Soil Characteristics of Padma Multipurpose Bridge

Hossain Md. Shahin

Member, Panel of Experts (PoE), Padma Multipurpose Bridge Project
Department of Civil and Environmental Engineering, Islamic University of Technology (IUT), Gazipur, Bangladesh

emails: *shahin@iut-dhaka.edu; shahinnit@gmail.com*

A R T I C L E   I N F O

A B S T R A C T

Padma Multipurpose Bridge is situated on a section of the Padma river where the soil condition is highly heterogeneous and intermixed with different soil layers, for which pile design was the critical part. This paper describes the soil properties and ground profiling of the Padma Multipurpose Bridge. Different types of field and laboratory tests were carried out to obtain accurate soil parameters and ground profiles along the whole alignment of the bridge. Standard Penetration Tests, Cone Penetration Tests, and grain size analyses were conducted for the soil classifications and ground profiling. Flat Plate Dilatometer Tests, High Pressure Dilatometer Tests, and Self-Boring Internal Friction Tests were performed to obtain the stiffness of the different soil layers and shear strength parameters in field conditions. Cross-hole and Seismic Tomography Geophysical tests were executed to obtain seismic waves and dynamic shear modulus of each soil layer. Gel Push Soil Sampling technique was used to collect undisturbed samples of sandy soils to get accurate stress–strength-dilatancy characteristics of sandy soils through different laboratory tests. The soil was classified into three Units and some sub-units based on grain size and SPT-N value. Soil parameters were finalized based on both laboratory and field test results. The pile design was possible for accurate measurement of soil parameters and ground profiling along the alignment of the bridge.

1. INTRODUCTION

Padma Multipurpose Bridge is located across Padma River in the central southern part of Bangladesh, at N23°24', E90°12' following UTM WGS 84 System. It is 6.15 km long, consisting of forty-one 150 m long-span steel trusses supported on 40 piers within the river and two transition piers on each riverbank. The river piers are supported by 3.0 m diameter steel tubular raked piles driven into the ground till the elevations of -98.0 m PWD (Public Works Department reference datum) to -122 m PWD depending on the locations. These large pile lengths resulted from a deep river bed scour from -20 m PWD to -62 m PWD in 100 year return period.

The pile design of the bridge was the critical part due to non-homogeneous intermixed soil stratifications along the length. Furthermore, the existence of stiff cohesive layers in some pile locations starting at an elevation of about -113 m PWD to -128 m PWD with a thickness ranging from about 3 m to more than 30 m made the pile design critical, which delayed the bridge construction by more than a year. To increase end bearing capacity, base grouting was employed below the pile tips. In addition, in some piles,

skin grouting was employed to increase the skin friction capacity of the piles. Due to the existence of fine-grain soil, micro-fine cement was required in the grouting to avoid hydrofracture of the ground. The type of soil and soil characteristics are discussed in this paper based on different in-situ tests and laboratory tests.

2. IN-SITU TESTS

Many types of in-situ tests were carried out during feasibility studies and construction stages to classify the soil correctly and get appropriate soil parameters. The following in-situ tests were carried out along the alignment of the bridge:

- Standard Penetration Test (SPT),
- Cone Penetration Test (CPT),
- Self Boring Pressuremeter Test (SBPT),
- Flat Plate Dilatometer (DMT),
- High Pressure Dilatometer (HPD),
- Self Boring Internal Friction Test (SBIFT),
- Cross-hole Geophysical test,

- Seismic Tomography Geophysical Test, and Some field tests are illustrated in the subsequent sections.

A. Standard Penetration Test (SPT)

The depth of the Standard Penetration Test (SPT) was up to -150 m PWD from the river bed. SPT was executed at 1.5 m and 2 m intervals of depth to determine SPT-N value together with relative density, consistency, and classification of soil at different elevations collecting disturbed soil samples from each interval.

i. Soil Classifications

Soils were classified into Soil Units 1, 2, and 3 following the criteria stated in Table 1 and Table 3 based on the grain size. As described in Table 3, Units 1a and 1b are the fine soils dominated by clay and silt, and Units 2 and 3 are the granular soils dominated by sand. Grain size and identification is defined according to BS 5930. Here, fines mean the grain size is smaller than 0.06 mm, and coarse means the grain size is larger than 2 mm. Unit 1 is divided into 1a and 1b based on the amount of fine materials. Soil with 50% or more fine materials is classified as Unit 1a, and soil with 20% to less than 50% fine material is classified as Unit 1b.

Table 2 shows sub-units classification for Unit 2 and Unit 3 based on the SPT-N value, where Unit 2 is divided into 2a, 2b, 2c, 2d, 2e, and 2f; Unit 3 is divided into 3a, 3b, 3c, 3d, 3e, and 3f. In Table 2, the classification of coarse grained soils from very loose to extremely dense was based on U.S. Navy, 1982, and Lambe and Whitman, 1969. In the sub-units, a & b represent very loose to loose soil, c represents medium dense soil, d depicts dense soil, e represents very dense soil, and f depicts extremely dense soil.

