RELIABILITY BASED LOAD TRANSFER CHARACTERISTICS OF BORED PRECAST PILES EQUIPPED WITH GROUTED BULB IN THE PILE TOE REGION

SHINICHI YAMATO and MADAN B. KARKEE

ABSTRACT

End bearing load transfer characteristics of bored precast piles equipped with expanded fabric bulb in toe region is investigated based on the use of hyperbolic transfer function for shaft resistance as well as for toe resistance of the enlarged bulb portion. Based on the static loading test results, a piling method specific approach to estimate the transfer function parameters from correlation with average N-values is proposed and its reliability investigated. The reliability investigations include the vertical load resistance aspects as well as the confidence limits for vertical movement. It is confirmed that the shaft resistance of the enlarged fabric bulb portion is much larger compared to the conventional bored precast piles and the increase is a function of the pressure applied in injecting the cement slurry into the fabric sack to form the bulb. On the average, it appears that the increase in bulb shaft resistance is of the order of 7% for every 0.1 N/mm² increase in pressure.

Key words: bearing capacity, bored precast piles, confidence limit for settlement, geosynthetic bulb, N-value, statistical analysis (IGC: E4/K7)

INTRODUCTION

Driven piles have become increasingly difficult to implement in Japan following the relevant regulations enacted in the late sixties and mid-seventies in response to serious public complaints about noise and vibration during construction, which in turn resulted in the development of alternative installation methods for precast piles to replace impact driven piles (Karkee, 1999). Several alternative installation methods with varying degree of difference in the approach to address noise and vibration concerns have been developed. Overall, these methods are termed as bored precast piles. Compared to about 75% of piles installed by driving in mid-seventies, the makeup of piling methods in Japan changed substantially from early eighties when the driven piles consisted of only about 13% (Karkee, 1999), indicating a fairly rapid changeover. Over the following decades, the trend has evolved into a continuous process of modification and improvement of the existing methods of bored precast piles, including the use of newly developed materials and equipment for better quality assurance and reliability. The bored precast pile equipped with grouted fabric bulb in the pile toe region, developed recently (Nakagawa et al., 1999; Yamato et al., 2000), is one such example.

Various methods of bored precast piles are not amenable to unified approach to quality assurance and bearing capacity determination similar to the use of pile driving analysis (Smith, 1960; Goble et al., 1980) in case of driven piles. As such, owing to the dominance of installation method resulting in various degrees and nature of disturbance to surrounding soil, it is difficult to adopt a coherent theoretical solution to evaluate the bearing capacity of bored precast piles based simply on the typical soil parameters obtained from soil exploration prior to pile installation. Consequently, the use of various methods of loading test (Karkee, 2000; Karkee and Kishida, 1999) provide the preferred and realistic option for logical evaluation of the load bearing characteristics of bored precast piles. In practice, the bearing capacity evaluation is based on the appraisal of a suite of data from static loading tests on fully instrumented piles (Yamagata et al., 1992). Actually, the bearing capacity relations developed based on static loading tests database system are specific to the type of pile and its method of installation (e.g. Karkee et al., 1998; Nakagawa et al., 1999). Beginning with the salient features of bored precast piles with pressure grouted fabric bulb in the pile toe region, this paper introduces the approach to estimate the load transfer characteristics of such piles. The approach is based on the correlation of load transfer parameters derived from the loading test results with the average N-value to propose the piling method specific

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The Japanese Geotechnical Society

SOILS AND FOUNDATIONS Vol. 44, No. 3, 57–68, June 2004
Japanese Geotechnical Society

57
formulations for estimation of the load movement characteristics specific to this piling method. A database system based on loading tests on fully instrumented piles was compiled for this purpose and the nature of confidence interval and reliability inherent in this approach are discussed. It is seen that the improved quality management during installation in this method results in narrower distribution of the estimated nominal resistance compared to conventional bored precast piles, and although the sample size utilized for statistical study is limited, the nature of distribution is found to conform with previous studies with larger sample size (Karkee et al., 1998, 1999). Investigation of the 90% confidence limit for movement and corresponding estimated reliability indices at some possible values of factor of safety indicate the importance of judgment that may be exercised for rational design.

**PILING METHOD AND QUALITY CONTROL**

The piling system considered in this investigation consists of the bored precast pile with a geosynthetic fabric sack attached prior to installation, such that a bulb is formed in the toe region when the sack is expanded by pressure grouting with cement slurry after placement. Figure 1(a) shows the section details of the pile and the general outline of the bulb formed in the toe region. The type of pile consists of the factory made spun high strength (more than 98 N/mm²) concrete pile encased in steel pipe shell, and is generally known as the SC piles in Japan. The SC piles to be utilized for this piling method are specially manufactured with grouting pipes embedded within the concrete as shown in Fig. 1(a). The longest pile segment manufactured is 15 m and, when segments are to be welded in-situ for longer piles, the grouting pipes are fitted with o-rings for proper continuity through the pile splice down to the inside of the seamless geosynthetic fabric sack.

The test pile instrumentation, consisting of the general layout of strain gages for the monitoring of axial load transfer, is also illustrated in Fig. 1(b). The piling method was developed to provide high and reliable bearing capacity from deeper bearing stratum without much reliance on the shaft resistance of overlying soft soil layers, including filled ground. This type of piling system finds rational foundation option at typical deep soil conditions in the bay city areas in Japan, where the sites often consist of reclaimed ground. Consequently, the purpose of the series of loading tests considered here is to evaluate the bearing resistance derived from the enlarged segment (bulb) at the pile toe region, and the general layout of the test pile instrumentation in Fig. 1(b) was adopted in consideration to this purpose. Accordingly, all the test piles were installed to provide practically negligible shaft resistance from the upper portion by adopting special methods to cut shaft friction, the effectiveness of which can be understood from the typical axial distribution diagram subsequently (Fig. 7).

