How to Improve Exchanges Between Academic Knowledge and Daily Practice?

September 20th, 2017
# List of Papers

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The role of monitoring pile behaviour to reduce gap between theory and practice

Le rôle du contrôle et suivi du comportement des pieux pour réduire l’écart entre la théorie et la pratique

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ABSTRACT: The effects of the installation technique on the bearing capacity and the load-settlement response of a single pile are discussed. The latter effect is shown to be less significant; a settlement controlled design is thus less dependent on the technological factors. Monitoring of the installation parameters shows some potential for controlling the pile response.


1. INTRODUCTION

It is long time that the scientific and technical communities is paying close attention to make more effective the link between theory and practice. A number of initiatives have been taken in the past to discuss how to reduce (to not say, how to close) the gap existing between academic and practitioners. Among the others, it is certainly worth of mention the great success that have had the ISSMGE global survey on State of the Art (SOA) and State of Practice (SOP) launched in 2017 by the combined efforts of the Technical Committees (TCs), the CAPG (Corporate Associates Presidential Group) and the TOC (Technical Oversight Committee). Some results have been made available on ISSMGE website at the section News on June 2017.

With reference to TC212 responses, this gap seems to be not decreasing. The reasons appear very similar to those already mentioned several years ago by Poulos (2003), summarized as follows:

- often, due to career advancement needs, people from academy focus their attention on refining problems which have been already treated with adequate thoroughness for practical purposes, instead of putting more efforts on issues of practical importance; in addition, the huge explosion in publications, often in a fragmentary form and in venues which are not readily accessible to the average practitioner, certainly does not help the process to converge;
- many practitioners are unwilling to abandon old (and often unsatisfactory) design methods and adopt more modern methods that have evolved from research; in addition, the use of innovative foundation systems (for instance, piled rafts) is prevented by over-conservative or over-prescriptive codes which the practitioner has to satisfy.

Thanks to the availability of powerful computers and numerical analysis tools, the researchers are now in the position to investigate very complex problems (geometry; loading conditions; material properties, etc.). It must be however reminded that “… all calculations, no matter how sophisticated and complex, cannot be more than rough approximation of the natural phenomenon they try to represents by means of a mathematical model …” (Candela, 1973). It follows that should be beneficial to check the outcomes of complex models and/or analyses with more simple methods, where possible; according to Bulleit (2008), it represents the only way to reduce model uncertainties and human errors.

In the writer’s knowledge, it is not so frequently done; consequently, the development of simple methods, which could be then used in routine design practice, is in some way prevented. Probably, this lack establishes a further obstacle between theory and practice.

On the contrary, when this methodological approach is followed, that obstacle is in some way removed, opening the door to a right exchange of knowledge from one world (theory) to the other one (practice).

Clearly, the undeniable statement by Candela (1973) makes priority to check the model/analysis reliability against experimental observations.

According to Burland et al. (1977), the benefits that derive from well documented case histories (they provide a list of all the vital information to be collected) are undeniable: “They provide the means of assessing the reliability of prediction methods, they give guidance to practitioners who are faced with the design of foundations and structures in similar circumstances, they can be used to develop an understanding of how structures interact with the ground and draw attention to weaknesses in design and construction. In short, well documented case studies provide the recorded precedents which are so valuable in developing the art of foundation engineering”.

In the following are reported some examples taken from the writer’s experience with reference to the response of single pile to axial loading.

The main goal is that of demonstrating that now are available very simple methods that, if properly calibrated against the experimental evidence, can give not only important suggestions for the design but also during the construction.

2. DESIGN AND CONTROL OF CFA PILES IN NAPLES AREA

2.1 Available knowledge

Mandolini et al. (2002) report the case of the construction of the foundations of some huge treatment plants in the south-west area of Naples. Based on the design considerations, 3300 CFA piles had to be installed, all with a length L=24m but with two different diameters d=0.60m and d=0.80m.

The area has been extensively investigated by n. 7 boreholes,
n. 17 CPT, n. 7 CPTU and n. 4 SCPTU; moreover, routine laboratory tests have been performed on 30 undisturbed samples.