Table 1
Soil Classifications

Geological Unit	Criteria	
	% of basic soil type of the geological unit	% of different particles size of the geological unit
Unit 1a	Fines ≥ 50%	Soil with 50% or more fine materials
Unit 1b	50% > Fines ≥ 20%	Soil with 20% to less than 50% fine materials
Unit 2	Fines < 20% and Coarse < 10%	Soil with less than 20% fine materials and less than 10% coarse materials
Unit 3	Fines < 20% and Coarse ≥ 10%	Soil with less than 20% fine materials and 10% or more coarse materials

Note: Fines = Particle size < 0.06 mm (clay and silt)
Coarse = Particle size > 2 mm (gravel)

Table 2
Sub-units classification for Unit 2 and Unit 3

Geological Sub-unit	Typical SPT N	Classification of soil
a & b	0 < N < 10	Very loose to loose
c	10 < N < 17	Medium dense
d	17 < N < 32	Dense
e	32 < N < 50	Very dense
f	N > 50	Extremely dense

Table 3
Typical descriptions of the soils

Geological Unit	Descriptions of the Materials
Unit 1a and Unit 1b	Fine soil dominant by Clay and Silt. Commonly silty Clay, clayey Silt and slightly sandy Silt, with laminations or trace of Mica. Various colours including brownish Grey, dark grey, and grey
Unit 2	Granular soil dominant by Sand. Commonly silty Sand and slightly silty Sand with laminations or trace of Mica. Mainly grey but randomly brownish grey.
Unit 3	Granular soil dominant by Sand, commonly slightly silty Sand and gravelly Sand with much to some gravel & pebbles, and cobbles in places with laminations or trace of Mica. Mainly grey but randomly brownish grey.

B. Cone Penetration Test (CPT)

Cone Penetration Test was carried out up to a depth of -60 m PWD using 20 Ton hydraulic thrust equipment. The cone was saturated using silicon oil prior to the tests. The CPT was employed to get continuous soil data for profiling the ground. Soil parameters, such as Cone resistance, Sleeve Friction, Dynamic Pore Pressure, and Friction Ratio, are obtained from the CPT. Here, a typical plot of the effective angle of internal friction is presented in Figure 1, where the values were estimated based on Lunne et. al. (1997). The average effective frictional angle, ϕ', was obtained from the CPT was about 35° for soil Unit 2, as seen in the figure.
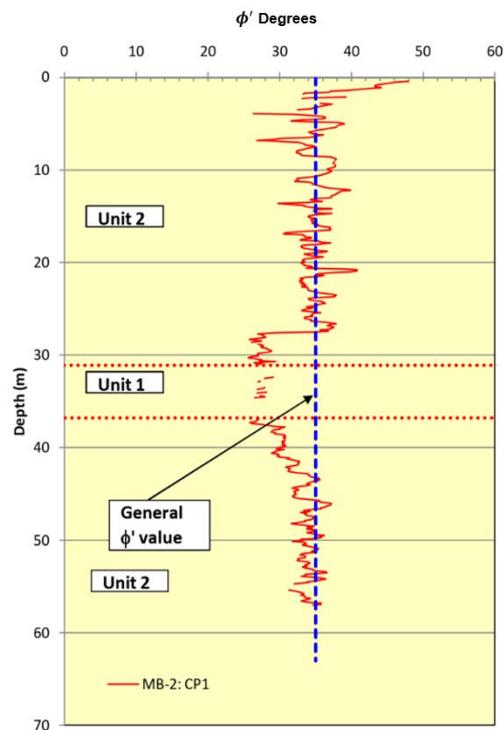
Figure 1: Angle of internal friction along depth (from CPT)

C. High Pressure Dilatometer (HPD) Test

Depth of the High Pressure Dilatometer Test (HPD) was up to -132 m PWD. The HPD measures the volume change under varying applied pressure on the drillhole wall. The pressuremeter membrane was 0.45 m long for a self boring

probe, and 0.6 m long for the HPD. The shear moduli, angle of internal friction, and the coefficient of earth pressure at rest, K_0, of the ground are obtained from the applied pressure and the cavity strain during the tests. Figure 2 shows a typical result of the HPD test in soil Unit 2. It is seen a substantial plastic response of sandy soil in the figure. In the HPD tests, a wide range of variation with K_0 ranging from 0.31 to 0.86 is obtained for soil Unit 2 and Unit 3, having recommended values of 0.40 to 0.45 for these soil types. For soil Unit 1, K_0 is ranging from 0.8 to 1.09 with a recommended value of 0.80. Shear Modulus (G_{ur}) from the unload-reload cycle was obtained, and the recommended value of G_{ur} is 100 MPa for soil Unit 1.
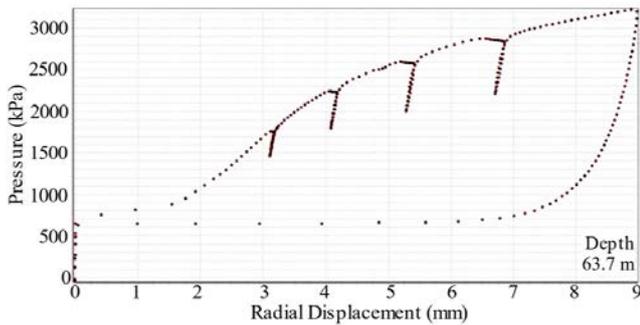
Figure 2: HPD test results in soil Unit 2

D. Self-Boring Internal Friction Test (SBIFT)

Self-Boring Internal Friction Tests (SBIFT) was conducted at three boreholes, GPS1B, GPS2B, and GPS3B, up to a depth of -75 m PWD. GPS1B was located in the middle of the alignment (Easting 220209, Northing 2594268), GPS2B was located near the Janjira area (Easting 220151, Northing 2592792), and GPS3B (Easting 220561, Northing 2598049) was located near the Mawa area. Soil stiffness was measured with the SBIFT. It also measures the internal angle of friction of the soil. The SBIFT probe is inserted into the ground, and different normal pressures and pull-up forces are applied, which generate shear stresses on the SBIFT probe wall. The cohesion (c') and effective angle of internal friction ϕ' were obtained by plotting the σ' and τ plot.

Shear strength parameters from the SBIFT are – (a) for soil Unit 1, c = 5 kPa, ϕ' = 28º, (b) for soil Unit 2, c' = 0 kPa and ϕ' = 34⁰, (c) for soil Unit 2, c' = 0 kPa and ϕ' = 35⁰. Pressuremeter modulus, E_p, corresponding to 1% strain is obtained from the SBIFT listed in Table 4. Here, the pressuremeter modulus increases with the depth having some exceptions due to a dense/stiff soil layer between the very dense soil layers.

