When constructing huge structures, large diameter drilled shafts are most commonly socketed into rocks to support the enormous load. Unconfined compressive strength is mostly used to calculate the capacity of drilled shaft socketed in rock; many empirical correlational equations of the sort were introduced worldwide as well. This paper employs the 23 cases of drilled shafts embedded in the Songdo area to verify the most suitable method, then moves on to suggest a new empirical formula.

In the case where it is difficult to collect the undisturbed core of weathered rocks, the traditional capacity computing equations are not useable due to the problems in defining the unconfined compressive strength. Thus, the paper employs the RQD, the deformation modulus and the elastic modulus from a pressuremeter test to deduct an empirical capacity equation.

In addition, the paper analyzes the aspect ratio which affects capacity as well. Finally, this paper has been found that the most accurate result in both the side shear resistance and the end bearing resistance.

Prediction of Ultimate Capacity of Axially Loaded Drilled Shafts Socketed in Weak Rock

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1. Introduction
2. Background Theory
3. Database
4. Result
5. Summary & Conclusions
6. Reference
1. Introduction

The recent economical growth has created a boom in the large-sized structural construction business as well. Especially, developmental efforts in the coastal area (e.g. Inchon Songdo area) has increased due to rise in demand for skyscrapers, bridges, airports, and harbors. This resulted an upheaval in the use of large-diameter drilled shafts because naturally coastal areas have soft soil.

During and after the mid 1970’s, the defining method for ultimate capacity of drilled shafts has been actively researched, and many researchers have suggested empirical prediction methods using the capacity results of load tests. However, the ultimate capacity prediction methods for previous drilled shafts socketed in rock were created with low strength and/or high quality rocks. Thus, in the cases for high strength and/or low quality rocks, the prediction methods have high error margins when compared with actual load test results. As a result it is harder to apply the prediction methods to domestic rocks such as the granite and gneiss variety among the weathered and soft; those have high strength but large amounts of discontinuity.

This paper includes a database from 15 cases of drilled shafts socketed in rock in the Song-Do area, 8 cases from the Inchon Bridge link road and land bridge; totalling 23 cases from load tests, load-transfer measured data, and subsurface exploration data. The paper predicts the behavior of pile socketed in rock by analyzing the correlation between the behavioral support of a rock section developed from assaying results from a pile load test, and the property of a rock condition obtained by a geotechnical investigation. Also, it contains information of the feasibility of previous capacity prediction methods on domestic rocks.

2. Background Theory

The most common assumption is that drilled shafts socketed in rock either supports the load by the sum of the side shear resistance and the end bearing resistance or by either one of these.

To calculate the axial capacity of pile foundation socketed in rock, we must first measure the side shear resistance and the end bearing resistance of the socketed part of the shaft. In addition, we must apprehend the load transfer mechanism
Prediction of Ultimate Capacity of Axially Loaded Drilled Shafts Socketed in Weak Rock

and the various factors that affect this effort to evaluate the general resistance of the socketed part.

2.1 Side shear resistance of drilled shaft using unconfined compressive strength

Load tests of the socketed shaft shows that the resistance of the rock layer takes up most of the total resistance. The side shear resistance of sand, clay, and fill layer above the weathered rock stratum occupies only a small portion of the whole. Thus, it is general practice to measure only the side shear resistance of the rock when designing the drilled shaft socketed in rock.

The mechanism where shear strength occurs is complicated. It relies on the friction and the strength of attachment of the contact surface between concrete and rock: that is, the direct relationship with the normal stress resulting from the dilatancy of the contact surface. Dilatancy can be addressed to the sliding effect of the contact surface.

However, since these factors are in reality hard to consider, and most of the experimental data from field and laboratory tests result in unit side shear resistance τ, it is general practice to use τ for designing purposes. Also, the experien-
to use $q_u$ for designing purposes. Also, the experiential prediction methods to estimate the unit end bearing resistance $q_u$ were proposed widely.

2.3 Resistance of drilled shaft using other properties

When given directly representative values of the strength of a rock to deformation modulus and elastic modulus from pressuremeter test, properties from field test can be assumed to have a close relationship to the end bearing resistance. Also, RQD, an index concerning the crack of a rock core, differs in value consequent to the degree of weathering; its space of discontinuity surface can be easily classified, and can be closely associated with the resistance of the rock as well. According to the study of Hoek (1983), the strength of a rock is strongly affected by the number and the interval between discontinuity surfaces; also, the research done by Kwon et al. (2008), which points to the fact that the end bearing resistance and the side shear resistance have a strong correlation with the deformation modulus: the elastic modulus: and the RQD.