**The Installation Sequence**

The basic sequence of pile installation is given in Yamato et al. (2000) and is illustrated schematically in Fig. 2. The method is similar to the conventional pre-boring and toe enlargement method (Karkee, 1999). When the drilling reaches the desired depth as the forward (clockwise) rotation continues, the enlargement bits in the auger-head are opened out by reverse rotation. The diameter of the borehole over a certain length at the bottom is enlarged as the reverse rotation of the enlargement bits is continued while moving the auger-head up and down over the length. The length of the enlargement bits is set such that the diameter of enlarged portion is twice the outer diameter of the pile $D_p$ (Fig. 4).

Once the enlargement operation is complete, the en-
largement bits are made to close in by rotating the auger-head in the forward direction again, and the drilling rod and auger is raised along with the injection of cement slurry (water cement ratio 100%). The volume of the slurry is adjusted so as to completely fill the enlarged portion, as illustrated in Fig. 2, when the pile attached with the fabric sack is inserted. If the sand or gravel content in the ground is high, bentonite is added to the cement slurry to aid stability of the borehole. The SC pile attached with the fabric sack at the pile toe is inserted centrally into the hole immediately after raising the auger-head completely from the borehole. When necessary, pile segments are welded in situ after the grouting pipe in the upper segment is inserted into the connecting block of the lower one. SC piles to be installed by this method have grouting pipes of 32 mm or 38 mm in diameter (depending on the pile diameter) embedded centrally within the concrete wall thickness (Fig. 1). The required length of the pile is set centrally in the borehole such that the pile toe lies approximately within 650 mm from the bottom as shown in Fig. 4. Immediately after the pile is set in position, the fabric sack is expanded by injection of cement slurry (water cement ratio 60%) under pressure so that a bulb is formed in the pile toe region.

The fabric sack is made of synthetic fibers such as nylon 66 woven as tightly as to prevent the passage of cement particles when cement slurry is injected into the seamless sack under pressure. The material of the fabric is designed and tested to have sufficient reserve strength when subjected to the designated maximum pouring pressures in the open space, without any surrounding restraint actually available during the pile installation described above.

**Monitoring and Quality Control**

Since the piling method was developed to provide high and reliable resistance from the enlarged bulb region without much reliance on the shaft resistance of overlying soft soil layers, the quality control during construction hinges on ensuring adequate expansion of the fabric sack to form a solid bulk properly locked in the ground after the cement slurry hardens. Accordingly, the installation method adopts standard procedures for proper monitoring and documentation during the grouting operation of injecting cement slurry into the fabric sack under pressure. The pressure rises significantly as the expanding fabric bulb tries to occupy the enlarged portion of the borehole. As a safety precaution to keep the pressure well within capacity of the fabric and attachments, the maximum pressure is limited to 0.59 MPa. The flow rate (ℓ/min), cumulative volume (ℓ) and pressure (N/mm²) are continuously recorded by the monitoring system shown in Fig. 3(a) during the grouting operation, forming an integral part of the quality control documentation. The standard sequence of injection followed is depicted in Fig. 3(b), where the general trend of pressure variation with time can be noted. When the pressure reaches the prescribed maximum of 0.59 MPa, the pump is made to stop such that the pressure drops as the seepage of water through the fabric sack progresses slowly with time. It may be noted that the sack is woven tightly to prevent washing of cement particles when water in the cement slurry escapes slowly under pressure. Injection of slurry by pumping is resumed after the pressure drops to

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<tbody>
<tr>
<td>Dp (mm)</td>
<td>Dp (mm)</td>
<td>Dp (mm)</td>
<td>Dp (mm)</td>
<td>Dp (mm)</td>
<td>Dp (mm)</td>
</tr>
<tr>
<td>300 (318.5)</td>
<td>450</td>
<td>600</td>
<td>515</td>
<td>2.10</td>
<td></td>
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<tr>
<td>350 (355)</td>
<td>500</td>
<td>700</td>
<td>600</td>
<td>2.30</td>
<td></td>
</tr>
<tr>
<td>400 (400)</td>
<td>550</td>
<td>800</td>
<td>690</td>
<td>2.55</td>
<td></td>
</tr>
<tr>
<td>450 (450)</td>
<td>600</td>
<td>900</td>
<td>775</td>
<td>2.60</td>
<td></td>
</tr>
<tr>
<td>500 (500)</td>
<td>650</td>
<td>1000</td>
<td>860</td>
<td>2.65</td>
<td></td>
</tr>
<tr>
<td>600 (600)</td>
<td>750</td>
<td>1200</td>
<td>1030</td>
<td>2.70</td>
<td></td>
</tr>
</tbody>
</table>

* Figures in brackets show actual diameter of SC piles as determined by steel casings commercially available
* h₁ depends on the auger type, but is usually about 350 mm
* h₂ is kept to within about 650 mm

![Fig. 4. Standard bore diameters and grouted bulb sizes for various pile diameters](image-url)
0.49 MPa, and the process is repeated for three more cycles as illustrated in Fig. 3(b).

Owing to the grouting pipe sizes, the maximum rate of injection (volume per unit time) depends on the pile diameter. The standard volume of cement slurry also depends on the pile diameter and is designated as the volume necessary to achieve the standard bulb diameter of 1.7 times the pile diameter. Allowable range of variation above and below the standard volume is defined for rational monitoring to achieve targeted quality. Upper limit in volume corresponds to 1.2 times the standard bulb diameter, which sets the limit to which the fabric sack can be expanded without excessive pressure. The lower limit in turn corresponds to 0.8 times the standard bulb diameter and aims to set the limit below which the pile bearing capacity may be compromised. The bore diameters and expanded fabric bulb diameters specified for various SC pile diameters in use are given in Fig. 4. The rigorous quality control regime followed in the installation process ensures consistent vertical load resistance of the expanded fabric bulb, as demonstrated subsequently.