E. Cross-hole Geophysical test

The seismic measurements were performed at three sites situated at the two river banks and in the center of the river up to a depth of -150 m PWD. Figure 3 shows typical P-wave and S-wave velocities along the depth. The p-wave velocities are found between 1600 and 1900 m/s. The range of s-wave velocity is between about 100 m/s at the surface and up to 500 m/s at a depth of about -150 m PWD. In general, the velocity, i.e., the soil stiffness, increased with the depth. Since the velocity of the s-wave (V_s) depends on the shear modulus G_0 and the density ρ,

the shear modulus can be calculated from $G_0 = \rho V_s^2$. The density was obtained from the laboratory test as described in the laboratory test section. The dynamic shear modulus ranges from about 30 MPa at the surface to about 500 MPa at greater depths.

Table 4
Pressuremeter modulus with depth at three locations
(GPS1B, GPS2B and GPS3B)

GPS1B		GPS2B		GPS1B	
Depth (m)	E_p (kPa)	Depth (m)	E_p (kPa)	Depth (m)	E_p (kPa)
5.10	23,990	6.50	24,460	8.03	17040
13.40	32,800	19.00	40,460	20.53	35600
20.90	29,200	31.50	45,510	33.03	45820
27.40	35,690	44.00	52,300	45.53	48670
35.90	23,180	56.50	48,340	58.03	62540
43.40	56,970	66.50	28,050	68.03	58640
50.90	54,060	71.50	97,740	73.03	81120
58.90	47,620	76.50	87,340	78.03	76620
65.90	53,080	81.50	106,960	83.03	70530
67.40	65,480	86.50	108,080	88.03	86150
68.40	69,530				
73.40	92,470				
80.90	92,330				
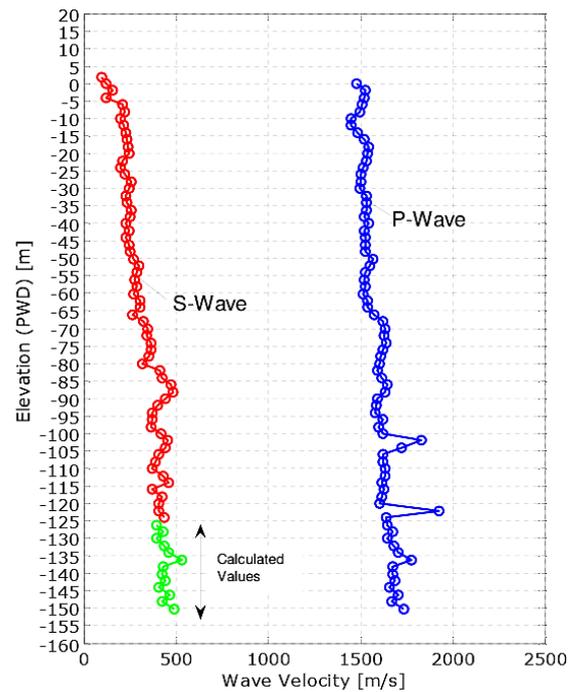
Figure 3: Typical P-wave and S-wave velocities along the depth

4. GROUND PROFILES

Based on the soil classifications, some typical ground profiles at pier locations P6, P16, P26, P33, P36, and P40, are illustrated in igure 4. P6 is located near the Mawa side, and P40 is on the Janjira side. It is seen in the figure that the ground profiles are highly non-homogeneous, having intermixed with different soil Units along the depth. In some locations, stiff cohesive layers (soil Unit 1a and Unit 1b) exist, starting at an elevation of about -113 m PWD to $-128\ m$ PWD having a thickness of about 3 m to more than 30 m.
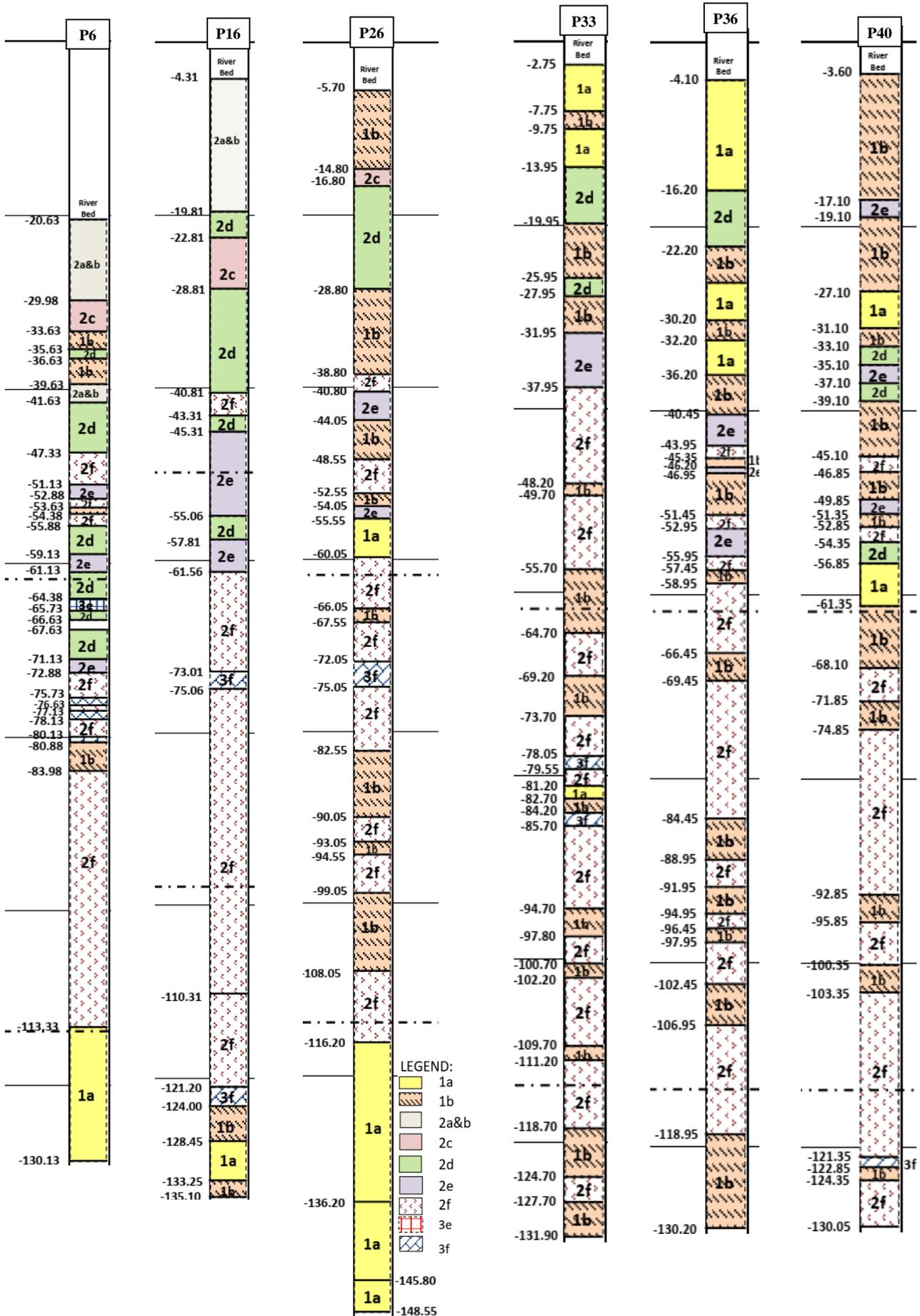
Figure 4: Ground Profiles at Piers 6, 16, 26, 33, 36, and 40