3. Database

As previously mentioned, to analyze the behavioral support of the drilled shaft socketed in a soft rock representative domestically, the paper uses a load test report of the large diameter drilled shaft installed in the Incheon free economic zone (Incheon City, Yeonsu-Gu, Dongchun-Dong area), Inchon bridge link road and land bridge. Drilled shafts totalling 23 cases were used ranging in diameter 1.2~2.85 m, in length 33~70 m, and in socketed depth 7~35 m. Also, the bedrocks used were metamorphic rock and biotite granite.

The method for the drilled shaft construction is the RCD (Reverse Circulation Drill) method, and bi-directional load tests are naturally used. As for the use of field property, the paper adapted the soil property of the relevant section when calculating the side shear resistance; and in the case where the property of the concerned segment was non-existent. The paper used the average property of the borehole. When calculating the end bearing resistance, the paper used the property of the pile tip depth that has 1D or less of the overall shaft. (Peck et al. 1974)
4. Result

Among the properties of a rock, the following shows the relationship between unconfined compressive strength: deformation modulus and elastic modulus using a pressuremeter test: RQD: aspect ratio: and unit side shear resistance and unit end bearing resistance of drilled shaft.

4.1 Unconfined compressive strength of rock vs. Side shear resistance

Below is the plotted Log-Log chart showing the relationship between the unit side shear resistance of a drilled shaft from the load test, and the unconfined compressive strength. The regression curve was gained from a exponential function, and the deducted empirical formula is as below. The Table 1. shows the average of the bias factor, standard deviation, and the covariance calculated by the 16 capacity prediction methods whereas the graph shows the 2 prediction methods that produced the closed results with the measured data. The bias factor in the chart, is capacity divided by predicted capacity. It can be said that the empirical formula is conservative when the average of the bias factor is higher than 1. Also the smaller the covariance, the smaller the deviation between the measured data and the predicted data. By analyzing the statistical property of the bias factor, one can see that the method suggested by Williams et al. (1980), Rosenberg and Journeaux (1976) produces results that are the closest with the measured data.

![Figure 1] The relationship between the unconfined compressive strength and the unit side shear resistance

\[ \tau_{\text{max}} = 0.32 \sigma_{\text{c}}^{1.50} \quad (R^2 = 0.57) \]

Here, the line is the regression of measured data, and the dotted line represents the predicted data produced from the previously suggested formula.

Evaluated the correctiveness of the approximation formulas by comparing the RMSE result of the suggested formula with the three most correct prediction formulas. Although there is only a slight difference, one can see that the suggested formula has the least deviation.
4.2 Unconfined compressive strength of rock vs. End bearing resistance

The follow is the plotted Log-Log graph showing the relationship between the unit end bearing resistance of a drilled shaft, and the unconfined compressive strength. The regression curve was gained from an exponential function, and the deducted empirical formula is as below.

\[ \sigma_{unconfined} = 6.16 \sigma_{c}^{0.39} \quad (R^2 = 0.39) \]

[Figure 4] The relationship between unit end bearing resistance and unconfined compressive strength

The Table 2. shows the average of the bias factor, standard deviation, and the covariance calculated by 8 of the 11 previously mentioned capacity prediction methods; whereas the graph shows the 2 prediction methods that produced the closed results with the measured data. In methods Carter and Kulhawy(1988) and Canadian Foundation Engineering Manual (2006), there were a few instances where
values of g (aperture of the discontinuities) and s (spacing of the discontinuities) could not be obtained; and in Hoek (1983), AASHTO (1989) the deviation was too large to interpret the result. By analyzing the statistical property of the bias factor, one can see that the method suggested by ARGEMA (1992), Zhang & Einstein (1988) produces results that are the closest with the measured data.

Here, the line is the regression of measured data, and the dotted line represents the predicted data produced from the previously suggested formula.

Evaluated the correctness of the approximation formulas by comparing the RMSE result of the suggested formula with the two most correct prediction formulas. One can see that the suggested formula has the least deviation.

4.3 Pressuremeter Test vs. Side Shear Resistance

The following graph showing the relationship between the unit side shear resistance of a drilled shaft, and the deformation modulus D and elastic modulus E from pressuremeter test. The empirical function, deducted from the linear regression, is as below. One can see

[Figure 5] The predicted relationship between unconfined compressive strength of rock and end bearing resistance using ARGEMA (1992) method and Zhang & Einstein (1988) method

[Figure 6] Assessment of the correctness of the approximation formulas by comparing the RMSE (End bearing resistance)
that the correlation coefficient of side shear resistance and the property of pressuremeter test is much higher than the other properties.