**Bulb Diameter and Strength of Hardened Cement Grout**

It is evident that the capacity of the inflated bulb to resist axial load is primarily derived from the transfer of load to the bearing stratum by the hardened cement grout of water cement (w/c) ratio of 60% injected into the seamless fabric sack. Consequently, it is crucial in this piling method to ensure that the hardened cement grout has adequate strength. The 25 m long SC pile shown in Fig. 5 was installed by following the sequence described above for the purpose of direct inspection and verification of the strength of hardened cement grout inside the expanded fabric bulb in the toe region. The pile was pulled out of the ground after encasing it completely by the casing of inside diameter larger than the fabric bulb diameter as shown in Fig. 5. The open-ended 1800ø casing was screwed in as the soil was loosened by water jet at the bottom, such that the soil around the pile and within the casing was mixed with water as the casing penetration progressed. The pile was easily pulled out after completion of the casing installation.

After the pile was pulled out and cleaned of debris, the bulb diameter and length was carefully measured. Length of the fabric bulb was in conformity with the length of seamless fabric sack fixed to the SC pile, indicating that the bulb length could be predetermined realistically by suitably setting the fabric sack. In addition, it was confirmed that the bulb diameter could be realistically estimated from the volume of cement slurry injected and the predetermined bulb length.

After the completion of measurements, cores of the hardened cement grout in the fabric bulb were taken from three locations A, B and C (one month after installation), as shown in Fig. 5. Three samples i, m and o, each 100 mm long and 50 mm × 50 mm in cross section, were taken from each of the cores A, B and C respectively as shown schematically in Fig. 5. It may be noted that the three test samples from each core segments subjected to compressive strength test were taken from inner (i), middle (m) and outer (o) portions of the bulb thickness. The compressive strength, Young’s modulus and Poisson’s ratio obtained from the test are given in Table 1. It can be noted that hardened cement grout has sufficiently large compressive strength of the order of 50 N/mm² and the Young’s modulus and Poisson’s ratio values are comparable to those of high strength concrete. Although the strength at outer part of the bulb is higher, the Young’s modulus and Poisson’s ratio are similar for samples from different regions of the bulb.

<table>
<thead>
<tr>
<th>Sample notation in Fig. 5</th>
<th>Compressive strength $\sigma_{max}$ (N/mm²)</th>
<th>Young's modulus $E \times 10^5$ (N/mm²)</th>
<th>Poisson's ratio</th>
</tr>
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<tbody>
<tr>
<td>Sample</td>
<td>Average</td>
<td>Sample</td>
<td>Average</td>
</tr>
<tr>
<td>i</td>
<td>A</td>
<td>81.4</td>
<td>77.03</td>
</tr>
<tr>
<td>m</td>
<td>B</td>
<td>77.0</td>
<td>2.40</td>
</tr>
<tr>
<td>C</td>
<td>72.7</td>
<td>2.41</td>
<td>0.23</td>
</tr>
<tr>
<td>o</td>
<td>A</td>
<td>54.6</td>
<td>2.44</td>
</tr>
<tr>
<td>C</td>
<td>B</td>
<td>49.2</td>
<td>51.10</td>
</tr>
<tr>
<td>C</td>
<td>49.5</td>
<td>2.40</td>
<td>0.24</td>
</tr>
<tr>
<td>i</td>
<td>A</td>
<td>51.6</td>
<td>51.33</td>
</tr>
<tr>
<td>m</td>
<td>B</td>
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<tr>
<td>C</td>
<td>51.8</td>
<td>2.46</td>
<td>0.23</td>
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</table>

**LOADING TEST ON FULLY INSTRUMENTED PILES**

Loading tests were conducted at various soil conditions on SC piles ranging from 300 to 600 mm in outer diameter. The length of test piles range from 15 m to 69 m.
such that the pressure grouted bulb is located within the bearing stratum. Since the purpose of the investigation was to evaluate the vertical resistance of the expanded fabric bulb in the pile toe region, all the test piles were provided with friction cut material along the pile shaft. However, short segments of pile shaft (such as FG and HI in Fig. 7) were left without friction cut for checking the extent of shaft resistance that may be expected in the absence of friction cut treatment.

Loading tests were conducted based on the Japan Geotechnical Society standard (JGS, 1990). Accordingly, the incremental loads were maintained at constant value for a period of 30 minutes and the load was cycled back to zero incrementally. The load applied at pile head ($P_h$), pile head movement ($S_h$) and pile toe movement ($S_t$) were measured directly. Grouting pipe used for injecting the cement milk into the seamless fabric sack was utilized for the measurement of $S_t$. The grouting pipe was washed with water right after completion of the pile installation process to leave the hole open for direct access down to the middle part of fabric bulb. A steel rod was inserted through the grouting pipe for direct measurement of $S_t$. Figure 6 shows the trend of pile head and pile toe movements ($S_h$ and $S_t$) with the increase in load applied at pile head ($P_h$) for the case of pile shown in Fig. 7. It can be seen that the difference between $S_t$ and $S_h$ indicates elastic compression of the pile body. Similar trend was observed in all the nine test piles considered in this investigation. Salient features of the test piles and site conditions are summarized in Table 2, where the bulb diameters ($D_b$) were as obtained from the predetermined length of the fabric sack attached to the pile and the volume of cement milk injected, as noted above.

**Axial Force Distribution**

The axial force distribution was derived from the strain gage measurements along the pile length, a typical layout of which is shown in Fig. 1(b). Test piles were specially manufactured with strain gauges attached to re-bars embedded centrally within the high strength concrete section of SC piles. The variation in the cross sectional area and material composition along the pile axis was taken into account in computing the axial force distribution from strain gage readings (Nakagawa et al., 1999, 2000). Figure 7 shows a typical axial force distribution together with the general soil profile and pile layout, where the strain gage locations at different levels are denoted alphabetically as A to J from the bottom. It may be noted that bulk of the pile head load is directly transmitted to and resisted by the expanded fabric bulb in the pile toe region. It may also be noted that shaft resistance develops adequately in segments without friction cut (e.g. segments FG and HI in Fig. 7).