4. LABORATORY TESTING

Various laboratory tests were conducted to get accurate soil properties. Some laboratory test results are described here. Both disturbed and undisturbed samples were collected from the field. Gel Push Sampler, having a length of 1000 mm and diameter of 75 mm, was used to collect good quality undisturbed sand samples. Therefore, it was possible to carry out laboratory tests (direct shear, triaxial, and consolidation tests) for the sandy soil with the original field density. Mazier Sampler, having a length of 1000 mm and diameter of 75 mm, was used to obtain samples of cohesive soils. Split-spoon Sampler was used to collect disturbed soil samples

A. Grain Size Analysis

Both Sieve and Hydrometer tests were conducted to get particle size distribution curves. This paper presents some typical grain size distribution curves for soil Unit 1, Unit 2, and Unit 3 based on the Gel Push soil samples obtained from boreholes GPS1A, GPS2A, and GPS3A. Figure 5 illustrates grain size distribution curves for Unit 1, Unit 2, and Unit 3. In borehole GPS2A, there was no soil of Unit 3. The soils of Unit 1 are dominated by clay and silt, as seen in the figure. The soils of Unit 2 are poorly graded silty sand in all three boreholes. In some locations, fine contents were found up to 19%; in that case, the soils of Unit 2 are well-graded silty sand. Soils of Unit 3 were

Figure 5: Grain size distribution curves

found only in one location of boreholes GPS1A and GPS3A. In GPS1A, coarse particles retain on 2 mm sieve are 25.5%. In GSP3A, coarse particles retain on 2 mm sieve are 10.2%. D_{50} for soils of Unit 1 ranges from 0.047 mm to 0.071 mm, for soils of Unit 2 is 0.135 mm to 0.5185 mm, and for soils of Unit 3 is 0.476 to 0.646 mm.

B. Mica Content

It is considered that the presence of mica particles makes the properties and behavior of the silty sand different from those of non-micaceous silica sands. The mica is likely to increase the porosity, increase compressibility, and decrease the shear strength of the soil. It is also expected to increase the damping ratio of soil due to the void ratio increase. Mica Content Tests were conducted by grain counting following ASTM D-285 guidelines. Table 5 shows mica contents at different depths of boreholes GPS1A, GPS2A, and GPS3A. GPS1A was located in the middle of the alignment (Easting 220209, Northing 2594268), GPS2A was located near the Janjira area (Easting 220151, Northing 2592792), and GPS3A (Easting 220561, Northing 2598049) was located near the Mawa area. It is seen that the mica content is higher at a shallower depth. Mica contents ranging from 17% to 44% were found in soils of Unit 1; for Unit 2, it was 0% to 17%; and for Unit 3, it was 0% to 9%.

Table 5
Mica Content in percentage at different locations and depths

GPS1A		GPS2A		GPS3A	
Depth (m)	Mica Content (%)	Depth (m)	Mica Content (%)	Depth (m)	Mica Content (%)
4.49 - 5.49	44	5.88 - 9.38	17	7.53 - 9.53	26
20.30 - 21.30	6	18.38 - 20.48	9	20.03 - 24.03	9
26.80 - 27.80	6	30.88 - 32.88	5	32.53 - 34.53	5
35.30 - 36.30	9	43.38 - 45.38	11	45.03 - 47.03	8
42.80 - 43.80	5	55.88 - 57.88	9	57.53 - 60.13	7
50.30 - 51.30	5	65.88 - 67.88	14	67.53 - 69.53	10
53.30 - 55.30	5	70.88 - 72.88	5	72.53 - 74.53	5
57.80 - 60.70	6	75.88 - 77.88	4	77.53 - 79.53	3
65.30 - 68.70	6	80.88 - 82.88	3	82.53 - 84.53	2
72.80 - 74.80	9	85.88 - 87.88	7	87.53 - 89.53	3
80.30 - 84.40	5			97.53 - 99.53	4

C. Soil Identification Tests

Table 6, Table **7**, and Table **8** represent the Geological Unit, natural water content, degree of saturation, bulk unit weight, degree of saturation, and field void ratio at boreholes GPS1A, GPS2A, and GPS3A. It is seen in the tables natural water content, w_n, decreases with depth, hence the degree of saturation, S_r, decreases. The unit weights (γ_t) of Unit 2f and Unit 3f are the highest, with some exceptions where the void ratios (e) of these soils are relatively higher.