[Table 1] The empirically predicted values of the statistical property of bias factor using the unconfined compressive strength of rock (Side shear resistance)

<table>
<thead>
<tr>
<th>Method</th>
<th>Bias factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean</td>
</tr>
<tr>
<td>1. Design Code of Highway Bridge (2008), AASHTO (1996)</td>
<td>1.54</td>
</tr>
<tr>
<td>2. Structural Foundation design criteria (2003), NAVFAC DM 7–2</td>
<td>1.87</td>
</tr>
<tr>
<td>3. Korea Expressway Corporation (2002), CFEM (2006)</td>
<td>2.66</td>
</tr>
<tr>
<td>4. AASHTO (2007), Horvath and Kenney (1979)</td>
<td>3.75</td>
</tr>
<tr>
<td>5. Railroad design criteria (1999), AASHTO LRFD (2004)</td>
<td>1.70</td>
</tr>
<tr>
<td>6. Horvath and Kenney (1979)</td>
<td>1.70</td>
</tr>
<tr>
<td>7. Carter and Kulhawy (1988)</td>
<td>1.79</td>
</tr>
<tr>
<td>8. Williams et al.(1980)</td>
<td>1.12</td>
</tr>
<tr>
<td>9. Rowe and Armitage (1984)</td>
<td>0.77</td>
</tr>
<tr>
<td>10. Rosenberg and Journeaux(1976)</td>
<td>1.03</td>
</tr>
<tr>
<td>11. Reynolds and Kaderabek (1980)</td>
<td>0.44</td>
</tr>
<tr>
<td>12. Gupton and Logan (1984)</td>
<td>0.66</td>
</tr>
<tr>
<td>13. Reese and O’Neill (1987)</td>
<td>0.89</td>
</tr>
<tr>
<td>14. Toh at al. (1989)</td>
<td>0.53</td>
</tr>
<tr>
<td>15. Meigh and Wolshi(1979)</td>
<td>1.30</td>
</tr>
<tr>
<td>16. Horvath (1982)</td>
<td>1.79</td>
</tr>
</tbody>
</table>

[Table 2] The statistical property of the empirical prediction values using the unconfined compressive strength of rock(End bearing resistance)

<table>
<thead>
<tr>
<th>Method</th>
<th>Bias factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean</td>
</tr>
<tr>
<td>1. Design Code of Highway Bridge (2008)</td>
<td>3.46</td>
</tr>
<tr>
<td>2. Structural Foundation design criteria (2003)</td>
<td>5.86</td>
</tr>
<tr>
<td>3. NAVFAC DM 7–2</td>
<td>4.90</td>
</tr>
<tr>
<td>4. Coates (1967)</td>
<td>0.39</td>
</tr>
<tr>
<td>5. Rowe and Armitage (1987b)</td>
<td>0.43</td>
</tr>
<tr>
<td>6. ARGEMA (1992)</td>
<td>1.89</td>
</tr>
<tr>
<td>7. Zhang &amp; Einstein (1988)</td>
<td>0.86</td>
</tr>
<tr>
<td>8. Teng (1962)</td>
<td>0.23</td>
</tr>
</tbody>
</table>
4.4 Pressuremeter Test vs. Bearing Resistance

Below is the graph showing the relationship between the unit end bearing resistance of a drilled shaft, and the deformation modulus and elastic modulus. The empirical function, deducted from the linear regression, is as below. One can see that the relationship of the deformation modulus and the elastic modulus with the end bearing resistance has a much lower correlation coefficient than that of the side shear resistance. Although the deformation modulus and the elastic modulus of rock is high, settlement of shaft could occurred excessively during the earlier stage of loading. This led to the presence of several cases of shafts with conservatively evaluated ultimate capacity. This factor is thought to be the cause of the phenomenon. Also, in a case
where the pile tip is positioned at the boundary of weathered rock and soft rock, or at the boundary of soft and hard rock, the capacity of the end bearing is more difficult to predict than that of the side shear because the capacity and the settlement can differ in variance with the ground investigation. Thus, the prediction of the pile tip resistance has to be made with great caution.

4.5 Rock Quality Designation (RQD) vs. Resistance

The follow is the graph showing the relationship between the unit side shear resistance and the unit end bearing resistance of a drilled shaft, and the RQD. Although an upfront prediction of the capacity using only the RQD value is unprobable, the empirical formula below is deducted from an approximate linear regression.

The correlation between the RQD and the side shear resistance is higher than that of the end bearing resistance. This is also because the settlement of shaft occurred excessively during the earlier stage of loading. This led to the presence of several cases of shafts with conservatively evaluated ultimate capacity. This factor is thought to be the cause of the phenomenon. The correlation is considered to be low because when the RQD is high while the rock strength is low or vice versa, it is hard to calculate the capacity using only the RQD.

4.6 Aspect Ratio vs. Resistance

Below is the graph showing the relationship between the unit side shear resistance and the unit end bearing resistance.
resistance of a drilled shaft from a load test, the aspect ratio of the shafts.