**Load Transfer by Bulb Shaft Resistance and Toe Resistance**

Since bulk of the vertical load in this type of pile is to be resisted by the expanded fabric bulb in the toe region, it was decided to make a detailed measurement of the stresses developed over the enlarged bulb portion, as indicated by close spacing of strain gages (A to E) in the toe region shown in Fig. 1(b). It is clear that the capacity of expanded fabric bulb to resist vertical load is derived from the shaft resistance as well as the toe bearing.
resistance. For consistency, strain gage readings between levels B and D were utilized in all the cases to obtain the average shaft resistance ($\tau$) of the expanded bulb portion (i.e., based on the fabric bulb diameter $D_b$) at various movement extent of pile movement. Two typical load transfer ($\tau$ versus $S_p$) relations for the bulb portion thus obtained from load test numbers 1 and 6 in Table 2 are shown in Fig. 8. Similarly, the corresponding load transfer curves for pile toe resistance ($q$), represented by $q$ versus $S_p$, relations are shown in Fig. 9. The strain gage location at B has been assumed to represent the toe resistance because the strain gage A was located only 200 mm above the lower end of the pile (as may be noted in Fig. 1), which tends to lie in the transition region where the toe dimension increases from pile diameter $D_p$ to the fabric bulb diameter $D_b$. Strain gage location B was located within 500 mm above strain gage location A, where the bulb diameter $D_b$ could be considered with confidence as the pile toe diameter to obtain pile toe resistance ($q$). Since the fabric bulb portion is relatively short compared to total pile length, the difference in bulb shaft movement and the pile toe movement was found to be negligible. Accordingly, the measured pile toe movement $S_p$ has been utilized to obtain load transfer curves for bulb shaft resistance ($\tau$) as well as for toe resistance ($q$).

**LOAD TRANSFER CURVES AND SOIL PARAMETERS**

Attempt was made to explore the adequacy of modeling the load transfer relations for shaft resistance of the fabric bulb portion and the toe resistance as defined above by transfer functions commonly used for this purpose. For comparison, hyperbolic curves and parabolic (second order) curves were considered for representation of the load transfer by toe resistance. Based on the correlation with measured load transfer relation, it was concluded that hyperbolic curves provided better overall representation of the load transfer by toe resistance. Consequently, the load transfer curves for the shaft resistance as well as the toe resistance were approximated by hyperbolic equations fitted to measured data for all the cases, of which the typical cases are shown in Figs. 8 and 9. For fitting of the hyperbolic equations to the data, plots of $S_p/\tau$ or $S_p/q$ with respect to $S_p$ were prepared and straight lines were fitted to the data by least square regression analysis, such that the intercept and the

![Fig. 8. Typical curves for load transfer by shaft resistance in the expanded fabric bulb](image)

![Fig. 9. Typical curves for load transfer by toe resistance of expanded fabric bulb](image)

**Table 3. Parameters of hyperbolic load transfer curves fitted to the data**

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$\frac{1}{\alpha}$ (N/mm²)</th>
<th>$\frac{1}{b}$ (N/mm²)</th>
<th>$\tau^2$</th>
<th>$\frac{1}{c}$ (N/mm²)</th>
<th>$\frac{1}{d}$ (N/mm²)</th>
<th>$\tau^2$</th>
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<tr>
<td>1</td>
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<td>0.090</td>
<td>0.99</td>
<td>9.615</td>
<td>0.322</td>
<td>0.93</td>
</tr>
<tr>
<td>2</td>
<td>0.806</td>
<td>0.088</td>
<td>0.96</td>
<td>6.562</td>
<td>0.659</td>
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<tr>
<td>3</td>
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<td>0.099</td>
<td>0.98</td>
<td>8.568</td>
<td>0.921</td>
<td>0.98</td>
</tr>
<tr>
<td>4</td>
<td>0.509</td>
<td>0.292</td>
<td>0.99</td>
<td>10.693</td>
<td>0.660</td>
<td>0.98</td>
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<tr>
<td>5</td>
<td>0.860</td>
<td>0.228</td>
<td>0.98</td>
<td>9.158</td>
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<tr>
<td>6</td>
<td>0.564</td>
<td>0.106</td>
<td>0.98</td>
<td>17.068</td>
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<td>0.71</td>
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<td>7</td>
<td>0.663</td>
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<td>14.941</td>
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<tr>
<td>9</td>
<td>0.838</td>
<td>0.066</td>
<td>0.94</td>
<td>15.740</td>
<td>0.473</td>
<td>0.95</td>
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slope of the fitted straight lines provided the relevant parameters of the hyperbolic transfer curves (Hirayama, 1990). The relevant parameters of the hyperbolic load transfer curves for various loading tests obtained by fitting the straight lines are shown in Table 3, where the multiple correlation coefficients squared ($r^2$) are above 0.95 in most cases as shown, indicating good fit with the data. Typical results of fitting straight lines to $S_u/r$ versus $S_u$ and $S_u/q$ versus $S_u$ data are shown as corresponding hyperbolic curves together with $r^2$ values in Figs. 8 and 9 respectively. As also shown in Table 3, $a$ and $b$ refer to parameters defining the hyperbolic curves for load transfer by bulb shaft resistance, while $c$ and $d$ refer to those for load transfer by toe resistance. From the nature of hyperbolic transfer function, it is known that $1/a$ and $1/c$ represent the ultimate shaft resistance of the enlarged bulb part and the ultimate pile toe bearing resistance respectively.