Table 6
Some basic parameters of soils in GPS1A

Depth (m)	Geological Unit	w_n %	S_r %	γ_t (kN/m³)	e
4.49 - 5.49	Unit 1a	32.10	99.6	18.77	0.876
11.80 - 13.80	Unit 2d	27.90	96.9	19.07	0.778
20.30 - 21.30	Unit 2d	25.20	81.8	18.15	0.835
26.80 - 27.80	Unit 2d	28.30	95.9	18.95	0.800
35.30 - 36.30	Unit 2d	32.50	97.7	18.55	0.903
42.80 - 43.80	Unit 2d	24.60	84.7	18.60	0.793
50.30 - 51.30	Unit 2e	27.40	83.5	17.94	0.892
53.30 - 55.30	Unit 2e	28.20	91.0	18.49	0.836
57.80 - 60.70	Unit 2d	17.90	72.5	18.70	0.665
65.30 - 68.70	Unit 2f	18.70	74.0	18.69	0.682
72.80 - 74.80	Unit 2f	18.40	84.1	19.61	0.586
80.30 - 84.40	Unit 3f	13.60	66.1	19.43	0.560

Table 7
Some basic parameters of soils in GPS2A

Depth (m)	Geological Unit	w_n %	S_r %	γ_t (kN/m³)	e
5.88 - 9.38	Unit 1a	29.4	98.0	18.97	0.813
18.38 - 20.48	Unit 2c	25.3	85.1	18.47	0.808
30.88 - 32.88	Unit 2c	29.7	90.4	18.22	0.888
43.38 - 45.38	Unit 2d	29.4	83.7	17.69	0.958
55.88 - 57.88	Unit 2d	15.8	52.7	16.97	0.812
65.88 - 67.88	Unit 2d	28.1	97.6	19.08	0.777
70.88 - 72.88	Unit 2f	15.5	62.4	18.41	0.677
75.88 - 77.88	Unit 2f	10.8	48.8	18.39	0.598
80.88 - 82.88	Unit 2f	10.7	48.3	18.35	0.599
85.88 - 87.88	Unit 2f	17.4	64.5	18.50	0.765

Table 8
Some basic parameters of soils in GPS3A

Depth (m)	Geological Unit	w_n %	S_r %	γ_t (kN/m³)	e
7.53 - 9.53	Unit 1a	30.0	78.6	17.07	1.044
20.03 - 24.03	Unit 2c	16.7	50.2	16.26	0.897
32.53 - 34.53	Unit 2d	25.3	83.7	18.27	0.815
45.03 - 47.03	Unit 2e	16.2	52.9	16.91	0.834
57.53 - 60.13	Unit 2f	27.6	89.8	18.52	0.835
67.53 - 69.53	Unit 2d	14.5	50.2	17.10	0.783
72.53 - 74.53	Unit 2e	21.0	76.6	18.51	0.748
77.53 - 79.53	Unit 2e	11.2	44.1	17.46	0.685
82.53 - 84.53	Unit 2f	11.4	47.1	17.75	0.648
87.53 - 89.53	Unit 3f	13.5	56.8	18.28	0.639
97.53 - 99.53	Unit 2f	21.9	81.1	18.64	0.727

D. Compression Behavior

Consolidation tests were carried out for different soils of undisturbed samples of Unit 1, Unit 2, and Unit 3. In this paper, some typical results of the consolidation tests are presented. Here, undisturbed samples of Unit 1a were obtained from the boreholes at Piers 2, 8, and 15; for Unit 1b, the sample was obtained from the borehole at Pier 2. Here, undisturbed samples were obtained for sandy soil by Gel Push tests at boreholes GPS1A, GPS2A, and GPS3A. Figure 6 illustrates the relationships of void ratio, e, and consolidation pressure for soils of Unit 1, Unit 2, and Unit 3. The compression index and swelling index are listed in Table 9. As usual, the compression index of Unit 1a is higher than those of the other soil Units. The compression index of Unit 1a is 0.339 at a depth of 126 m. which is located at Pier 2. The value was found as large as 0.432 at a depth of 130 m in another location, which restricted
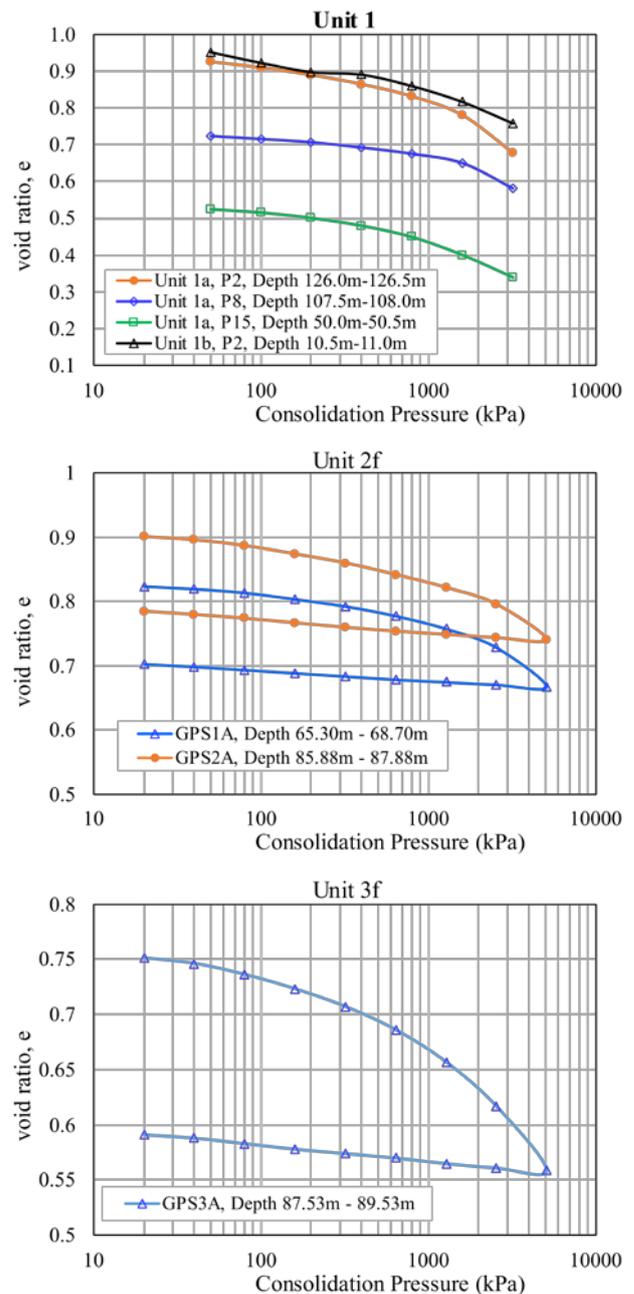
Figure 6: e-log p cuves