Figure 1 reports a typical CPT profile: the cone resistance $q_c$ is of the order of few MPa (average value $q_c$=2.2MPa) in the upper 20 m from the ground surface (where alluvial soils of pyroclastic origin tightly interbedded with organic silt layers are found) and increases at greater depths where base formation of pozzolana is found (average value $q_c$=15.8MPa).

The groundwater table fluctuates between 1.2m and 1.6m below ground surface.

Before to start with the installation of the production piles, three load tests to failure have been carried out on trial CFA piles, all instrumented along the entire length to measure separately the shaft and the base contribution.

Figure 1 reports the observed load-settlement curves for the three piles. The main results are summarized in Table 1.

Table 1. Load test results (Mandolini et al., 2002).

<table>
<thead>
<tr>
<th>Pile</th>
<th>#1</th>
<th>#2</th>
<th>#3</th>
</tr>
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<tbody>
<tr>
<td>$L$ (m)</td>
<td>24.0</td>
<td>22.5</td>
<td>24.1</td>
</tr>
<tr>
<td>$d_b$ (m)</td>
<td>0.80</td>
<td>0.60</td>
<td>0.80</td>
</tr>
<tr>
<td>$d_s$ / $d_b$ (m)</td>
<td>0.83 / 0.84</td>
<td>0.61 / 0.61</td>
<td>0.85 / 0.94</td>
</tr>
<tr>
<td>$I_{eq}$ (%)</td>
<td>0.66</td>
<td>0.96</td>
<td>1.05</td>
</tr>
<tr>
<td>$I_{th}$ (%)</td>
<td>0.22</td>
<td>0.81</td>
<td>0.29</td>
</tr>
<tr>
<td>$q_{max}$ (MN)</td>
<td>4.08</td>
<td>3.26</td>
<td>5.30</td>
</tr>
<tr>
<td>$w_{max}$ (mm)</td>
<td>75.6</td>
<td>81.9</td>
<td>22.8</td>
</tr>
<tr>
<td>$S_{max}$ (MN)</td>
<td>2.81</td>
<td>2.59</td>
<td>3.94</td>
</tr>
<tr>
<td>$P_{max}$ (MN)</td>
<td>1.55</td>
<td>0.89</td>
<td>1.36</td>
</tr>
<tr>
<td>$q_{max}$ (kPa)</td>
<td>45</td>
<td>60</td>
<td>68</td>
</tr>
<tr>
<td>$q_{max}$ (MPa)</td>
<td>2.8</td>
<td>3.0</td>
<td>2.9</td>
</tr>
</tbody>
</table>

At that stage, it was quite surprisingly that two identical piles (#1 and #3, same length $L$=24m and nominal diameter $d_b$=0.80m, installed with the same pile equipment by the same operator) exhibited very different responses.

For pile #1, the maximum measured pile head settlement was $w_{max}=75.6$ mm ($\sim 9.5\% - d_b$), corresponding to an applied load $Q_{max}=4.08$ MN; for pile #3, the maximum measured pile head settlement was $w_{max}=22.8$ mm ($\sim 2.9\% - d_b$), corresponding to an applied load $Q_{max}=5.30$ MN.

These discrepancies were explained by the authors by means of the simple screw theory as adapted by Viggiani (1989, 1993) to consider the screw penetration in homogeneous soil.

Provided that during the construction of the piles, all the installation parameters during the penetration stage (rate of revolution $n$, rate of penetration $V_p$ and torque $M_T$) are recorded, it is possible to compare the local values of $V_p$ measured at any depth with the corresponding values of the critical rate of penetration $V_{Pcrit}$, defined by Viggiani (1989) as follows:

$$ V_{Pcrit} = n p \left[ (d - (d/d_s))^2 \right] $$

In Eq. (1), $p$ is the pitch of the screw, $d$ is the overall diameter of the auger (nominal); $d_b$ the outer diameter of the central hollow stem.

