MONITORING OF LOAD DISTRIBUTION OF THE PILES OF A BRIDGE DURING AND AFTER CONSTRUCTION

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ABSTRACT

A temporary road bridge was constructed over the existing Hokuriku railway line at the Morimoto site, Kanazawa, Japan. Pile foundations were employed for abutments of the bridge. A rapid pile load test, called the Statnamic test, was conducted on one of the bored concrete piles of an abutment of the bridge. During the test, measurements of ground surface accelerations and noise level were performed at different distances away from the test pile. Affects of the Statnamic test on the surroundings were found to be small. One-dimensional stress-wave analysis of the Statnamic test was carried out to estimate the static load-settlement behaviour of the test pile and the profile of the soil properties. Some piles in two abutments of the bridge were instrumented with strain gauges to measure the axial load of each pile throughout the construction steps of the superstructure and during public use. A simplified analysis program PRAB was used to predict the load distribution and the settlements of the piles in the abutment using the profile of the soil properties obtained from the signal matching analysis of the Statnamic test signals. It was found that the field measurements agreed well with the predictions. Through the field measurements and the analyses, the safety of the pile foundation was confirmed.

Key words: load monitoring, pile, pile group, signal matching analysis, simplified deformation analysis, Statnamic pile load test (IGC: C7/C8/E4)

INTRODUCTION

New railways are under construction in Kanazawa City, Japan, for bullet trains (Hokuriku Shinkansen). In connection with the construction work, a temporary road bridge having a span of 42.5 m and a width of 10 m was constructed over the existing Hokuriku railway line in 2002 (see Fig. 1). Pile foundations were employed for abutments of the bridge. Although the bridge will be used only temporarily until the year of 2005, the number of car crossings per day is about 2000, and about 200 trains pass under the bridge in a day. Therefore, much attention was paid to the design of the pile foundation and load distribution of the piles during the construction of the bridge and during public use.

The Statnamic test, which is a rapid pile load test, was conducted on one of the constructed piles in the abutment A1 to confirm the load-settlement relation. During the Statnamic test, ground vibrations and noise level were also monitored. In order to estimate the static load-settlement curve of the test pile and the profile of the soil properties from the Statnamic test signals, a computer program KWave (Matsumoto and Takei, 1991) was used for one-dimensional stress-wave analysis (signal matching analysis) of the Statnamic test.

In the design process of the pile foundation, ignoring the group effects, it was assumed that all piles in the same row in the group would carry the same amount of the vertical load. However, the authors thought that uniform load distribution between the piles could not be obtained in practice. In this work, three piles in a 5 pile row in the abutments A1 and A2 were instrumented with strain

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gauges to monitor the axial load on each pile throughout the construction steps of the superstructure such as mount of bridge girders, construction of pavement, and construction of other facilities, as well as during public use. Using the profile of the soil properties obtained from the signal matching analysis of the Statnamic test signals, a simplified analytical program PRAB (Piled Raft Analysis with Batter piles) developed by Kitiyodom and Matsumoto (2002, 2003) was employed to predict the load distribution and the settlements of the piles during the construction of the bridge.

SITE DESCRIPTION

The construction site was located very close to the Hokuriku railway line near Morimoto railway station. The soil profiles at the construction sites of the abutments A1 and A2 are shown in Fig. 2. The Statnamic test was conducted on pile E in the abutment A1 (see Fig. 3). The soil profile at the test site is shown in Fig. 2(a). The top soft clay with a thickness of 2.8 m is underlain by a relatively uniform loose to medium sand layer of 14.5 m in thickness. A soft clay layer of 1.9 m in thickness is intercalated between the dense sand layer of 4.5 m in thickness and a hard gravel layer existing below a depth of 23.6 m. Figure 2(b) shows the soil profile at the abutment A2. A medium sand layer of 6.6 m in thickness underlies the top soft clay with a thickness of 3.3 m. A thick dense sand layer exists below the depth of 9.9 m.