placing the pile tip at a location for which the load of the bridge reaches the layer of soil Unit 1. Consequently, the pile length was shortened in 22 piers inserting one more pile in each pier, and in some piles, skin grouting was applied to enhance the bearing capacity of the pile group.

Table 9
Compression index and swelling index of soils

Geological Unit	Location	Sample Depth (m)	Cc	Cs
Uni 1a	Pier 2	126.0-126.5	0.339	-
	Pier 8	107.5-108.0	0.226	-
	Pier 15	50.0-50.5	0.203	-
Unit 1b	Pier 2	10.5-11.0	0.193	-
Unit 2f	GPS1A	65.3 - 68.7	0.203	0.013
	GPS2A	85.88 - 87.88	0.183	0.016
Unit 3f	GPS3A	87.53 - 89.53	0.193	0.012

E. *Cohesion and Angle of Internal Friction*

Gel Push samples were used to obtain effective cohesions (c') and angle of internal friction angles (ϕ'). Figure 7 illustrates the relationships between deviatoric stress (σ'_1-σ'_3)/2 and mean effective stress (σ'_1+σ'_3)/2 for three soil units, Unit 1, Unit 2, and Unit 3. The values of c' and ϕ' are estimated based on the effective normal stress (σ') and shear stress (τ) plot.

The effective cohesion and internal friction for soil Unit 1 were obtained from Isotropically Consolidated Undrained Compression (CUC) triaxial tests.

For soil Unit 2, Isotropically Consolidated Drained Compression (CDC), Isotropically Consolidated Undrained Compression (CUC), K_0 Consolidated Drained Compression (CK$_0$DC), K_0 Consolidated Undrained Compression (CK$_0$UC), and Isotropically Consolidated Rebound Drained Compression (CRDC) triaxial tests were carried out to obtain the effective cohesion and angle of internal friction as seen in the legend of the figure for Unit 2.

The parameters (c' and ϕ') for Unit 3 are obtained from CDC, CUC, and CK$_0$UC triaxial tests. From the trend lines of Figure 7, it is found that the effective cohesion for soil Unit 1 is 5.0 kPa; and the effective angle of internal frictions for this Unit is 32°, for soil Unit 2 is 34°, and for soil Unit 3 is 35°.

Table 10 shows the values of the effective cohesions and internal frictions obtained from different field and laboratory tests. These values were used in the pile design for piers 1 to 18 and 20 to 24. As seen in the table, soil shear strength parameters (c' and ϕ') values vary field soil conditions.

Table 11 shows the effective internal frictions that were used in the pile design for piers 19 and 25 to 42. For these piers, both effective cohesions and friction angles were not