Although an upfront prediction of the capacity using only the aspect ratio is unprobable, the graph shows an approximate linear regression. The L/B, D/B shown below represents aspect ratio and length of socketed part over shaft diameter respectively. One can see that the unit side shear resistance is negatively correlated with the aspect ratio, while the unit end bearing resistance is positively correlated.

4.7 Appraisal for the empirical formulae of unconfined compressive strength

Analyzing the statistical properties of the bias factor concerning the side shear resistance while referring to the table 1, showed results that when the average of the bias factors is close to 1 and the covariance is small, the prediction method has a smaller deviation with the measured data. Also, when the average of the bias
factors is much larger than 1 the prediction method is conservative; while when the average was much smaller than 1 the prediction method overestimates. The covariance of the methods here, that have an average close to 1, are distributed around the 0.4 mark.

The following 3 methods have the most accurate predictions.

8. Williams et al. (1980)
   \[ \tau = 0.44\sigma^0.36 \text{ (MPa)} \]
10. Rosenberg and Journeaux (1976)
   \[ \tau = 0.34\sigma^0.51 \text{ (MPa)} \]
15. Meigh and Wolshi (1979)
   \[ \tau = 0.22\sigma^0.6 \text{ (MPa)} \]

The methods No. 1, 2, 3, 4, 5, 6, 7, 16 have comparatively conservative predictions. And the methods No. 9, 11, 12, 13, 14 overestimate the capacity.

5. Summary & Conclusions

Having analyzed the correlations between the side shear resistance and the end bearing resistance of drilled shafts that have been casted in the Songdo area and Incheon Bridge with properties of rock, which is an indicator of the strength and the weathered degree, a formula that can be applied to both the domestic weathered and soft rock has been derived. Furthermore, the following results have been inferred from the comparison and analysis of previous formulas that use unconfined compressive strength.

1. As the following table shows, proposed

   6. ARGEMA (1992)
      \[ qu = 4.5\sigma^0.51 \text{ (Mpa)} \]
      \[ qu = 4.83\sigma^0.51 \]

   The methods No. 1, 2, 3 have the most conservative predictions. And the methods No. 4, 5, 8 overestimate the capacity.
formulas that use properties for both the side shear resistance and end bearing resistance have been derived.

<table>
<thead>
<tr>
<th></th>
<th>Unit Side Shear Resistance (Mpa)</th>
<th>$\tau$ Correlation Coefficient ($R^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi_e$</td>
<td>$\tau_{max} = 0.32\sigma_{o,50}$</td>
<td>0.57</td>
</tr>
<tr>
<td>PMT D</td>
<td>$\tau_{max} = 0.0014D + 0.51$</td>
<td>0.68</td>
</tr>
<tr>
<td>PMT E</td>
<td>$\tau_{max} = 0.00082E + 0.45$</td>
<td>0.69</td>
</tr>
<tr>
<td>RQD</td>
<td>$\tau_{max} = 0.026RQD + 0.59$</td>
<td>0.34</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Unit End Bearing Resistance(Mpa)</th>
<th>$q_e$ Correlation Coefficient ($R^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi$</td>
<td>$q_{max} = 6.15\sigma_{o,50}$</td>
<td>0.39</td>
</tr>
<tr>
<td>PMT D</td>
<td>$q_{max} = 0.0025D + 13.2$</td>
<td>0.20</td>
</tr>
<tr>
<td>PMT E</td>
<td>$q_{max} = 0.00088E + 15.1$</td>
<td>0.043 (low correlation)</td>
</tr>
<tr>
<td>RQD</td>
<td>$q_{max} = 0.028E + 17.8$</td>
<td>0.0091 (low correlation)</td>
</tr>
</tbody>
</table>

2. It has been observed that as the aspect ratio of shaft and the length of socketed part over shaft diameter in negatively correlated with the unit side shear resistance, while it is positively correlated unit end bearing resistance.

3. After conducting an analysis of the correlation and statistical properties of the bias factor concerning the predicted values deducted from the traditional empirical prediction methods of unconfined compressive strength, it has been found that the proposed formula shows the most accurate result in both the side shear resistance and the end bearing resistance. Other than the proposed formula, the Williams et al., Rosenberg & Journeaux method shows the closest result for side shear resistance, and Zhang & Einstein and ARGEMA formula shows the closest result for end bearing resistance. And the methods can be recommended, when it comes to domestic weak rock.

4. As for the prediction methods for unconfined compressive strength, 5 out of 16 methods have a tendency to overestimate the side shear resistance, and 3 methods tend to overestimate the end bearing resistance. The empirical formulae that overestimate have a slightly higher variance, ranging from 0.7 to 0.8.

6. Reference


Resources.