The design of piles in the abutments was based on the
Table 1. Geometrical and mechanical properties of the piles

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length, $L$ (m)</td>
<td>Abutment A1 16</td>
</tr>
<tr>
<td></td>
<td>Abutment A2 10</td>
</tr>
<tr>
<td>Diameter, $d$ (m)</td>
<td></td>
</tr>
<tr>
<td>Cross-sectional area, $A$ (m²)</td>
<td>Total 0.332</td>
</tr>
<tr>
<td></td>
<td>Concrete 0.310</td>
</tr>
<tr>
<td></td>
<td>H-core 0.022</td>
</tr>
<tr>
<td>Young’s modulus, $E$ (kN/m²)</td>
<td>Concrete $3 \times 10^5$</td>
</tr>
<tr>
<td></td>
<td>H-core $20 \times 10^5$</td>
</tr>
<tr>
<td>Density, $\rho$ (ton/m³)</td>
<td>Concrete 1.760</td>
</tr>
<tr>
<td></td>
<td>H-core 7.865</td>
</tr>
<tr>
<td>Mass, $M$ (ton)</td>
<td>Total 11.48</td>
</tr>
<tr>
<td></td>
<td>Concrete 8.72</td>
</tr>
<tr>
<td></td>
<td>H-core 2.75</td>
</tr>
<tr>
<td>Velocity, $c$ (m/s)</td>
<td>Concrete 4129</td>
</tr>
<tr>
<td></td>
<td>H-core 5043</td>
</tr>
</tbody>
</table>

Section of H-core

Specifications for highway bridges by the Japan Road Association (2002).

A total of 20 piles with a pile length of 16 m and 10 piles with a pile length of 10 m were constructed for the abutments A1 and A2, respectively, at the site as shown in Fig. 3. In the abutment A1, the piles were embedded in the relatively uniform loose to medium sand layer having the mean SPT $N$-value of 10. On the other hand, the pile tips of the piles in the abutment A2 were embedded in the dense sand layer having the mean SPT $N$-value of 24. The piles were bored concrete piles with H-shaped steel reinforcing bars. The geometrical and mechanical properties of the piles are summarized in Table 1. Note that in the analyses described later in this paper, the pile was modelled as an equivalent homogeneous pile. For example, equivalent Young’s modulus was estimated as an average value of Young’s moduli of the steel and the concrete by weighting each sectional area. The same procedure was also used for the estimation of equivalent density.

The H-shaped steel bars were extended above the ground level by 6.1 m to support the superstructure. Hence, the load of the superstructure was transmitted to the piles directly through the extended H-shaped steel bars. As shown in Fig. 3, piles A, C and E in both abutments A1 and A2 were instrumented with strain gauges at a height of around 1 m from the ground surface to monitor the axial load of each pile throughout the construction steps of the superstructure and during public use.

Fig. 4. Measured Statnamic signals, together with signal matching result

STATNAMIC TEST

The Statnamic test was conducted on pile E in the abutment A1. During the test, the pile head was instrumented with a load cell and an accelerometer. A laser displacement transducer was employed to measure the settlement of the pile head. More details of the test equipments and test procedures can be found in Birmingham and Janes (1989). The Statnamic test signals are shown in Fig. 4. The measured pile head force, $F_{\text{sh}}$, increases smoothly with time and attains its peak of 2.2 MN and thereafter decreases smoothly to zero. The loading duration, $t_s$, is about 125 ms. It has been stated in JGS 1815–2002, Method for Rapid Load Test of Single Piles, Standards of Japanese Geotechnical Society for Vertical Load Test of Piles (The Japanese Geotechnical Society, 2002) that the relative loading duration, $T_r$, should be more than 5 in order to avoid wave propagation effects in the pile. The relative loading duration, $T_r$, is defined in Eq. (1).

$$T_r = t_s / (2L/c)$$

In this Statnamic test, the value of the relative loading duration, $T_r$, is equal to 17. As shown in Fig. 4, the pile
head displacement attains its peak at a time delayed behind the peak of the pile head force, which indicates that dynamic effects (i.e., inertia of the pile body and dynamic soil effect) are not negligible. The pile head velocity is less than 0.2 m/s, and the pile head acceleration is less than 15 m/s².

