.65-10.95	fine sand	0.00	3	11.0	26.2	Yes	6.40		
	4	15.15-15.45	fine sand	0.00	3	20.0	29.8	Yes	1.03		
	4	16.65-16.95	fine sand	0.00	3	27.0	30.9	Yes	0.40		
	4	18.15-18.45	fine sand	0.00	3	24.0	31.8	Yes	0.33		
K14	3-1	3.15-3.45	fine sand	0.00	3	9.0	15.7	Yes	9.18	23.58	severe
	3-1	4.65-4.95	fine sand	0.00	3	11.0	18.6	Yes	5.93		
	3-1	7.65-7.95	fine sand	0.00	3	12.0	23.0	Yes	8.47		

### 5.3.3 Evaluation on Liquefaction of Subsoil

14 boreholes are liquefaction holes, and 4 of them are of slight liquefaction, 3 of them are of moderate liquefaction, 7 of them are of severe liquefaction. The liquefaction Subsoil is ③-1 silty fine sand, ③-2 silty fine sand, ④ silty fine. The liquefaction grade may be slight to severe and the liquefaction depth may be 20.00m.

Because the back filling height is about 9.5m, the liquefaction grade needs to be judged again at next stage.

### **5.4 Classification of Seismic Fortification**

According to table 4.1.1 of Code for seismic design of buildings (GB50011-2001), the proposed site is an unfavorable section can be built on.

## **6 Subgrade Evaluation of LNG Power Plant**

### **6.1 Evaluation of Shallow Foundation**

At present, the original elevation of the plant area is generally lower than 1 m, while the design elevation of the plant area is 10 m, and the thickness of the backfill in the future is about 9.0-10.0m. The strata revealed at this stage are not suitable for use as shallow foundation due to the excessive burial depth.

### **6.2 Pretreatment of backfill**

Because the backfill area is large and the backfill height is about 9.0-10.0m, the backfilling site needs pretreatment. The pretreatment can adopt the methods of overload preloading. The scheme of land formation and ground improvement is: hydraulic filling dredging soil to +10.0 m, keep filling dredging sand to +15.0m(+13.5m), preloading, then unload preloading materials and compact to +10.5 m after consolidation of soft soil, and the ground shall be dynamic compacted, vibration rolled and leveled up to the final land formation elevation +10.0 m.

### **6.3 Pile foundation evaluation of LNG Power Plant**

According to geotechnical engineering conditions in site, combined with local pile foundation construction experience, steel pipe pile, Prestressed High-strength Concrete Pipe Pile (PHC Pile) and cast-in-place pile are all suitable for this project.

#### **6.3.1 Pile Type Selection**

Steel pipe piles or cast-in-place piles can be used for main structures, subsidiary structures with heavy loads or sensitive to foundation settlement, the pile tip is suggested to be placed in ⑥ silty fine sand or the layer below ⑥.

Prestressed High-strength Concrete Pipe Pile (PHC Pile) is recommended for auxiliary building with light load, the pile tip is suggested to be placed in ⑤ silty clay with fine sand or ⑥ silty fine sand layer.

Whether steel pipe pile, Prestressed High-strength Concrete Pipe Pile

or cast-in-place pile are used, in-situ pile loading test should be conducted before construction to verify their applicability. The design parameters of pile foundation should be based on the final conclusions of prototype test report.

### 6.3.2 Ultimate Bearing Capacity of PHC Pile

According to the geotechnical engineering evaluation of each soil layers in the site, ultimate shaft resistance  $q_{sik}$  and ultimate tip Resistance  $q_{pk}$  of pile are suggested in accordance with table 5.3.5-1 and table 5.3.5-2 in technical code for building pile foundations (JGJ94-2008) and shown in table 6.3.2-1.

Tab 6.3.2-1 Ultimate Shaft Resistance  $q_{sik}$  and Ultimate Tip Resistance  $q_{pk}$  of PHC Pile

stratum	$q_{sik}$ (kPa)	$q_{pk}$ (kPa)
②clay	20	
③silty clay	25	
③-1 silty fine sand	22	
③-2 silty sand	48	
④silty sand	66	
④-1 silty clay	55	
⑤silt clay with fine sand	55	3000
⑥silty fine sand	66	4000
Pile length	$16 < L \leq 30m$	

Note:

- ① Indexes in the table were suggested without considering reduction of liquefaction;
- ② Parameters of the pile suggested in the table can only be used for estimating the bearing capacity in preliminary design phase. The design of pile foundation should be based on the *Report on Preliminary Pile Tests*.

