Dynamic load test of full-scale pile for the construction and rehabilitation of bridges

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Abstract. Seven dynamic load tests were performed on six large diameter piles, installed near the Gumti river Bangladesh. The diameter of five cast in situ piles was 1.5 m, and that of the steel pile was 1 m. The pile lengths varied from 45 m to 71.8 m. The distribution of pile bearing resistance under static load were evaluated using the program DLTWAVE by signal matching. In addition, standard penetration tests were conducted in the vicinity of all the six piles. The results of dynamic load test were calibrated by comparing the static capacity obtained from static and dynamic load tests, conducted on two identical cast in place piles. Therefore, the skin frictional resistance and pile tip resistance are calculated using the conventional formulae of static analysis utilizing the developed correlation for the clay layers. The consistency of clay soil is observed to play an important role in the development of shaft resistance. For very stiff to hard clay, DLTWAVE indicates the development of shaft resistance smaller than that obtained from the formula (static analysis). Greater skin frictional resistance was obtained in DLT re-drive, as compared to that during initial drive.

1. Introduction

Determination of pile capacity from load test is recognized important, as the soil condition surrounding the pile may be significantly modified during the construction/installation [1]. At present, dynamic load test (DLT) on pile is practiced in more than 40 countries to evaluate the static capacity and structural integrity of pile using the measurement of both force and velocity. The test method of DLT has been standardized by different relevant codes, such as ASTM 4945. In DLT, dynamic impact is applied, signals are captured, and then static capacity, skin frictional resistance and tip resistance of pile are obtained after post-processing of signals. In addition to the assessment of pile’s static capacity, DLT provides information about pile driveability [2], pile integrity and changes in cross-section (if any). DLT is significantly faster and economical than static test, as at least two bored piles and even more driven piles can be tested in one day with proper arrangements of DLT. In determining pile’s capacity, DLT has several advantages over static load test in the following cases:

- Large diameter piles
- Offshore and onshore piling
- If there is space constraint to conduct static load test, especially for jetties, flyover, bridge and other construction projects in crowded area
Moreover, the use of the formulae of static analysis is limited in bridge construction projects, as it requires large number of borelog so that the variability in soil deposition at the river-bed can be addressed. In order to measure skin frictional resistance and pile tip resistance from static load test, pile needs to be instrumented with strain gauges at different sections of the pile, while in DLT these can be measured without any such instrumentation. Moreover, time after EOD (end of driving) influences the measurements of pile capacity and skin frictional and tip resistances [5]. DLT makes it possible to investigate “set up’ time that has a significant effect on the pile capacity measurement, as it can be executed easily [6, 7]. Pile resistances can be reasonably obtained from DLT in 50% shorter period after EOD, as compared to static load test [5]. Ultimate pile capacity of a square pile (30 cm x 30 cm) from a DLT was about 75% of that obtained from static load tests [8]. DLT is found as a useful method for predicting pile capacity for helical pile in cohesive soil [9]. DLT predicts pile resistances reasonably well but correlations and further analysis based on the DLT results. In recent years researchers paid attention in numerical modelling of pile for predicting pile resistances [10]. The objectives of the present study are to evaluate DLT in the determination of pile capacity, shaft resistance developed on the pile surface and pile tip resistance, when the pile is embedded in layered soils. A program DLTWAVE of Profound BV is used to obtain the static load settlement curve of the pile by signal matching.

2. Experimental program
In this study, seven dynamic load tests on six single vertical driven piles, along with DLTWAVE signal matching, were conducted. DLT on steel pile was repeated by re-driving for comparing the pile soil interaction during driving and re-driving. The sub-surface soil exploration, near each test pile location, was carried out by conducting standard penetration tests. In addition, undisturbed specimens were collected from cohesive layers to determine UU triaxial tests. Table 1 presents the detail of the experimental program carried out in-situ and in the laboratory as well.

| Table 1. Experimental Program.       |
|--------------------------------------|
| Name of the test | No. of tests | Outcome                                      |
| Field Tests     |             |                                              |
| Dynamic load test | 7           | skin frictional resistance, end-bearing       |
|                 |             | resistance and ultimate pile capacity       |
| Standard penetration test | 6           | borelog                                     |
| Laboratory Tests |             |                                              |
| UU Triaxial     | 5           | undrained shear strength                     |
| Direct Shear Test | 1           | angle of internal friction                   |