During the Statnamic test, in order to investigate the effects of the Statnamic test on the surroundings, measurements of ground surface accelerations and noise levels were conducted at different distances away from the test pile as shown in Fig. 5. Four accelerometers were placed on the ground surface at the distances of 14, 10, 7 and 5 m away from the test pile. Figure 6 shows the measured values of the ground surface accelerations during the Statnamic test. It can be clearly seen that the magnitude of the vertical acceleration decreases as the distance between the measured point and the test pile increases. Two peak accelerations can be found in each graph. The first one was observed during the launching of the reaction mass on the pile head, and the second one was observed during the landing of the launched mass. Noise levels during the Statnamic test was measured at two points, 10 and 15 m (points 1 and 2, see Fig. 5) away from the test pile. The heights of the measuring points were 0.8 m and 1.5 m, respectively. The noise levels were 103 dB at point 1 and 114 dB at point 2. The noise level when a train passed by was also measured to be 130 dB at point 2. Thus, the noise level during the Statnamic test was lower than that of the train. From the above results, it may be concluded that the effects of the Statnamic test on the surroundings are small.

In this study, a computer program KWAVE, developed by Matsumoto and Takei (1991), was used for the one-dimensional stress-wave analysis of the Statnamic test. The shaft resistance model (Fig. 7(a)) proposed by Randolph and Simons (1986) and the base resistance model (Fig. 7(b)) proposed by Deeks and Randolph (1993) were incorporated in KWAVE.

The values of the shaft spring, $k_s$, and the radiation damping, $c_r$, per unit shaft area were approximated based on the work of Novak et al. (1978) as follows:

$$k_s = \frac{2.75G_s}{\pi d}$$  \hspace{1cm} (2)

$$c_r = G_s / V_s$$  \hspace{1cm} (3)

where $G_s$ and $V_s$ are the shear modulus and the shear wave velocity of the surrounding soil respectively, and $d$ is the outer diameter of the pile.

The soil spring in the shaft model under static loading, $k_{s(\text{static})}$, which is lower than that under dynamic loading is estimated following Randolph and Wroth (1978).

$$k_{s(\text{static})} = \frac{2G_s}{\zeta d}$$  \hspace{1cm} (4)

Fig. 5. Points of acceleration and noise level measurements

Fig. 6. Measured ground surface accelerations at different distances away from the pile during the Statnamic test
Fig. 7. Rational soil models used in one-dimensional stress-wave propagation analysis

\( \zeta = \ln \left( \frac{r_w}{r_o} \right) \)  \hspace{1cm} (5)

\( r_w = 2.5L(1 - v_s) \)  \hspace{1cm} (6)

in which \( r_w \) is the outer pile radius, \( L \) the pile length, \( r_o \) the influential radius, and \( v_s \) is the Poisson's ratio of the soil.

The values of the soil spring at the pile base, \( k_b \), the damping, \( c_b \), and the lumped soil mass, \( m_b \), per unit base area can be estimated as follows (Deeks and Randolph, 1993):

\[ k_b = \frac{2G_s}{\pi d(1 - v_s)} \]  \hspace{1cm} (7)

\[ c_b = \frac{3.4}{\pi (1 - v_s)} G_s \]  \hspace{1cm} (8)

\[ m_b = 16r_o \rho_s \frac{0.1 - v_s^d}{\pi (1 - v_s)} \]  \hspace{1cm} (9)

in which \( \rho_s \) is the soil density.

Applicability of the soil resistance models shown in Fig. 7 to the analysis of a Statnamic (rapid) load test was examined by Matsumoto et al. (1999) through comparison of an one-dimensional stress-wave analysis and a dynamic FEM analysis of the Statnamic test load on a cast-in-situ concrete pile.