### 6.3.3 Ultimate Bearing Capacity of cast in place pile

According to the geotechnical engineering evaluation of each soil layers in the site, ultimate shaft resistance  $q_{sik}$  and ultimate tip Resistance  $q_{pk}$  of pile are suggested in accordance with table 5.3.5-1 and table 5.3.5-2 in technical code for building pile foundations (JGJ94-2008) and shown in table 6.3.3-1.

Tab 6.3.3-2 Ultimate Shaft Resistance  $q_{sik}$  and Ultimate Tip Resistance  $q_{pk}$  of

Pile cast in place

stratum	$q_{sik}$ (kPa)	$q_{pk}$ (kPa)
②clay	20	
③silty clay	20	
③-1 silty fine sand	20	
③-2 silty sand	46	
④silty sand	60	
④-1 silty clay	53	
⑤silt clay with fine sand	53	
⑥silty fine sand	64	1100
⑥-1 silty clay	68	1600
⑦silt clay with fine sand	64	1200
⑦-1 silty fine sand	70	1500
⑧silty fine sand	75	1900
⑧-1 silt clay with fine sand	96	1800
⑨silt clay with fine sand	75	1500
⑨-1 silty fine sand	20	
Pile Length	$l > 30m$	

Note:

- ① Indexes in the table were suggested without considering reduction of liquefaction;
- ② Parameters of the pile suggested in the table can only be used for estimating the bearing capacity in preliminary design phase. The design of pile foundation should be based on the *Report on Preliminary Pile Tests*.

#### 6.3.4 Ultimate Bearing Capacity of steel pipe pile

According to the geotechnical engineering evaluation of each soil layers in the site, ultimate shaft resistance  $q_{sik}$  and ultimate tip Resistance  $q_{pk}$  of pile are suggested in accordance with table 5.3.5-1 and table 5.3.5-2 in technical code for building pile foundations (JGJ94-2008) and shown in table 0-1.

Tab 6.3.4-1 Ultimate Shaft Resistance  $q_{sik}$  and Ultimate Tip Resistance  $q_{pk}$  of steel pipe Pile

stratum	$q_{sik}$ (kPa)	$q_{pk}$ (kPa)
②clay	20	
③silty clay	25	
③-1 silty fine sand	22	
③-2 silty sand	48	
④silty sand	66	
④-1 silty clay	55	
⑤silt clay with fine sand	66	
⑥silty fine sand	55	
⑥-1 silty clay	66	5500
⑦silt clay with fine sand	70	5000
⑦-1 silty fine sand	66	5500
⑧silty fine sand	72	10000
⑧-1 silt clay with fine sand	80	10000
⑨silt clay with fine sand	98	6500
⑨-1 silty fine sand	78	10000
Pile Length	$l > 30m$	

Note:

- ① Indexes in the table were suggested without considering reduction of liquefaction;
- ② Parameters of the pile suggested in the table can only be used for estimating the bearing capacity in preliminary design phase. The design of pile foundation should be based on the *Report on Preliminary Pile Tests*.

## **7 Photovoltaic plant area**

### **7.1 Geomorphology**

The geomorphic type of the Photovoltaic plant area is coastal plain with flat topography and wide terrain. The surface elevation is generally varying from 0.33m to 1.32m.

### **7.2 Sub-soils**

① plain fill: The constituent material mainly comprises grey or brownish grey, very soft, saturated clay and silty clay.

② clay: The constituent material mainly comprises soft, saturated, taupe or yellowish-brown mucky silty clay. This layer is widely distributed in the site.

③ silty clay: The constituent material mainly comprises soft, saturated, grey silty clay. This layer is widely distributed in the site.

③-1 silty fine sand: The constituent material mainly comprises very loose, saturated, grey silty fine sand. This layer is partially distributed in the site.

③-2 silty sand: The constituent material mainly comprises medium dense, saturated, grey silty fine sand. This layer is widely distributed in the site.

④ silty sand: The constituent material mainly comprises medium dense / dense, saturated, grey silty fine sand. This layer is widely distributed in the site.