For a given screw ($p$, $d_b$ and $d_s$), if $V_p$ and $n$ satisfy Eq. (1), during penetration the displaced volume equals the removed volume and the soil surrounding the pile is not decompressed. If $V_p > V_{Pcrit}$ (or the velocity index $I_v = V_p/(V_{Pcrit}) > 1$) the removed volume is less than the displaced one (net compression effect, similar to that of a displacement pile); if $V_p < V_{Pcrit}$ ($I_v < 1$) the opposite is true (net decompression effect, similar to that of a non-displacement pile).

Figure 2 reports the comparison, along the pile depth, between the measured values of $V_p$ and the derived values for $V_{Pcrit}$ (thick lines) for piles #1 and #3.

Along most of the upper part of the pile shaft, crossing the alluvial soils (from the ground surface to a depth of about 20 m), the condition $I_v \geq 1$ is satisfied for pile #3 but not for pile #1. Within the base formation of pozzolana, on the contrary, $I_v < 1$; in that soil, the piles were thus installed essentially by boring.

During the extraction of the auger, concrete is pumped through the hollow stem at a prescribed rate $V_c$, while the auger is retrieved at a rate $V_R$. In a given time interval $\Delta t$, a volume of concrete $V_c = Q_c \Delta t$ is installed, while raising the auger leaves a nominal volume $(\pi d_b^2/4) \Delta t = (\pi (d_s / 2)^2/4) \Delta t$.

The ratio between the volume of concrete and the nominal volume is equal to 1.27-$Q_c/(d_s^2/2V_c)$; if it is above unity, the effect is a lateral compression of the soil and hence a better behaviour of the pile, but also over-consumption of concrete and cost increase ($d^\pm d_s$).

Provided that $Q_c$ and $V_c$ are recorded during the extraction stage, is then possible to estimate the local pile diameter at any depth.

Figure 3 reports the comparison, along the pile depth, between the estimated pile diameters and the nominal values ($d_b$=0.80m) for piles #1 and #3: the installed piles have diameters systematically greater than the nominal ones. Average values along the shaft ($d$) and at the base ($d_s$) are reported in Table 1.
Data from load tests were clearly interpreted with reference to the true geometries of the piles. The derived values for the unit shaft resistance $q_{\text{Smax}}$ and unit base resistance $q_{\text{Bmax}}$ are reported in Table 1.

![Figure 3. Estimated values of pile diameters along pile depth (data from Mandolini et al., 2002).](image)

Being the subsoil rather uniform, the remaining differences in behaviour among the piles are to be ascribed to differences in the installation details. The low unit shaft resistance of pile #1 is related to a penetration rate slower than the critical value (along the shaft $I_3$ averages 0.66x1, Table 1) determining an overall net decompression effect on the surrounding soil. On the contrary, during the installation of the other piles the rate of penetration was on average larger (for pile #3, along the shaft $I_3$ averages 1.05, Table 1), with a slight compression effect on the surrounding soil giving rise to larger unit shaft resistances. The unit base resistance for all the piles (but also the transfer curves, here not reported), once corrected for the actual base diameter, are practically coincident being equal the conditions of penetration of the auger ($I_3$=0.22 and 0.29 for pile #1 and #3, respectively).

It may be noted that, in the absence of monitoring of the installation parameters and hence without a correction of the diameter, the higher unit base resistance for pile #3 would have been probably interpreted as due to random soil variability.

It was then possible to establish a relation between the velocity indexes $I_3$ and $I_4$ and the coefficients $a_S$ and $a_B$ currently used in pile design methods based on CPT results ($q_S=a_S q_{\text{Smax}}; q_B=a_B q_{\text{Bmax}}$).

The results are shown in Figure 4; they clearly indicates a trend for $\alpha$ coefficients (hence, of the axial pile capacity) to increase as $I_4$ increases.

![Figure 4. Relationships between $\alpha$ and $I_4$ (from Mandolini et al., 2005).](image)

As it was to be expected, the larger is $I_4$ (either along the shaft or at the base), the larger is the corresponding coefficient $\alpha$. According to Mandolini et al. (2005), these findings confirm that the behaviour of CFA piles is influenced by the installation procedures.