2.1. Test site
The test site was located near the Gumti river in Bangladesh. The Gumti river is a tributary of the Ganges. During monsoon, the hilly river has strong and rapid current, and its breadth is about 100 m (on an average). The Gumti river causes floods as a common and regular phenomena. Bangladesh water development board (BWDB) took several measures to train the river. At present, there is a bridge across the Gumti river and the construction of the second is in progress. Moreover, the rehabilitation projects of the existing bridges across other rivers with strong current are currently ongoing. Therefore, the site has been chosen for the investigation of pile resistance by employing advanced technology of dynamic load test.
2.2. Soil condition

During the sub-surface exploration, standard penetration tests were conducted at 1.5 m interval up to 80 m depth below the ground level. The borelogs, as given in table 2 and table 3, demonstrate that different layers of SP-SC, SC, ML and CL soil layers exist at the test sites, according to unified soil classification system. SC and CL types of soils are most commonly encountered at the test site. Deep stratum with SPT N of 50 was found at 65 m or greater depths. According to SPT resistances, CL soil layers at the depth up to 45 m were found stiff to very stiff, while those at greater depths were very stiff to hard. The consistency of SC soil layers was identified as medium dense type.

In the laboratory, UU triaxial tests were carried out on undisturbed cohesive specimens, and the undisturbed shear strength varies from 45 to 129 kPa. The water level was found to vary from 0-5 m above the river bed, as of June 2017. The specimen collected from SC soil layer was tested in direct shear test apparatus, and the angle of internal friction was found between 32° and 37°. During SPT, N_{60} of SC soil layer gave a value ranging from 12 to 14.

2.3. Pile

In this study, dynamic load tests were conducted on 5 concrete piles and 1 steel pile. The dimensions and types of piles are given in table 4. The locations of test piles are also given in table 4.

| Depth  | Soil Type | N_{avg} | Depth  | Soil Type | N_{avg} | Depth  | Soil Type | N_{avg} |
|--------|-----------|---------|--------|-----------|---------|--------|-----------|---------|
| 0 - 8.5| SC        | 10      | 0 - 5  | ML        | 2       | 0 - 4  | SP-SM     | 16      |
| 8.5 - 15.5| SC  | 30      | 5 - 10 | SM        | 5       | 4.1 - 11.1 | ML | 2      |
| 15.5 - 17.5| CL  | 20      | 10 - 22| SM        | 17      | 11.1 - 14.1 | SC | 6      |
| 17.5 - 22.5| SC  | 21      | 22 - 26| SM        | 32      | 14.1 - 16.1 | CL | 19     |
| 22.5 - 27.5| CL  | 18      | 26 - 29| CL        | 15      | 16.1 - 30.1 | SC | 21     |
| 27.5 - 30 | CL   | 17      | 29 - 31| SM        | 24      | 30.1 - 40.1 | CL | 23     |
| 30 - 31 | SC      | 17      | 31 - 44| CL        | 25      | 40.1 - 49.1 | SC | 27     |
| 31 - 44 | CL      | 22      | 44 - 56.5| CL    | 34      | 49.1 - 64.1 | CL | 25     |
| 44 - 56.5| CL    | 35      | 56.5 - 62| SM    | 50      | 64.1 - 70.1 | CH | 30     |
| 56.5 - 72| SC    | 50      | 62 - 67| ML        | 32      | 70.1 - 83.1 | SC | 50     |
### Table 3. Soil profile at the locations of piers: P1, A1 and A2.

| Depth (m) | Soil Type | \( N_{\text{avg}} \) | Depth (m) | Soil Type | \( N_{\text{avg}} \) | Depth (m) | Soil Type | \( N_{\text{avg}} \) |
|-----------|-----------|-----------------|-----------|-----------|-----------------|-----------|-----------|-----------------|
| 0 - 8.2   | SC        | 5               | 0 - 3.2   | SC        | 14              | 0 - 6.2   | SC        | 12              |
| 8.2 - 15.2| SP-SC     | 24              | 3.2 - 21.2| SC        | 32              | 6.2 - 13.2| CL        | 14              |
| 15.2 - 30.7| SP-SC    | 29              | 21.2 - 35.2| SC        | 37              | 13.2 - 24.2| SC        | 33              |
| 30.7 - 36.2| CL       | 29              | 35.2 - 39.2| CL        | 25              | 24.2 - 30.2| CL        | 23              |
| 36.2 - 50.2| CL       | 21              | 39.2 - 43.2| CL        | 43              | 30.2 - 57  | CL        | 35              |
| 50.2 - 55.2| CL       | 32              | 43.2 - 52.2| CL        | 33              |            |           |                 |
| 55.2 - 59.2| CL       | 30              | 52.2 - 57.5| ML        | 40              |            |           |                 |
| 59.2 - 66.2| CL       | 20              | 57.2 - 63.5| ML        | 40              |            |           |                 |
| 66.2 - 71.8| CL       | 50              |            |           |                 |            |           |                 |