④-1 silty clay with silty sand interbedded: The constituent material mainly comprises soft, saturated, brownish grey or yellowish brown silty clay with thickness varies from 2mm to 20mm and interbedded with loose dense, saturated, grey or yellowish brown silty sand with thickness varies from 2mm to 18mm. This layer is partially distributed in the site.

⑤ silt clay with fine sand: The constituent material mainly comprises hard, saturated, grey silty clay with fine sand which exist in the form of intercalation. This layer is widely distributed in the site.

### 7.3 Surface Water and it's corrosivity

The site is covered by surface water, the depth of water is about 0.5m to 1.0m.

According to corrosivity test result of surface water samples collected in the proposed site, surface water is strongly corrosive on concrete and bars in reinforced concrete structures.

### 7.4 Subsoils Bearing Capacity of Photovoltaic plant area

Referring to chapter 4.2.3, the allowable Bearing Capacity of each stratum is listed as bellow:

Tab 7.4.1 the allowable Bearing Capacity of each stratum

stratum / $f_{ak}$ (kPa)	Recommended result
②clay	60~80
③silty clay	80~100
③-1silty fine sand	80~100
③-2silty fine sand	140~160

### 7.5 Estimation of stability of Photovoltaic plant area

There are no harmful geological activities in the area of Ash Disposal Pond. It is considered that the proposed ash disposal pond is in a relatively stable area and suitable for construction.

### 7.6 Liquefaction

The liquefaction Subsoil is ③-1silty fine sand, ③-2silty fine sand, ④ silty fine. The liquefaction grade may be slight to severe and the liquefaction depth may be 20.00m.

Because the back filling height is about 5.5m, the liquefaction grade needs to be judged again at next stage.

### 7.7 Subgrade Evaluation of Photovoltaic plant area

#### 7.7.1 Evaluation of Shallow Foundation

At present, the original elevation of the plant area is generally lower

than 1 m, while the design elevation of the plant area is 5.5 m, and the thickness of the backfill in the future is about 4.0-5.0m. The strata revealed at this stage are not suitable for use as shallow foundation due to the excessive burial depth.

### 7.7.2 Pile foundation evaluation of Photovoltaic plant area

According to geotechnical engineering conditions in site, combined with local pile foundation construction experience, Prestressed High-strength Concrete Pipe Pile (PHC Pile) is suitable for Photovoltaic plant area.

Prestressed High-strength Concrete Pipe Pile (PHC Pile) is suggested to be placed in ③-2 silty sand and ④silty sand.

According to the geotechnical engineering evaluation of each soil layers in the site, ultimate shaft resistance  $q_{sik}$  and ultimate tip Resistance  $q_{pk}$  of pile are suggested in accordance with table 5.3.5-1 and table 5.3.5-2 in technical code for building pile foundations (JGJ94-2008) and shown in table 6.3.2-1.

Tab 7.7.2 Ultimate Shaft Resistance  $q_{sik}$  and Ultimate Tip Resistance  $q_{pk}$  of PHC Pile

stratum	$q_{sik}$ (kPa)	$q_{pk}$ (kPa)
②clay	20	
③silty clay	25	
③-1 silty fine sand	22	
③-2 silty sand	48	2100
④silty sand	66	2500
Pile length	$9 < L \leq 16m$	

Note:

- ①Indexes in the table were suggested without considering reduction of liquefaction;
- ② Parameters of the pile suggested in the table can only be used for estimating the bearing capacity in preliminary design phase. The design of pile foundation should be based on the *Report on Preliminary Pile Tests*.

## **8 Foundation pit excavation and dewatering**

### **8.1 Foundation pit excavation**

In the future, the soil layer within the scope of foundation pit excavation is mainly plain fill, and the vertical stability of pit wall is weak. In order to ensure the stability of foundation pit wall during excavation, it is recommended to adopt necessary support measures, and there should not be load on the top of the slope.

### **8.2 Pit dewatering**

The highest level of surface water in site is 2.4m, the highest flood level affected by storm surge is 5.45m, while the design site elevation is 10m, buried depth of most foundation is above the highest water level, so no dewatering is needed. For buildings with deep foundation, excavation must be after dewatering. According to the local construction experience and combined with the site engineering geological conditions, the well-point dewatering scheme can be adopted.