Another important aspect is that related to the extraction stage, playing a significant role too. In fact, a proper graduation of concrete pumping rates can compensate soil loosening occurred in the penetration stage and improve the performance of the piles, by increasing the pile diameter along the shaft and/or at the base and the horizontal soil pressure on the shaft.

All the above findings suggest the possibility of moving from monitoring the installation parameters (just to know what has been done, thus “passive” attitude) to selecting the proper installation parameters in order to get the desired pile response in terms of axial capacity (“active” attitude). Clearly, in the “active” case, the achievable target depends on the combination of pile geometry, soil type and pile equipment.

### 2.2 Further developments

Modern piling engineering is always more frequently focused on settlement rather than capacity. It follows that the reliable estimation of the axial pile stiffness is becoming more and more vital.

A number of solutions are available in literature, starting from the pioneering works by Poulos and Davies (1968), Randolph (1977), Randolph and Wroth (1978).

Of particular interest is the further development of his method proposed by Randolph (1994) to consider the installation effect on the axial stiffness of a pile. The method is based on the following assumptions:

- a linear radial variation of the shear modulus in the region $d/2<r<R$ from a value $G_0$ adjacent to the pile shaft to the “undisturbed value” $G_0$;
- the external load applied to the pile is transmitted to the surrounding soil primarily by the shaft.

The second assumption is typically satisfied because piles are longer than the critical length $L_c$, which is the length beyond which any increase of the pile length causes little or no increase of the pile stiffness (Fleming et al., 1992).

The change of the axial pile stiffness may be quantified in terms of the parameter:

$$\zeta = \zeta_0 + \beta (R^* G^*/R')$$

where: $R^*$ is the extension of the disturbed zone; $G^*=G_0/G_0$ is the change of the soil stiffness; $\beta=(R^*-1)/(R^*G^*-1)$ is a coefficient representing the intensity of the disturb (for $R^*=1$ and $G^*=1$, $\zeta_0=\zeta_0$; $\zeta_0$ is a measure of radius of influence of pile, Fleming et al., 1992).

Based on some available experimental evidence (Van Weele, 1988; Peiffer & Van Impe, 1993; Viggiani, 1993; Mandolini (2003) found out that $G^*$ and $R^*$ may be expected to fall in the range 0.5 to 3 and 3 to 5, respectively. The range of values $G^*<1$ is representative of non-displacement piles, for which a lower soil stiffness in the zone immediately around the shaft may be expected; values of $G^*>1$, on the contrary, represent displacement piles.

Data from load tests has then been used to quantify the installation effects on the initial axial pile stiffness. In order to process the data in an objective and repeatable way, the initial axial pile stiffness $K_0$ has been evaluated as the initial tangent of a hyperbola fitted to the first three points on the experimental load-settlement curve. The following relationships are found:

$$R^*=3 \rightarrow G^*=0.094 +0.412 I_3$$
(3a)

$$R^*=5 \rightarrow G^*=0.123 +0.407 I_3$$
(3b)

As it can be seen, the influence exerted by the selected value for $R^*$ is limited: the derived values for $G^*$ are within a scatter of 10% and decreases as $I_3$ increases. Therefore, it does not represent a crucial choice for the following.

Looking at the data recorded from all the production piles, the average index velocity along the shaft ranges between 0.59 and 1.25 (slightly larger than that experienced with trial piles, see Table 1), yielding to $G^*$ approximately equal to 0.34 and 0.63.
respectively.

Assuming the undisturbed small strain stiffness for the soil $G_o(z)=21.2+2.7z$ (depth $z$ in meter $\rightarrow$ $G_o$ in MPa), these values lead to an estimated initial axial pile stiffness $K_o$ for a pile with a length $L=24.0$ m and diameter $d=0.80$ m ranging between 438 MN/m and 717 MN/m.

The expected values for $K_o$ was then compared with those derived from 14 load tests on production piles (Figure 5), which gave the following results: $K_{o_{\text{max}}}=1050$ MN/m; $K_{o_{\text{min}}}=591$ MN/m; $K_{o_{\text{avg}}}=710$ MN/m with a standard deviation equal to 133 MN/m and a coefficient of variation equal to 18%.