**Soil Parameter for Correlation with Transfer Curve Parameters**

In case of non-cohesive soils such as sand and sandy gravel, it is common practice in Japan to evaluate the pile toe resistance of bored piles based on the average $N$-value (blow counts). Considering that the soil type in toe region actually consists of sandy or sandy gravel in most cases, it is fair to say that average $N$-value forms the basis for evaluating the toe resistance in design practice. It is, however, important to consider how the pile toe region over which the $N$-values are averaged is defined. Evidently drawn from the concept of pile toe failure mechanism comprising shear zones extending upward over part of the pile shaft conceived by Mayerhof (1951), the region over which the $N$-value is averaged was defined as $4D_n$ above and $1D_n$ below the pile toe, where $D$ is the pile diameter in toe region. However, Mayerhof’s general failure mechanism is inconsistent with the assumption of incompressible materials and is actually recognized to be unrealistic. In recognition of this discrepancy, other approaches such as considering the region $1D_n$ above and $1D_n$ below the pile toe (Yamagata et al., 1992) for obtaining the average $N$-value applicable to pile toe resistance have been proposed as an alternative.

However, when the pile toe is embedded into a deep stratum below the ground surface, the soil failure mechanism is likely to be dominated by punching mode, in which a triangular wedge is formed below the pile toe and radial shear zones around the wedge penetrate down into the soil below the pile. Actually, the general nature of load movement curves at pile toe obtained from loading tests can, in general, be regarded as consistent with the failure mechanism controlled by punching mode. In this situation, larger region of the soil, below the pile toe than above the pile toe may be logically considered to contribute to pile toe resistance. Consequently, it would be rational to adequately consider contribution of comparatively larger region of soil below the pile toe while averaging $N$-values regarded as contribution to pile toe resistance. This problem has been investigated in detail by Saeki et al. (2000) and based on their study the weighted average of $N$-values over the region $4D_n$ below the pile toe is proposed to be appropriate. As a way to recognize the contribution of a larger region of soil below the pile toe, while also accounting for soil immediately above, simple average of $N$-values in the region $4D_n$ below and $1D_n$ above the pile toe has been adopted in this paper. Since the pile diameter in the toe region corresponds to fabric bulb diameter $D_n$, the average $N$-values contributing to pile toe resistance, as shown in Table 2, correspond to the simple average over $4D_n$ below and $1D_n$ above the pile toe.

**Correlation of Load Transfer Curve Parameters with $N$-values**

An attempt was made to investigate the possibility of correlating the hyperbolic transfer curve parameters shown in Table 3 with the average $N$-values. Considering the limited sample size, the distinction between sandy and sandy gravel soils was disregarded in developing the correlations. The average $N$-values considered to be applicable for toe bearing (also shown in Table 2) are designated as $N_b$. The average $N$-values over the interval of strain gage levels B and D (Fig. 1), designated as $N_s$, are considered to be applicable for bulb shaft resistance. The correlation of the corresponding load transfer curve parameters with $N_b$ and $N_s$ are shown in Figs. 10 and 11 respectively, where the shaded portions indicate the 90% confidence interval assuming unit normal distribution of the statistic. It may be noted that confidence interval is defined in terms of the intercept in all cases except for $1/c$ in Fig. 11(a), where the intercept is zero, the 90% confidence interval applicable to the slope is indicated. This is
in consideration of the trend observed in practice (e.g., Karkee et al., 1998) that the correlation of pile toe resistance with average N-value in non-cohesive soil tends to be represented closely by a straight line passing through the origin.

For comparison, Figs. 10(a) and 11(a) also show the relationships proposed by Hirayama et al. (1997) for bored precast nodular piles at sandy soils. Although based on a comparatively larger sample size, their relationships are limited to loading tests at sites with pile toe region N-values of less than about 30. Accordingly, relations proposed by Hirayama et al. (1997) are shown as extrapolated by dotted lines to N-value ranges of interest in this study. It may be noted that the ultimate value of unit toe resistance q, as represented by 1/c in Fig. 11(a), compare closely. However, the ultimate value of the unit shaft resistance r of the pile bulb represented by 1/a in Fig. 10(a) is clearly much larger for the enlarged bulb considered for investigation in this paper. Such significantly large shaft resistance of the pile bulb is attributed to the expansion of fabric sack by injecting the cement slurry under pressure. This phenomenon is investigated and explained further subsequently.

MEASURED AND COMPUTED RESISTANCE

The hyperbolic transfer curve parameters obtained directly from the loading test data, regarded as measured values, and those estimated from correlation with N-values, regarded as computed values, were compared by defining the ratio between measured and computed values. For example, the ratio between measured and computed values of the ultimate bulb shaft resistance given by 1/a was represented by ψ, and its distribution is shown in Fig. 12. Although the sample size is small, the distribution of ψ, tends to be skewed with larger dispersion on the higher side. The trend is characteristically similar to that noted by Karkee et al. (1999) for a much larger sample size, where the lognormal distribution was found to be appropriate for the statistics. Accordingly, the probability density of ψ, assuming lognormal distribution and its cumulative probability, are also shown in Fig. 12. It may be noted that the lognormal distribution provides a reasonable approximation of the statistic and that the probability of computed value exceeding the measured value, p(ψ<1), is about 46%. This is reasonable considering that computed values of 1/a are based on average relationship in Fig. 10(a). It is also noted from the cumulative probability curve that the 90% confidence interval for ψ, is 0.65 to 1.6, as indicated by the horizontal bar in Fig. 12. Similar trend was noted for all the four load transfer curve parameters which are summarized in Table 4.