The one-dimensional stress-wave analysis was repeated with assumed values for the maximum shaft resistance, \( \tau_{\text{max}} \), and the base resistance, \( q_b \), using the measured \( F_{\text{sn}} \) as the force boundary condition at the pile head, until a good matching between the calculated and the measured pile head displacements was obtained. The viscous damping, \( c_s \), of the shaft resistance was assumed to be zero in this particular analysis, because the maximum value of the downward velocity of the pile measured in the Statnamic test was only about 0.1 m/s (see Fig. 4). Note that the relative velocity between the pile and the surrounding soil is certainly less than the pile velocity. Practically, the shear modulus of the soil at the low strain, \( G_0 \), is determined from the given shear wave velocity, \( V_s \), at the site. In this site only the profile of SPT \( N \)-value was available (see Fig. 2(a)). Hence, the shear wave velocity of the soil was estimated by means of an empirical equation as shown in Eq. (10) following Imai (1977).

\[ V_s = 91 \lambda_0^{0.337} \]  \hspace{1cm} (10)

in which the shear modulus of the soil at the low strain, \( G_0 \), can be calculated as follows

\[ G_0 = \rho_l V_s^2 \]  \hspace{1cm} (11)

where the measured soil density of 1.7 ton/m³ was used.

The profile of \( G_0 \) thus estimated was used as the initial assumption for the profile of \( G_s \) in the signal matching analysis of the Statnamic test. The profiles of \( G_s \), the distribution of the maximum shaft resistance, \( \tau_{\text{max}} \), and the base resistance, \( q_b \), were varied until a good matching between the calculated and the measured pile head displacements was obtained.

In the final matching analysis, the profile of the soil shear modulus, \( G_s \), shown in Fig. 8 was used. The pile head displacement obtained from the final matching is shown by the dashed line in Fig. 4. It was found that the values of the shear modulus of the soils, \( G_0 \), at the pile shaft and the pile base were 0.2 and 0.4 respectively of the shear moduli of the soils at the low strain, \( G_0 \), calculated using Eqs. (10) and (11). Yamashita et al. (1994) suggests that in the problem of pile foundation, the range of \( 0.1 G_0 < 0.3 G_0 \) is usually employed for the shear modulus of the soil. The values of the shear modulus obtained from the final matching analysis are comparable with their suggestion. The distributions of the corresponding shaft spring, \( k_s \), the corresponding shaft radiation damping, \( c_s \), and the maximum shaft resistance, \( \tau_{\text{max}} \), are also shown in the figure. The base spring value, \( k_b \), the base damping constant, \( c_b \), and the base resistance, \( q_b \), were 310 MN/m³, 561 kN/s/m³ and 1050 kN/m² respectively, in the final signal matching analysis. The static load-settlement relation was then estimated using the \( \tau_{\text{max}} \) and \( q_b \) identified from the signal matching analysis with the static shaft spring value, \( k_{\text{static}} \).

Figure 9 shows the load-settlement curve, \( F_{\text{sn}} \) vs \( u \),
measured in the Statnamic test. Figure 9 also shows the soil resistance $F_{\text{soil}}$ calculated by means of Eq. (12).

$$F_{\text{soil}} = F - M_p\alpha$$  \hspace{1cm} (12)

in which $M_p$ is the pile mass and $\alpha$ is the measured pile head acceleration. The soil resistance $F_{\text{soil}}$ is thought to be the sum of the static resistance and the dynamic resistance (Middendorp et al., 1992). In examining the characteristics of the static behaviour of the pile, the Unloading Point method revised by the Japanese Research Group on Rapid Pile Load Test Methods (Kusakabe and Matsumoto, 1995) is frequently applied in order to eliminate the dynamic effects from the observed behaviour of the pile. This method is based upon the hypothesis that the pile is a rigid mass sustained by a dashpot (dynamic resistance) and a spring (static resistance) of the ground, following Middendorp et al. (1992). The static load-settlement curve, $F_a$ vs $u$, as estimated by the Unloading Point method is also indicated in Fig. 9.