It is worth of mention that the minimum predicted value is only 15% smaller than the measured minimum value and that the maximum predicted value is practically coincident with the average measured value.

2.3 A possible integrated use of the data on site

In the previous sections, it has been shown that, for a CFA pile with a known geometry (not necessarily the nominal one), the index of velocity $I_V$ plays a major role in determining their expected axial capacity and initial axial stiffness.

At a given site where, as for the case here discussed, CPT data are available and installation parameters are recorded, it becomes possible to establish direct correlations between the latter and the expected performance.

In particular, for any pile is possible to predict in a very simple way the entire load-settlement curve, for instance assuming a hyperbola. So doing, it is possible to identify, in a reasonably way, that pile (or those piles) for which the worst performance is expected. In this way, it could be possible to select that pile for a load test (rational approach) instead of relying on luck.

![Figure 5. Experimental and expected load-settlement curves.](image-url)

In Figure 5 the predicted load-settlement curves in the worst and in the best cases ($I_{V1}$, $I_{V2}$, $d_5$ and $d_9$) are compared with those experimentally observed during 14 load tests on production pile (the maximum load test was 1.2 MN for 7 piles and 1.6 MN for the remaining 7 piles). As it can be seen that 12 out of 14 curves fall within the prediction range; 2 piles behaved better than expected but none worse (and it is the most important result).

3. CONCLUSION

Although who is writing certainly belongs to the “academic part of the world”, he is decidedly convinced that the wall separating theory and practice is ready to collapse.

In the paper it was tried to demonstrate that, for a given problem, as for instance that of CFA piles, a competent use of (simple but not simplistic) theories (for instance, Viggiani, 1989; 1993) and methods (for instance, Randolph, 1994) is of great help for clarifying which parameters play a major role in determining their response to axial loading.

The availability of well documented case histories, in the sense proposed by Burland et al. (1977), makes possible to check their capability to catch the main aspects and, if the case, to proceed with their calibration for a defined goal.

4. REFERENCES


The importance of site investigations and quality control in pile installation.

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ABSTRACT: In this paper, gap between the theoretical prediction and the actual field test results were investigated through full-scale prebored and precast pile loading test. The effect of the relatively complex installation process and the quality control of prebored and precast pile on capacity and the behavior was discussed. The load-settlement curve of the prebored and precast pile is obtained from 20 field loading tests, and the effect of the installation process and quality control is observed through additional site investigation after the loading test. Based on the pile loading test and additional site investigation, the behavior of the pile due to axial loading was defined into four categories; 1) normal, 2) excessive skin friction, 3) slime and 4) high skin friction with slime. Through further analysis, it was concluded that the installation process and quality control during installation played a critical role in the settlement and the capacity of the prebored and precast pile. Moreover, the quality control during installation also had an influence on the behavior of prebored and precast pile.

KEYWORDS: Prebored and precast pile, site investigation, Single Pile, Axial Capacity, Load-settlement curve, Borehole imaging profile system (BIPS).

1. INTRODUCTION

The vibration and noise induced during installation have become a significant consideration. For this reason, various construction projects, such as urban area construction or highway construction, are substituting driven piles with prebored and precast piles.

A prebored and precast pile is installed by boring a hole in the ground and placing the precast PHC (Pretentioned Spun High Strength Concrete) or steel pile in the borehole, and finished by casting cement milk around the pile. The preboring process reduces the noise and vibration significantly compared to the driven piles, and is more cost-effective compared to drill shafts. The quality of a prebored and precast pile depends highly on the workmanship of the technicians and relatively complex installation process. For this reason, the full mobilization of the skin friction and the end bearing condition may not be accomplished. And due to these uncertainties, it is difficult to predict the behavior and the settlement of the pile due to various uncertainties. Moreover, compared to driven piles and drilled shafts, very few studies, let alone full-scale tests on prebored and precast pile have been performed.