The JGS (1990) standard defines ultimate resistance obtained from loading test as the smaller of the pile head load (Pb) at a pile toe movement (Sb) of 10% of the pile diameter in the toe region and the load at which the load movement curve may be regarded as parallel to the movement axis. In case of loading tests considered in this investigation, it was noted that none of the load movement curves tended to be parallel to the movement axis before Sb reaching 10% of the bulb diameter Db. Accordingly, the ultimate vertical capacity was defined as the load at Sb=0.1Db for all the cases in this investigation. Measured ultimate resistance at pile head (Rmb) was obtained from the Pb=Sb curve as illustrated in Fig. 6. Similarly, the measured pile bulb resistance (Rmb) was obtained as the load at strain gage E in Fig. 1(b), located just above the pile bulb, when Sb=0.1Db. It may be noted that the difference between Rmb and Rnb is the shaft resistance of SC piles above the pile bulb portion, over which the pile was treated to reduce friction resistance to a minimum as described above.

The computed nominal resistance of the pile bulb (Rn) was obtained as the sum of bulb shaft resistance and toe resistance at Sb=0.1Db. For this, the hyperbolic transfer functions with parameters determined from correlations with N-values given in Figs. 10 and 11 were utilized. As noted above, the toe resistance is assumed to correspond to strain gage location B in Fig. 1(b), which is at a distance of 600 mm from the bottom for all cases. Accordingly, the bulb shaft resistance was computed for the portion (Lb=600), where Lb is the bulb length as illustrated in Fig. 4. The applicable values of Lb and Db utilized in the computations are as given in Table 2. It may be noted that the nominal resistance Rn is also referred to as the computed ultimate resistance.

Validity of Computed Nominal Resistance

Similar to the case of hyperbolic transfer curve parameter in Fig. 12, the validity of the computed nominal resistance Rn, based on the average relationships in Figs. 10 and 11, can be investigated by defining the

<table>
<thead>
<tr>
<th>Probability of ratio  &lt;1.0</th>
<th>46%</th>
<th>59%</th>
<th>63%</th>
<th>53%</th>
</tr>
</thead>
<tbody>
<tr>
<td>90% confidence interval</td>
<td>0.65-1.60</td>
<td>0.50-1.70</td>
<td>0.55-1.45</td>
<td>0.55-1.70</td>
</tr>
</tbody>
</table>

Table 4. Hyperbolic load transfer curve parameters
ratios $\psi_b = R_{mb}/R_s$ and $\psi_h = R_{mh}/R_s$. The distribution of the values of $\psi_b$ and $\psi_h$ are shown in Fig. 13, where it can be noted that ratios lie within a narrower range compared to Fig. 12, while again tending to follow lognormal distribution. The corresponding probability densities and cumulative probabilities are also shown in Fig. 13. As may be expected for $R_s$ computed based on average relationships, probability of $\psi_b < 1$ (i.e., $R_s > R_{mh}$) is of the order of 50%, and the 90% confidence interval for $\psi_b$ lies between 0.80 and 1.25, as indicated by the horizontal bar in Fig. 13(a). This interval is much narrower than those for hyperbolic transfer function parameters summarized in Table 4, of which Fig. 12 constitutes a typical case.

Figure 13(b) shows the distribution of $\psi_h$, together with the applicable probability density and cumulative probability assuming lognormal distribution. Even though the test piles were specially prepared to have minimum shaft resistance as described above, $R_{mh}$ is slightly larger than $R_{mb}$ because of the existence of some shaft resistance as noted in Fig. 7 as a typical case. Although $R_{mh}$ is only about 9% higher than $R_{mb}$ on the average for nine loading tests described in this study, the probability of $\psi_h < 1$ in Fig. 13(b) reduces to 10% (i.e., 90% confidence of $R_s$ not exceeding $R_{mh}$) from that of 50% for $\psi_b < 1$ in Fig. 13(a). In addition, the 90% confidence interval for $\psi_h$ becomes narrower (lies between 0.97 and 1.37) as shown by the horizontal bar in Fig. 13(b). Reasonably low probability of only 10% for $R_s$ exceeding $R_{mh}$ combined with a narrow 90% confidence interval of 0.97–1.37 for $\psi_h$ indicates the validity and reliability of evaluating the nominal resistance of pile bulb following the piling method specific approach proposed in this paper. Normally, the shaft resistance of pile above the fabric bulb portion, which was purposely treated to reduced to minimum in the test piles considered here, can be expected to be quite substantial. Accordingly, the computed nominal resistance of pile bulb ($R_s$) can be expected to constitute a conservative estimate of the nominal resistance of this type of pile.

**Factor of Safety and Reliability Index**

Figure 13 may be logically regarded as representing the nature of distribution of the computed resistance, for which the corresponding values of average $\mu$, standard deviation $\sigma$ and coefficient of variation $V_s$ are also shown. Becker (1966) reports that the coefficient of variation $V_s$ of the load effects ranges between 0.12–0.19 and has a typical value of 0.15. Assuming the ratio of mean resistance to mean load as the factor of safety (FS), it would be simple to evaluate the reliability index $\beta$ for various values of FS (Karkee et al., 1999). Usual practice in Japan has been to apply FS of 3.0 to the ultimate resistance. Based on FS = 3.0, the reliability index $\beta$ for the computed nominal resistance $R_s$ works out to be about 6.0 in relation to $R_{mh}$ and about 7.0 in relation to $R_{mb}$, which are substantially larger than $\beta = 3.2$ proposed for the axial resistance of deep foundations for analysis based on static loading test results (Becker, 1996). Attempt was made to estimate $\beta$ by decreasing FS to 2.5 and 2.0 and the results are shown in Table 5. It appears that FS of as low as 2.0 may be acceptable for the purpose of satisfying the reliability condition for strength requirement. The results indicate some clear trends to be expected in reliability-based approach. However, the variability in and reliability of $N$-values utilized for correlation with transfer function parameters also needs to be considered for more conclusive results. In addition, the design requirement should also include the need to satisfy settlement limitations with adequate reliability. This aspect is invested further as follows.