The derived static load-settlement curve, $F_{\text{static}}$ vs $u$, using the soil profiles identified from the signal matching analysis is also shown in Fig. 9. It is seen from this figure that there is a difference between the $F_a$ from the Unloading Point method and $F_{\text{static}}$ obtained from the signal matching analysis. According to Matsumoto et al. (2003),

the Unloading Point method does not perfectly diminish the dynamic effects in Statnamic testing. Hence, an overshooting behaviour is detected in the $F_a$ vs $u$. Note that in the next section, the soil parameters identified from the final signal matching analysis shown in Fig. 8 will be utilised to predict the behaviour of the piles in the abutments.

**MONITORING OF LOAD DISTRIBUTION OF THE PILES**

In a design process of the pile foundation, it was assumed that all piles in the same row in a group would carry the same amount of the vertical load. However, the authors views were that uniform load distribution of the piles could not be obtained in practice. Therefore, 3 piles in a row (piles A, C and E in the abutments A1 and A2, see Fig. 3) were instrumented with strain gauges. Two strain gauges were attached to both sides of the H-core in the vertical and horizontal directions in order to eliminate the vertical strain induced by change in the temperature. The settlement and the axial load of each pile were monitored throughout the construction steps of the superstructure and during public use.

In this study, the settlement and the axial load of each pile in the abutment A1 were also predicted using a simplified deformation analytical program PRAB that has been developed by Kitiyodom and Matsumoto (2002, 2003). This program is capable of estimating the deformation and load distribution of piled raft foundations subjected to vertical, lateral and moment loads, using a hybrid model in which the flexible raft is modelled as thin plates and the piles as elastic beams and the soil is treated as springs (Fig. 10). Pile-soil-pile, pile-soil-raft and raft-soil-raft interactions are taken into account based on Mindlin's solutions (Mindlin, 1936) for both vertical and lateral forces. When using PRAB to analyse the problem of a pile group, the soil resistance (soil spring value) at the raft base is set to zero. In the method, the considered soil profile may be homogeneous semi-infinite, arbitrarily layered and/or underlain by a rigid base stratum.
The vertical soil spring, \( K_{v} \), at the pile base nodes and the vertical soil springs, \( K_{z} \), at the pile shaft nodes are estimated using Eqs. (13) to (15), following Lee (1991).

\[
K_{v} = \frac{4G_{s}r_{s}}{1 - v_{s}} \times \left\{ \frac{1}{1 - \exp \left( -h^{*}/2r_{s} \right)} \right\}
\]

\[
K_{z} = \frac{2\pi G_{s}AL}{\ln \left( r_{m}/r_{o} \right)}
\]

\[
r_{m} = 2.5 \left[ \sum_{i=1}^{n} G_{i}L_{i}\sqrt{\frac{G_{m}}{G_{i}}} \chi_{i}(1 - v_{i})/G_{m}L_{i} \right],
\]

\[
\chi = 1 - \exp \left( 1 - h/L \right)
\]

where \( h \) is the finite soil depth, \( h^{*} \) the distance between the pile base and the rigid bed stratum, \( r_{o} \) the pile radius, \( \Delta L \) the pile segment length, \( L \) the pile length, \( G_{m} \) the maximum soil shear modulus and \( G_{i} \) is the shear modulus of the soil at the pile base. \( G_{i} \) and \( L_{i} \) are the shear modulus of the soil layer \( i \) and the length of pile embedded in soil layer \( i \), and \( n_{p} \) is the total number of soil layers along the pile embedment length.