The main source of the bearing capacity of prebored and precast pile, as well as other type of pile foundation, is the skin friction and the end bearing capacity. However, the proportion of the skin friction in prebored and precast pile capacity is significantly higher compared to other piling methods. The main source of the skin friction is the cement milk around the precast pile. The skin friction of the pile was reported to increase 700% as the cement milk was hardened and the proportion of the skin friction to the total bearing capacity also increased up to 400% (Park, 2004).

In this paper, the gap between the theoretical approach (ideal case) and the actual pile behavior in the field will be displayed based on the full-scale pile loading test. The effect of installation process and quality control of prebored and precast pile will be the main discussion point. Among the 20 loading test results, representative test results, which shows four different load-settlement behavior, were analyzed based on the site investigation and borehole imaging profile system (BIPS) conducted after the pile loading test. Moreover, the capacity estimation using the empirical method was compared with the actual field test results.

2. FULL-SCALE PILE LOADING TEST IN KOREA

2.1 1st Site investigation and pile installation

The full-scale pile loading test was carried out on southern region of the Korean peninsula. Site investigation was carried out prior to the pile installation. The ground profile and properties were obtained by boring three boreholes near the planned pile installation point. After a series of site investigations and laboratory tests, the ground conditions was shown to reflect the typical soil condition of the Korean peninsula, consisting of land fill, a sedimentary layer, weathered soil and weathered rock layers.

The diameter of the steel pile used in the loading test was 0.508m, and the thickness was 0.012m. The strain gauges reading the settlement and the load-transfer curve of the test pile were installed at the surface of the test pile and the spacing between the gauges was 0.5m. The gauges were placed on two side of the pile to prevent loss of data due to damage during installation and load test. The length and the end bearing condition (rock type and N-value) of the four test piles are shown in Table 1.

2.2 Theoretical prediction

Prior to the actual pile loading test, theoretical (ideal case) prediction of the pile behavior and pile capacity were carried out. The prediction of the pile load-settlement curve was based on the
3D FE analysis program ABAQUS CAE 6.13 (2013). The modelling used for analysis is shown in figure 1. The 3D mesh was modelled based on an ideal installation condition of the prebored and precast pile, with a steel pile, cement milk layer and a cement milk inside the steel pile. The cement milk layer in the shaft of the pile was modelled to 0.1m, and the cement milk layer in the pile was modelled to 4D + 1m, where D is the diameter of the steel pile. The interface between the cement milk and the steel pile was modelled as a “hard contact”. However, the interface between the cement milk and the surrounding soil was modelled as a slip model considering the relative displacement, using the user-subroutine “FRIC”. The capacity of the prebored and precast pile was estimated based on the Korean Land and Housing Corporation structural design manual (Korean Road and Bridge Corporation, 2008), Korean Structure and Foundation Design Standards (Korean Geotechnical Society, 2015), and the Japanese Road and Bridge Structure Design Manual (Japan Road Association, 1996), and they are summarized in Table 2.

Table 1. Pile installation condition (designed)

<table>
<thead>
<tr>
<th>Method</th>
<th>Unit Ultimate Bearing Capacity (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>End Bearing</td>
<td>Skin Friction</td>
</tr>
<tr>
<td>Korean road and bridge structure design manual (2008)</td>
<td>200N (N≤60)</td>
</tr>
<tr>
<td>Korean land and housing corporation manual (2008)</td>
<td>250N (N≤60)</td>
</tr>
<tr>
<td>Japanese road and bridge structure design manual (1996)</td>
<td>100N² (N≤60)</td>
</tr>
</tbody>
</table>

*W.R : Weathered Rock

Table 2. Empirical solutions for estimating ultimate bearing capacity for prebored and precast piles (for sandy soil)

<table>
<thead>
<tr>
<th>Method</th>
<th>Unit Ultimate Bearing Capacity (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>End Bearing</td>
<td>Skin Friction</td>
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<tr>
<td>Korean road and bridge structure design manual (2008)</td>
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<td>Korean land and housing corporation manual (2008)</td>
<td>250N (N≤60)</td>
</tr>
<tr>
<td>Japanese road and bridge structure design manual (1996)</td>
<td>100N² (N≤60)</td>
</tr>
</tbody>
</table>

*Allowable bearing capacity = (Ultimate bearing capacity)/3

The material properties used in the analysis is based on the site investigation and laboratory tests (Table 3).