**Table 5. Design considerations by varying factor of safety (FS)**

<table>
<thead>
<tr>
<th>Particulars</th>
<th>$\psi_b = R_{mb}/R_s$ (COV = 0.12)</th>
<th>$\psi_h = R_{mh}/R_s$ (COV = 0.10)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reliability index $\beta$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>FS = 3.0</td>
<td>6.0</td>
<td>9 mm</td>
</tr>
<tr>
<td>FS = 2.5</td>
<td>3.7</td>
<td>12 mm</td>
</tr>
<tr>
<td>FS = 2.0</td>
<td>5.4</td>
<td>17 mm</td>
</tr>
<tr>
<td>90% confidence limit for $S_s$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$S_{mb}$</td>
<td>8 mm</td>
<td>4.1</td>
</tr>
<tr>
<td>$S_{mh}$</td>
<td>9 mm</td>
<td>11 mm</td>
</tr>
<tr>
<td></td>
<td>9 mm</td>
<td>18 mm</td>
</tr>
</tbody>
</table>
Reliability in Limiting Pile Toe Movement

It is evident from Fig. 6 that the load resisted by piles equipped with expanded fabric bulb tends to continue increasing at a decreasing rate as the movement progresses, and the reliability from strength considerations alone cannot produce satisfactory design. Another important aspect is the estimation of the extent of pile movement under loading conditions expected over the long term, together with the reliability with which such movement can be estimated. It is in this context that the piling method specific design approach proposed in this paper emphasizes estimation of the load movement characteristics, rather than the strength considerations alone. As noted above, measured ($R_{pm}$ and $R_{mh}$) as well as computed ($R_e$) nominal resistance values were defined at $S_p = 0.1D_p$ for all cases. It would be of interest to see how the corresponding measured and computed values of pile toe movement ($S_p$) compare for various values of FS applied to $R_e$. For rigorous consideration, the pile diameter itself can be expected to be a factor in defining the limiting toe movement. However, since the diameter of the fabric bulb lies with a fairly narrow range about 0.7 m to 1.0 m (Table 2), it is considered reasonable to disregard this factor for tentative evaluation of limiting pile toe movement.

The computed pile toe movement ($S_{pm}$) values correspond to the pile toe movement at the load of $R_e$ divided by certain FS obtained from the hyperbolic curves comprising parameters $c$ and $d$ shown in Table 3, while the measured toe movement ($S_{pm}$) values are those obtained directly from loading test results ($P_m-S_p$ curves) for the same load. The distributions of $S_{pm}$ and $S_{p}$, for various values of FS applied to $R_e$ are shown in Fig. 14. Again, the data seems to follow lognormal distribution, as also noted in the previous study (Karkee et al., 1999). The applicable probability density and cumulative probability assuming lognormal distribution are shown in Fig. 14, based on which the 90% confidence limit for movement, meaning 90% probability of not exceeding the limit, may be defined.

It is interesting to note in Fig. 14 that the distributions of $S_p$ and $S_{pm}$ are quite similar for the corresponding FS values. The distributions become flatter with consequent increase in the 90% confidence limit for movement as the FS decreases, as seen in Fig. 14, and this trend is similar and comparable between computed and measured movements. The results indicate that the correlations of hyperbolic transfer function parameters with N-values can potentially provide realistic estimation of the pile toe movement of bored pile equipped with pressure grouted fabric bulb in the toe region. The concurrence and consistency between measured and computed movement distributions may be attributed to the use clearly defined quality control parameters, such as pressure and volume control during installation as discussed above, adopted in the piling method. The 90% confidence limit for pile toe movement for various cases are also shown in Table 5.

GROUTING PRESSURE AND BULB SHAFT RESISTANCE

As noted in Fig. 10, the shaft resistance for the expanded fabric bulb is observed to be substantially higher than what may be expected in case of ordinary bored precast piles. It is expected that the development of high shaft resistance in the expanded fabric bulb results from the process of injecting cement slurry under pressure. To investigate the effect of pressure $f_p$ applied to expand the fabric bulb, two 19 m long and 400 mm diameter SC piles installed at the same site with soil condition shown in Fig. 15(a) but with different grouting pressure were compared based on loading test results. Details of the fabric bulb portions of the two piles are...
shown in Fig. 15(b), where the pressure \( f_p \) is noted to be 0.2 N/mm\(^2\) and 0.6 N/mm\(^2\) respectively. It may be noted in Fig. 15 that the diameter of enlarged boring in pile toe region where the fabric bulb had expanded, were kept slightly different.

Larger boring diameter of 800 mm was used for the case of lower grouting pressure of 0.2 N/mm\(^2\) and a slightly smaller diameter of 700 mm was used for case of higher grouting pressure of 0.6 N/mm\(^2\). This was out of concern to ensure adequate injection of the required volume of slurry for full expansion of the fabric bulb during installation. From experience with conventional bored precast piling practice, it may be expected that the increase in boring diameter tends to contribute to increased shaft resistance. Considering that the purpose of loading tests on the set of two piles described in Fig. 15 is to investigate the effect of difference in grouting pressure on bulb shaft resistance, slightly larger boring diameter in case of fabric bulb with lower pressure can be expected to contribute to reduction of the difference with higher shaft resistance to be expected in case of fabric bulb with higher grouting pressure. That is, the difference in bulb shaft resistance between the set of two piles described in Fig. 15 can be expected to be more, rather than less, compared to what may be observed from loading test results. Figure 15 also shows the volume injected and the diameter \( D_o \) computed based on the volume.

The strain gage instrumentation layout (1 to 5) for the two loading tests to investigate the effect of pressure on bulb shaft resistance is shown in Fig. 15. Again the SC piles were treated to have minimum shaft resistance as described above. Bulb shaft resistance and pile toe resistance characteristics for the two piles in Fig. 15 were obtained from the loading test data in a manner similar to the case of loading tests described above. The shaft resistance was obtained as the average over the interval of strain gage locations 1 and 3, and the toe resistance in this case corresponds to the strain gage location 1 (Fig. 15).