considered for soil Units 1a and 1b. Besides, if soil Units 2a and 2b exist in a shallower depth from the river bed, both effective cohesions and friction angles were not considered in these piers. However, the effective friction angle for this soil type (Units 2a and 2b) was considered 28° for a deeper depth.
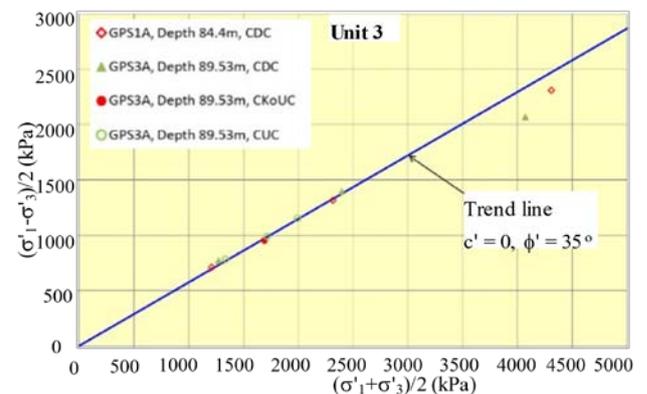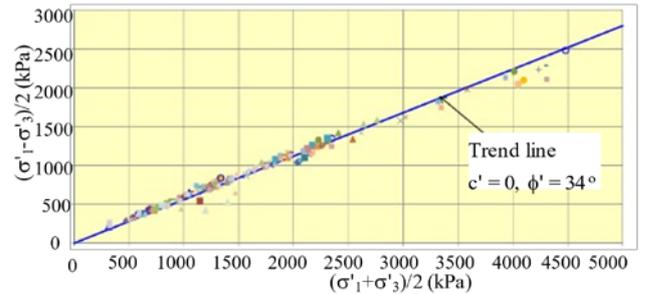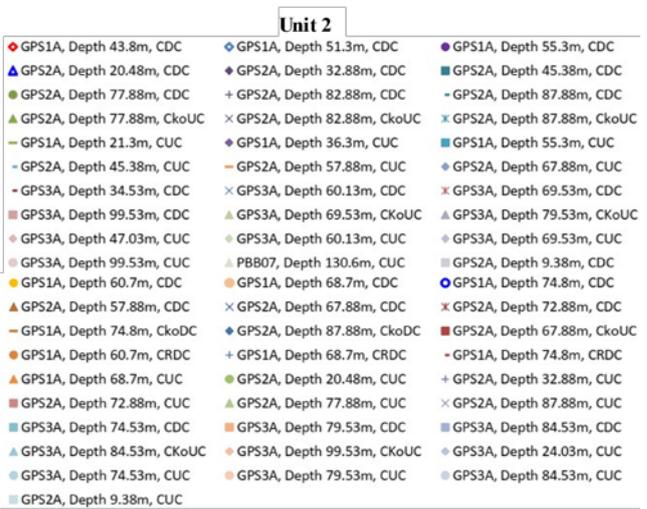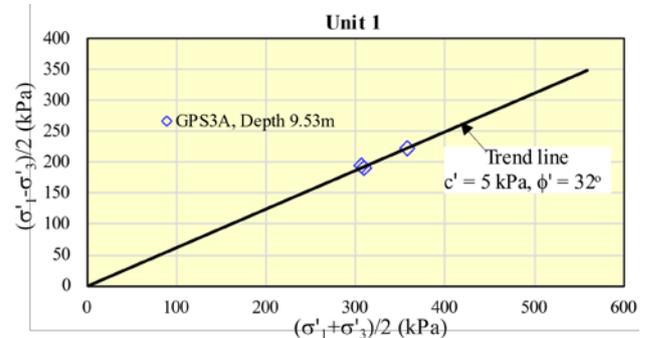
Figure 7: Relationships between mean deviatoric stress and mean effective stress

Table 10
Effective cohesion and angle of internal friction at
piers 1 to 18 and 20 to 24

Geological Unit	c' (kPa)	ϕ' (degree)
Unit 1a	15 to 33	3 to 20
Unit 1b	3 to 20	17 to 29
Unit 2a, 2b	-	24 to 25
Unit 2c	-	24 to 30
Unit 2d	-	26 to 32
Unit 2e	-	28 to 33
Unit 2f	-	28 to 34
Unit 3e	-	31 to 33
Unit 3f	-	31 to 34

Table 11
Effective cohesions and angle of internal frictions at
piers 19 and 25 to 42

Geological Unit	c' (kPa)	ϕ' (degree)
Unit 1a, 1b	-	-
Unit 2a, 2b	Shallower depth	-
	else	28
Unit 2c	-	30
Unit 2d	-	32
Unit 2e	-	34
Unit 2f	-	36
Unit 3f	-	38

5. SITE SEISMICITY

The maximum Peak Ground Acceleration (PGA) was used based on the report of the Bureau of Research, Testing & Consultation (BRTC), BUET, 2009. The maximum PGAs were derived using the attenuation relationship (Abrahamson & Silva, 2008). The site specific ground accelerations at an elevation of -120 m PWD and river bed are listed in Table 12. Here, bedrock is considered at an elevation of -120 m PWD.

Table 12
PGA values for various Return Periods

Return Period (Years)	Horizontal PGA at $-120\ m$ PWD (g)	Horizontal PGA at river bed (g)
2	0.004	0.008
10	0.011	0.022
50	0.032	0.064
100	0.051	0.102
200	0.080	0.160
475	0.141	0.282
1000	0.230	0.460

Ground response analysis was performed with SHAKE software. As seen in Table 12, the general design

amplification factor for the site soil conditions is about 2.0. The response analysis was based on the assumption that bedrock is at an elevation of -120 m PWD.

Three strong ground motions (No.1, No.2, No.3) were adopted in the dynamic analysis of the bridge those proposed by the Japanese Codes, "Design Specification for Highway Bridge Part V: Seismic Design", published by the Japan Road Association. In addition, two strong ground motions (No. 4 and No. 5) were proposed by the BRTC, BUET, 2009. The strong ground motions are depicted in Figure 8.
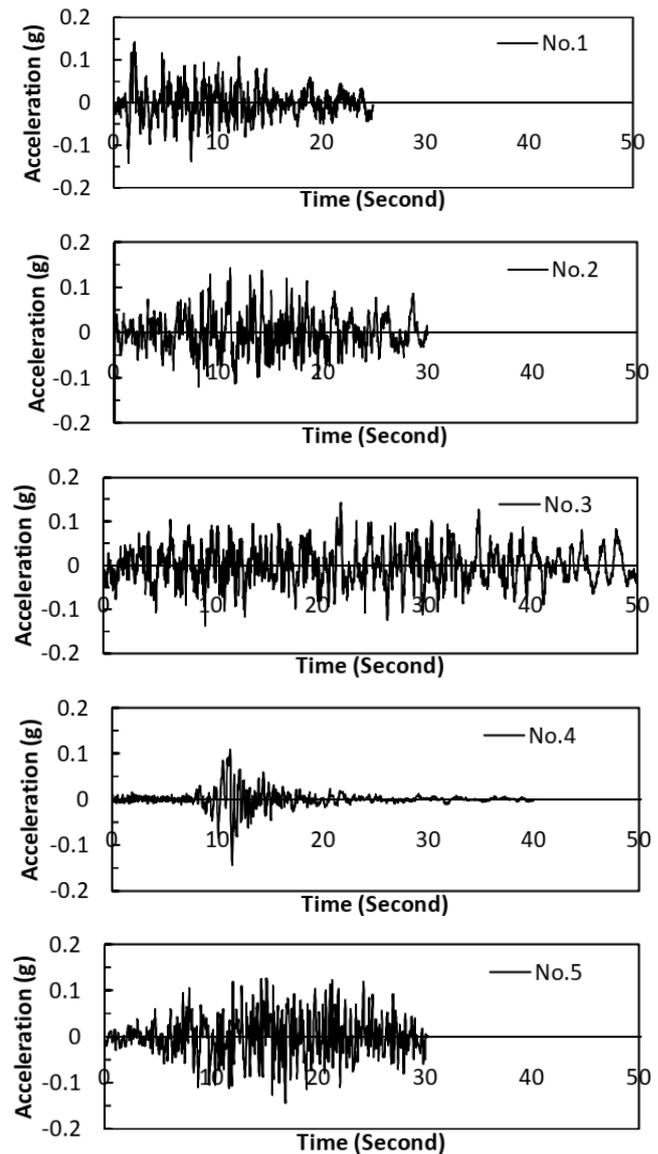
Figure 8: The Strong Ground Motions used in SHAKE Analyses