## **9 Adverse geological activities**

According to the field investigation and local engineering experience, the main adverse geological actions of the project site are as follows:

### **(1) Deep settlement caused by large area backfilling**

The entire site will be back-filled dramatically. Under the great load of backfilling, the lower soil layer will be compressed and consolidated, causing long-term and large-scale settlement.

### **(2) Seismic effect**

Liquefaction will arise in ③-1 silty fine sand, ③-2 silty sand and ④ silty fine sand in the site.

Except for 2 adverse geological activities mentioned above, there are no adverse geological processes such as karst, collapse, landslide, collapse, debris flow, large area ground subsidence, goaf subsidence and so on in the proposed site area.

## 10 Conclusions and Recommendations

(1) Based on the comprehensive analysis of regional Active Faults, historical earthquakes, ground motion parameters and adverse geological effects in this area, it is preliminarily considered that the proposed site is in a relatively stable region and suitable for large-scale engineering construction.

(2) The geomorphic type of the photovoltaic plant area is coastal plain with flat topography and wide terrain.

(3) The sub-soil conditions as revealed from investigations so far indicate that the constituent material of sub-soil comprises clay, silty clay, silty fine sand, silt clay with fine sand which belong to quaternary alluvium.

The net allowable bearing capacity of each layer is recommended as following:

②clay:	$f_{ak}=60\sim 80$	kPa
③silty clay:	$f_{ak}=80\sim 100$	kPa
③-1 silty fine sand:	$f_{ak}=80\sim 100$	kPa
③-2 silty fine sand:	$f_{ak}=140\sim 160$	kPa

(4) The site area is shrimp pond. Due to the blocking effect of surrounding dams and the backflow of tidal water during high tide, water accumulated easily in rainy season, which can only be discharged by several small rivers during evaporation and ebb tide. The perennial accumulated water depth is about 0.5-1.0m, the highest accumulated water level is 2.4m, and the highest flood level affected by storm surge is 5.45m.

According to corrosivity test result of surface water samples collected in the proposed site, surface water is strongly corrosivity on concrete and bars in reinforced concrete structures.

(5) Referring to the engineering properties of the soil, the soil category of the proposed site can be classified into soft-weak soil / medium-hard soil preliminarily in accordance with Code for seismic design of buildings (GB50011-2010).

Referring to the thickness of covering layer, engineering properties of subsoil and reference data of other nearby projects, the site category of

the proposed site is suggested to be classified as type IV in accordance with Code for seismic design of buildings (GB50011-2010).

According to the liquefaction calculation results, if the earthquake intensity reached 8 degree, liquefaction may happen in the stratum which are ③-1 silty fine sand, ③-2 silty sand and ④ silty sand. The liquefaction degree may be slight to serious and the liquefaction depth may be 20.00m.

(6) Because the backfill area is large and the backfill height is about 9.5m, the backfilling site needs pretreatment. The pretreatment can adopt the methods of overload preloading. The scheme of land formation and ground improvement is: hydraulic filling dredging soil to +10.0 m, keep filling dredging sand to +15.0m(+13.5m), preloading, then unload preloading materials and compact to +10.5 m after consolidation of soft soil, and the ground shall be dynamic compacted, vibration rolled and leveled up to the final land formation elevation +10.0 m.

(7) According to geotechnical engineering conditions in site, combined with local pile foundation construction experience, steel pipe pile, Prestressed High-strength Concrete Pipe Pile and cast-in-place pile are all suitable for LNG Power Plant. Steel pipe piles or cast-in-place piles can be used for main structures, subsidiary structures with heavy loads or sensitive to foundation settlement. Prestressed High-strength Concrete Pipe Pile is recommended for auxiliary building with light load.

(8) For Photovoltaic plant area, Prestressed High-strength Concrete Pipe Pile (PHC Pile) is suitable.

Whether steel pipe pile, Prestressed High-strength Concrete Pipe Pile or cast-in-place pile are used, in-situ pile loading test should be conducted before construction to verify their applicability. The design parameters of pile foundation should be based on the final conclusions of prototype test report.

(9) According to on-site investigation and desk study, there is no mineral resources, key cultural relics protection units and relics in the proposed site.

(10) According to the meteorological data of Bangladesh, the lowest annual temperature of the project site is higher than 0°C, and there is no frozen soil.