### Table 4. Piles tested in this study.

| Location   | Co-ordinate          | Pile ID | Diameter (m) | Length (m) | Pile type        |
|------------|----------------------|---------|--------------|------------|------------------|
| Pier 1     | N-2604059.006m E-264739.936 m | P1      | 1.5          | 71.8       | cast in place    |
| Pier 15    | N-2604119.910m E-265956.400 m | P15     | 1.5          | 70.1       | cast in place    |
| Pier 16    | N-2604120.696m E-266038.646 m | P16     | 1.5          | 71.4       | cast in place    |
| Abutment 1 | N-2604023.240m E-264701.230 m | A1      | 1.5          | 57.5       | cast in place    |
| Abutment 2 | N-2604103.123m E-266104.697 m | A2      | 1.5          | 46.5       | cast in place    |
| Pier 5     | N-2604076.790m E-265088.470 m | P5      | 1           | 45         | Driven, steel    |

### 3. Dynamic pile load test

Dynamic load test was carried out with two pairs bolt-on strain and acceleration transducers (sensors) attached to diagonally opposite sides of pile at some distance (1.5 times pile diameter) below the pile head (figure 1). This high strain dynamic pile test conforms ASTM D4945 standard. For connecting sensors to the pile, anchor bolts and welded mounting block were used for concrete and steel piles, respectively. The compressive stress wave that was generated by the strike of driving hammer, travels down the piles and reflects upward from the pile toe. The reflected stress waves are picked up by the transducers and the signals are stored in the computer. The overall arrangement of dynamic load test is shown in figure 2.
Figure 1. Transducers mounted on pile surface: (a) schematic diagram, (b) cast-in-place pile and (c) steel pile.

While striking pile by falling a heavy hammer from a pre-determined height, the strain transducers attached to the pile measure induced strains, whereas accelerometers record the accelerations generated in the pile. The Pile Driving Analyzer converts strain into force, and acceleration records into velocities. Both force and velocity x impedance signals with time were recorded. The resistance developed by the pile is then a function of force and velocity, and it includes few factors such as the quake and damping parameters as inputs based on the soil type. The maximum pile top compression is obtained by integrating the pile top velocity. A more accurate value of these parameters is then obtained from software analysis conducted on field data.

Figure 2. Overall arrangement of dynamic load test.

Further the analysis is carried out in DLTWAVE (version 8.173), a signal matching program of Profound BV. One-dimensional wave equation theory is applied to model the pile-soil interaction. The signal matching process utilizes an iterative method, in which the results of each analysis are compared to the actual measured pile behaviour (figure 3) employing wave equation theory. The downward and upward forces are expressed by eq 1 and eq 2, respectively. At a pile section, force is the unbalanced portion of upward and downward forces.

\[
F^u = (F + Z \cdot v)/2 \quad \text{(1)}
\]
\[
F^d = (F - Z \cdot v)/2 \quad \text{(2)}
\]

where, \(Z\) = impedance and \(v\) = velocity
Reliability of DLT results in determining static capacity is checked by field calibration and comparing the value of static capacity obtained from DLT with that from static load test. Customarily, both static pile load test and DLT are carried out on the same test pile, while DLT is conducted on the same pile after a setting time after conducting static load test; 7 days and 15 days for pile in sand and clay, respectively. This study finds that static capacity obtained from DLT is found less than that from static load test, when both the tests are conducted on the same pile. Therefore, for field calibration, these two tests were conducted on two identical piles at the same pier location.

