Soil Characteristics of Padma Multipurpose Bridge

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ARTICLE INFO

Article History:
Received: 31st October 2022
Revised: 15th November 2022
Accepted: 26th November 2022
Published: 30th November 2022

Keywords:
Soil Characteristics
Padma Multipurpose Bridge
Gel Push Sampling

ABSTRACT

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 N23º24', E90º12' 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,
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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.

1. 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

| Geological Unit | % 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

| 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        |

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, $\phi'$, was obtained from the CPT was about 35° for soil Unit 2, as seen in the figure.

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
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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 ($\phi'$) were obtained by plotting the $\sigma'$ and $\tau$ plot.

Shear strength parameters from the SBIFT are – (a) for soil Unit 1, $c = 5$ kPa, $\phi' = 28^\circ$, (b) for soil Unit 2, $c' = 0$ kPa and $\phi' = 34^\circ$, (c) for soil Unit 2, $c' = 0$ kPa and $\phi' = 35^\circ$. 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 $\rho$,

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 | 48,340 | 30.03 | 62540 |
| 27.40 | 35,690 | 44.00 | 52,300 | 45.53 | 48670 |
| 35.90 | 23,180 | 56.50 | 48,340 | 50.03 | 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 figure 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 collect 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, while the soils of Unit 3 are well-graded silty sand. Soils of Unit 3 were found only in one location of boreholes GPS1A and GPS3A. In GPS1A, coarse particles retain on 2 mm sieve are 25.5%. In GPS3A, coarse particles retain on 2 mm sieve are 10.2%. D50 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-micaeous 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. It is seen in the middle of the alignment (Eastin 220209, Northing 2594268), GPS2A was located near the Janjira area (Eastin 220151, Northing 2592792), and GPS3A (Eastin 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%.

| Depth (m) | GPS1A | GPS2A | GPS3A |
|----------|-------|-------|-------|
| 4.49-5.49 | 44    | 5.88-9.38 | 17    | 7.53-9.53 | 26  |
| 20.39-21.30 | 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   | 70.08-72.88 | 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     |            |     |

Figure 5: Grain size distribution curves

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, decreases with depth, hence the degree of saturation, Sr, decreases. The unit weights (γ) 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 | w₀ % | S₀ % | \(Y'_c\) (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 |

Table 7
Some basic parameters of soils in GPS2A

| Depth (m) | Geological Unit | w₀ % | S₀ % | \(Y'_c\) (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 - 57.98 | 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 | w₀ % | S₀ % | \(Y'_c\) (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...
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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 cohesion (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), K0 Consolidated Drained Compression (CK0DC), K0 Consolidated Undrained Compression (CK0UC), 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 CK0UC 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 friction angles 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     | Shallow depth | - |
| Unit 2c         | -        | 28          |
| 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 site coefficients S=1.2 and S=1.5, respectively.
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.

ACKNOWLEDGEMENTS

The Author is grateful to Bangladesh Bridge Authority (BBA) for permission to publish this paper. All the data used in this paper are from the consulting and construction companies – MAUNSELL-AECOM, Nippon Koei Co., Ltd., Construction Project Consultants, Bureau of Research, Testing & Consultation (BRTC), BUET, Kisorajiban Consultants Co. Ltd., Japan, Cambridge Insitu Ltd., UK, Foundation Consultants Ltd., and China Railway Major Bridge Engineering Group Co. Ltd. (MBEC), EGS (Asia) Ltd. The Author acknowledges the contribution of the above mentioned consulting and construction companies.

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