The variations of the shaft resistance of fabric bulb \( \tau \) and toe resistance \( q \) with \( S_o \) are shown in Fig. 16. While the effect of the two different pressure levels on the pile toe resistance \( q \) is difficult to discern in Fig. 16(b), the bulb shaft resistance \( \tau \) for the higher grouting pressure of 0.6 N/mm\(^2\) is clearly much larger than that for the lower grouting pressure of 0.2 N/mm\(^2\). This is a clear indication of the effect of pressure applied while expanding the fabric sack to form the bulb and explains why larger shaft resistance was observed in Fig. 10.

Attempt was made to approximate the bulb shaft resistance \( \tau \) for the two different values of \( f_p \) by fitting the hyperbolic transfer functions to the data, as described above in Fig. 8. Since there was no distinct difference between two values of \( f_p \) in case of pile toe bearing resistance \( q \), a single hyperbolic transfer function was fitted for both cases. The extent of fit to the data can be noted in Fig. 16. The ultimate shaft resistance given by \( 1/a \) of the hyperbolic transfer function for the fabric bulb with \( f_p = 0.6 \) N/mm\(^2\) is about 0.51 N/mm\(^2\) while that for \( f_p = 0.2 \) N/mm\(^2\) is about 0.40 N/mm\(^2\). This is an increase in bulb shaft resistance of about 28% for the increase in \( f_p \) from 0.2 to 0.6 N/mm\(^2\), which works out to be about 7% for every 0.1 N/mm\(^2\) increase in \( f_p \). Considering the effect of larger boring diameter for \( f_p = 0.2 \) N/mm\(^2\) case as discussed above, the actual increase may have been even higher.

To consider the results in another manner, extrapolation assuming linear relation between \( 1/a \) and \( f_p \) shows that \( 1/a \) at \( f_p = 0 \) would have been about 0.35 N/mm\(^2\). Now, the \( N \)-value averaged over the region between strain gage locations 1 and 3 for the test site shown in Fig. 15 works out to be about 35. From the relationship proposed by Hirayama et al. (1997) shown in Fig. 10(a) for comparison, \( 1/a \) can be estimated to be about 0.26 N/mm\(^2\) for \( N = 35 \). If the data utilized by them to obtain the relationship with \( N \)-values is assumed to correspond to \( f_p = 0 \), the value of \( 1/a \) obtained from linear extrapolation described above may be regarded as higher than expected. However, as mentioned above, the value of \( 1/a \) corresponding \( f_p = 0.2 \) N/mm\(^2\) may have in fact been enhanced by the use of larger boring diameter, which in turn results in a higher than expected estimation of \( 1/a \) at \( f_p = 0 \). Besides, the assumption of linear extrapolation may not be applicable and the relationship proposed by Hirayama et al. (1997) shown in Fig. 10(a) is limited to non-cohesive soils with \( N \)-values less than 30.

CONCLUSIONS

A systematic approach to developing a specific piling design method based on the correlation of hyperbolic transfer function parameters with average \( N \)-values has been proposed and its applicability to the bored precast piles equipped with expanded fabric bulb in toe region has been demonstrated. Although the measured to calculated ratios for the hyperbolic transfer function parameters show fairly wide variation, the dispersion narrows down considerably in the case of measured to calculated ratios for the ultimate resistance, as indicated by the relevant 90% confidence intervals. The data tends to follow lognormal distribution, as also observed for much larger sample sizes in previous studies.

At the ultimate level defined at pile toe movement of 10% pile bulb diameter, the vertical load bearing resistance of the fabric bulb calculated based on the
proposed method has 90% confidence of not exceeding the measured pile head load from loading test, and the 90% confidence interval for the measured to calculated ratio lies within a narrow band of 0.97 to 1.37. Application of a factor of safety (FS) of 3.0 to the mean value of ultimate resistance seems to translate into a reliability index $\beta$ of the order of 7.0, and even FS of as low as 2.0 seems to mean $\beta$ of about 4.0, higher than which is recommended for axial resistance of deep foundations based on static loading test results.

The distributions of calculated and measured pile toe movements at various FS levels compare well and show similar trends, indicating the validity of using the proposed approach to estimating the movement of pile bulb based on correlations with average $N$-values. The 90% confidence limits for measured movement of 8 mm and 10 mm compare well with those for computed movement of 9 mm and 11 mm at FS of 3.0 and 2.5 respectively. The 90% confidence intervals for measured and computed movements becomes 14 mm and 18 mm at FS of 2.0, indicating a larger difference between the two as the FS decreases.

It is confirmed that the shaft resistance of the enlarged fabric bulb portion is much larger compared to conventional bored precast piles and that this increase is a function of the pressure applied in injecting the cement slurry into the fabric sack to form the bulb. On the average, it appears that the increase in bulb shaft resistance is of the order of 7% for every 0.1 N/mm$^2$ increase in pressure.

Although the sample size utilized in this study is limited, small difference in 90% confidence limit for movement between FS of 3.0 and 2.5 combined with fairly high reliability index of about 5.4 at FS = 2.5 indicate the importance of rational judgment to be exercised in the design of the type of pile studies in this paper. When substantial shaft friction that may normally exist over the pile length above the enlarged bulb in toe region is considered, the computed nominal resistance obtained by the proposed method can be regarded as quite conservative.

Further investigation with larger sample size can provide further clarification and reconfirmation. Besides, the variability in and reliability of $N$-values utilized for correlation with transfer function parameters also needs to be considered for more conclusive results.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the suggestions and advices from Prof. Hideaki Kishida of Tokyo Science University during the preparation of the manuscript of this paper. The authors would also like to thank Mr. Hiroki Kobashi of the Geotechnical Development Department of Asahi Kasei Corporation for his help in the initial compilation and analysis of loading test data.

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