4. Results and analysis

The pile tips of four (out of five piles) cast in place concrete piles, tested in this study, rest on cohesive soil bed. The undrained cohesion of clay soil, existing below the pile tip, was determined in the laboratory and the correlation between \( c_u \) and \( N_{60} \) was observed, as shown in figure 4. It can be noted that undrained cohesion is found greater for lean clay with significant portion of fine sand than that obtained for lean clay only, though the \( N_{60} \) is the same for both of them.

In this study, the results of dynamic load test were calibrated by comparing the static capacity obtained from static and dynamic load tests, conducted on two identical cast in place piles (1.5 m in diameter and 45.1 m in length) at location of Abutment 1. Both the tests gave static capacity of pile within acceptable range: 1336 ton from static load test and 1006 ton from DLT.

In the prediction of pile tip resistance in low plastic clay, both DLT and undrained cohesion (estimated from SPT \( N_{60} \)) in the formula of static analysis follow the same trend-line with respect to \( N_{60} \) (shown in figure 5) for the test site near the Gumti river. While analyzing the influence of soil below the pile tip, clay soil was found to produce greater tip resistance than clayey sand though the value of \( N_{60} \) remained the same (figure 6).

The skin frictional resistance predicted from the formula of static analysis and DLTWAVE signal matching, is compared in table 5. It can be noted that skin friction during re-drive test is found greater that that obtained from initial drive. When the greater portion of pile length is in contact with clay soil (e.g., CL in present investigation) and its consistency is ‘very stiff to hard’, skin frictional resistance is found significantly greater than those obtained from the formula of static analysis. This indicates that

![Figure 3. Signal matching in modelling pile-soil interaction using DLTWAVE.](image)
much less skin frictional resistance was developed at the contact between ‘very stiff to hard’ clay and the pile surface than expected from the formula of static analysis.

![Figure 4. Undrained cohesion versus N60.](image)

**Figure 4.** Undrained cohesion versus N$_{60}$.

![Figure 5. Pile tip resistance versus N60.](image)

**Figure 5.** Pile tip resistance versus N$_{60}$ [Tip of concrete piles on clay; D = 1.5 m].
Table 5: Comparison of results: Skin frictional resistances.

| Pile ID | $Q_s$ from formula of static analysis | $Q_s$ from DLT (Initial Drive) | $Q_s$ from DLT (Re-Drive) | D | Pile length in contact with clay | Consistency of ‘CL’ soil |
|---------|--------------------------------------|-------------------------------|---------------------------|---|--------------------------------|-------------------------|
| P5      | 1.93 MN                               | 1.2 MN                        | 2.5 MN                    | 1 m | 24 m                     | 53.3%                   | Stiff to very stiff     |
| A2      | 7.5 MN                                | 7.7 MN                        | -                         | 1.5 m | 13 m                     | 27.9%                   | Very stiff             |
| P1      | 7.3 MN                                | 4.3 MN                        | -                         | 1.5 m | 41 m                     | 57.1%                   | Very stiff to hard      |
| A1      | 5.9 MN                                | 3.2 MN                        | -                         | 1.5 m | 22 m                     | 38.3%                   | Very stiff to hard      |

For the piles in contact with ‘stiff to very stiff’ clay (P5 and A2), skin frictional resistances from static analysis and DLT are in good agreement. For the piles in contact with ‘very stiff to hard’ clay (P1 and A1), static analysis derives greater skin frictional resistance than that obtained from DLT.

5. Conclusion
Based on the results of geotechnical investigation (such as SPT), undrained cohesion determined from laboratory investigation on undisturbed specimens and Dynamic Load Test (DLT), the following conclusion may be drawn:

1. Both DLT and the formula of static analysis gave a unique trend-line with respect to $N_{60}$ (shown in Figure 3) for the test site near the Gumti river.
2. Clay soil was found to produce greater tip resistance than clayey sand though the value of $N_{60}$ remained the same.
3. The greater the pile surface is in contact with very stiff to hard clay, the smaller the skin friction developed according DLT signal matching. This observation is found less pronounced for ‘stiff to very stiff’ consistency of clay soil than that for ‘very stiff to hard’ consistency.
4. Skin frictional resistance in re-drive test is found greater than that obtained from DLT during initial drive.
More DLTs in re-drive are needed to be carried out to investigate the variation in pile skin frictional resistance when significant portion of pile length is in contact with ‘very stiff to hard’ clay.

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