Figure 9 illustrates the ground response spectra those were obtained using the strong ground motions (No.1 to No.5) and the ground conditions at the site. The figure also shows a comparison with the response spectra recommended by the AASHTO (2009), Guide Specifications for LRFD Seismic Bridge Design – Section 3, corresponding to soil types II and III for to site coefficients S=1.2 and S=1.5, respectively.
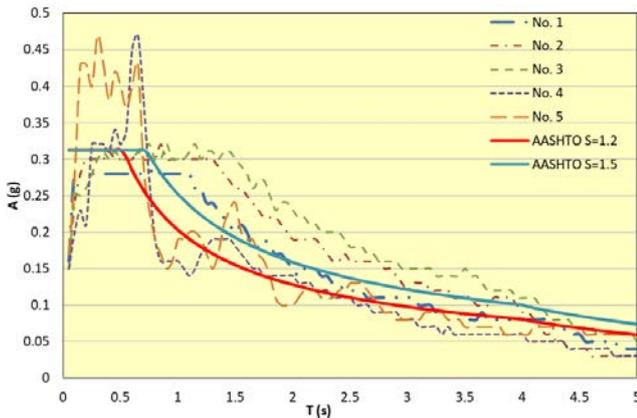
Figure 9: Ground Response Spectra

A. Seismic Hazard Levels and Performance Criteria

In the design, following two levels of seismic hazards and corresponding performance criteria were considered.

i. Level 1 – Operating Level Earthquake (OLE)

Seismic Hazard: the OLE events have a 65% probability of being exceeded in the design life of 100 years or a return period of 100 years. The OLE events have a PGA of 0.052g in the very dense sand at an elevation of -120 m PWD.

Performance Criteria: fully functional

The bridge shall survive the OLE events with no damage, and full service is available to all vehicles immediately after the OLE events.

ii. Level 2 – Contingency Level Earthquake(CLE)

Seismic Hazard: The CLE events have a 20% probability of being exceeded in the design life of 100 years or a return period of 475 years. The CLE events have a PGA of 0.144g in the very dense sand at an elevation of -120 m PWD.

Performance Criteria: Life Safety

The bridge shall survive the CLE events with moderate, readily detectable, and repairable damage. There is no collapse and no threat to life. Damage can be repaired to restore the full operational functioning of the structure without demolition and replacement of components.

9. CONCLUSIONS

From different in-situ tests, accurate ground profiling was done along the longitudinal section of the Padma Multipurpose Bridge. Also, soil parameters were obtained with a high degree of accuracy in the bridge area from the different types of laboratory and field tests. The soil parameters were verified by different test methods. With the proper soil parameters and ground profiling, it was possible to design the Padma Multipurpose Bridge with

safety. It was also possible to place the pile tips at safer ground locations for all piers of the bridge for accurate ground profiling. Therefore, it can be said that proper estimation of soil parameters and ground profiling is required to construct any important structure.

REFERENCES

Abrahamson, N. & Silva, W., (2008), Summary of the Abrahamson & Silva NGA Ground-Motion Relations. Earthquake Spectra, Vol. 24, No. 1, pp. 67-97.

Bureau of Research, Testing & Consultation (BRTC), BUET, (2009), Report on Detailed Design of Padma Multipurpose Bridge Project: Additional Study – Updating Site Seismic Assessment.

China Railway Major Bridge Engineering Group Co. Ltd. (MBEC), (2015), MBEC-Main Bridge Borehole GI Report, Padma Multipurpose Bridge Project.

EGS (Asia) Ltd., (2010), Padma Multi-Purpose Bridge Project, Marine and Land Geophysical Surveys: Final Report.

Foundation Consultants Ltd., (2009), Report on Geotechnical Investigation for Proposed Construction of Padma Multipurpose Bridge Project.

Foundation Consultants Ltd., (2009), Report on Cone Penetration Test for Proposed Construction of Padma Multipurpose Bridge Project.

Lambe, T. W., & Whitman, R. V., (1969). Soil Mechanics, Wiley, New York, NY, pp 553.

Lunne, T., Robertson, P.K. & Powell, J.J.M., (1997), Cone Penetration Testing in Geotechnical Practice. Blackie Academic/Chapman and Hall, E&FN Spon, London.

MAUNSELL – AECOM, (2011a), Marine ground investigation field works, Padma Multipurpose Bridge Project: Final Report.

MAUNSELL – AECOM (2011b), Padma Multipurpose Bridge Project, Detail Design Report – Main Bridge River Span.

Nippon Koei Co., LTD. & Construction Project Consultants, (2005), The Feasibility Study of Padma Bridge in The People's republic of Bangladesh: Final Report.

U.S. Department of the Navy, (1982). Foundations and Earth Structures. NAVFAC DM-7.2, Naval Facilities Engineering Command, Alexandria, VA.