The structure of the abutments was designed so that the load of the superstructure (i.e. bridge girders, pavement, and other facilities) as well as the live load carried by the bridge girders were directly applied to the front row of piles (piles A to E). In the analysis of the settlement and load distribution of each pile in the front row of piles in the abutment A1, the piles were modelled as a group of five piles with a cap. The profiles of the soil shear modulus, \( G_{s} \), the maximum shaft resistance, \( r_{max} \)

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Fig. 10. Plate-beam-spring modelling of a piled raft foundation (after Kitiyodom and Matsumoto, 2002, 2003)

Fig. 11. Load distributions of piles in abutment A1

<table>
<thead>
<tr>
<th>Step</th>
<th>Load at the point (kN)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>P1</td>
<td>P2</td>
</tr>
<tr>
<td>1</td>
<td>75.6</td>
<td>75.6</td>
</tr>
<tr>
<td>2</td>
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<td>3</td>
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<td>158</td>
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<tr>
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</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td></td>
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<tr>
<td>6</td>
<td>158-</td>
<td>168-</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
(see Fig. 8), and the base resistance, $q_b$, of 1050 kN/m$^2$ as used in the signal matching analysis were employed. The bending rigidity, $EI$, of the raft was set equal to the bending rigidity of the connected steel bars between each pile. Loads acting on the points P1 to P6 (see Fig. 3) at each construction step are summarised in Table 2. The bridge girders were separated into three pieces. In the first three construction steps, the field measurements were carried out after each piece of the bridge girders was mounted on the abutments of the bridge. The field measurements in step 4 were carried out after the completion of pavements and other facilities. Table 2 also shows the design loads, which are the sum of the Dead Load (DL) and the Live Load (LL), acting on the points P1 to P6 during public use. The field measurements from steps 5 to 11 were carried out during public use within five months after the opening of the bridge.

Figure 11 shows the comparisons between the load distributions of piles at each construction step and during public use calculated using PRAB and the monitoring values of piles A, C and E in the abutment A1. The calculated results using the simple beam method, in which the piles are assumed to be rigid bodies and the settlements of the piles are neglected, are also shown in the figure. It can be seen that both analysis results match well with the measured values at each construction step, and that during public use the measured values of the load in piles are well below the upper limit of the design load (Dead Load DL + Live Load LL). Note that the calculated results using the simple beam method agree well with the calculated results using PRAB. However, the settlements of the piles cannot be obtained by the simple beam method.

Figure 12 shows the settlements of the piles during construction calculated using PRAB. The settlements of the piles were also measured during the construction of the superstructure and during public use by a track level. As the precision of the track level was 2.5 mm, detailed measurements of the pile settlements were difficult. However, the measured pile settlements at the completion of the bridge and during public use did not exceed 2.5 mm as predicted. Figure 12 also shows that the settlements of the piles were well below the allowable settlement of 6.5 mm (1% of pile diameter).

Figure 13 shows the comparisons between the load distribution of piles at each construction step and during public use calculated using the simple beam method and the monitoring values of piles A, C and E in the abutment A2. It can be seen again that the analysis results match well with the measured values at each construction step, and that during public use the measured values of the load in piles are well below the upper limit of the design load (DL + LL).

From the above analysis results and the monitoring results, it can be concluded that all piles behave elastically under the working loads with a sufficient safety.

**CONCLUSIONS**

A temporary road bridge was constructed over the existing Hokuriku railway line at the Morimoto site, Kanazawa, Japan. A Statnamic test was conducted on a bored concrete pile in one of the abutments of the bridge.
The static load-settlement behaviour of the test pile and the profile of the soil properties were obtained through one-dimensional stress-wave analysis of the Statnamic test signals. The load distribution and the settlements of the piles in the abutments were measured throughout the construction steps and during public use. The field measurement values match well with the calculated results using a simplified analytical method PRAB. The load distribution and the settlements of the piles were found to be well below the limit values. Through the field measurements and the analyses, the safety of the pile foundation was confirmed.

This case study suggests that a combination of field measurements with an appropriate deformation analysis of a whole pile foundation structure may lead to a rational reduction of the safety factor.

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REFERENCES