Table 3. Material properties used in analysis

<table>
<thead>
<tr>
<th>Physical properties</th>
<th>Model</th>
<th>$v$ (kN/m²)</th>
<th>$E$ (MPa)</th>
<th>$e$ (kPa)</th>
<th>$\phi$ (°)</th>
<th>$c$</th>
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</thead>
<tbody>
<tr>
<td>Pile</td>
<td>Linear elastic</td>
<td>75</td>
<td>200,000</td>
<td>0.2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Cement milk</td>
<td>Linear elastic</td>
<td>20</td>
<td>5,000</td>
<td>0.25</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Fill</td>
<td>Mohr Coulomb</td>
<td>17</td>
<td>10</td>
<td>0.30</td>
<td>0</td>
<td>29</td>
</tr>
</tbody>
</table>

2.3 Pile loading test

A standard static loading test was carried out after the site investigation. Based on the site investigation, the base of the pile was installed under various N-value conditions. The procedure of the standard static loading test was based on the ASTMD1143-81(1994). The period of the loading was based on four steps by loading 25%, 50% and 75%, sustaining the load until the settlement of the head was less than 0.25mm per hour (maximum of two hours). After the completion of the loading, the unloading process was carried out by unloading 50% over 20 minutes. The yielding load was assumed to be reached when a sudden settlement occurred during the loading process, and the test had come to an end.

Total of 20 loading test cases were carried out following the procedure stated above, and the load-settlement curve and the
The skin friction is a major factor which affects the behavior and the bearing capacity of the prebored and precast pile. For this reason, an investigation on the side of the pile was additionally conducted. Borehole imaging profile system (BIPS) takes photographs of the boresholes from the inside. The BIPS photographs can indicate whether the cement milk poured in to the borehole in the installation process penetrates in to the surrounding ground, forming an excessive bonding layer between the cement milk layer and the ground, which can cause an abnormally high skin friction.

The results of the 2nd site investigation including BIPS is shown in figure 4 and table 4. The investigation results showed that pile #1, which show a similar behavior between the predicted and actual loading test result showed no difference in end bearing condition and no abnormal layer causing excessive high skin friction was indicated. However, for piles #2, #3 and #4 different conditions were shown through additional site investigation. Pile #2, which showed excessively high skin friction was found to have an abnormal layer in the shaft of the pile, and brittle yielding behavior in the skin friction. Through BIPS photos, it was found that cement milk poured around the pile infiltrated in to the surrounding ground, forming a soil-cement layer (Reddy et al., 1993). The BIPS photo of the cement milk infiltrated into the surrounding ground is shown in figure 5. As shown in the figure, a soil-cement like layer is formed around the pile. Pile #3, which showed a dramatic settlement after the yielding of the skin friction, was found to have an extremely weak slime layer in the toe of the pile. As for pile #4, which showed a brittle yielding behavior after the significantly high skin friction and rapid settlement, was found to have an unpredicted layer in the toe and the shaft of the pile. The N-value of the significantly weak unpredicted layer showed an extremely weak value, 13/30, compared to the targeted pile toe condition.

Through analysis of the pile behavior and the additional site investigation, it can be concluded that the toe and the shaft of the pile showed different conditions compared to the designed conditions, and this was the cause of the gap between the predicted and actual load-settlement behavior.

Based on the 2nd site investigation, a modified analysis model considering the actual site condition was established, considering the cement milk infiltrated layer around the shaft and the different pile toe condition.

The load transfer curve was obtained based on the loading tests. However, after a careful evaluation of the test outputs, it was shown that the loading test results showed various tendencies in the load-settlement curve despite their similar soil profile and pile installation conditions. In addition, the bearing capacity of the test piles were also differed significantly. After a thorough analysis of the loading test results based on the load-transfer curve, it was found that there were four apparently different failure mechanisms. Figure 3 shows the four representative test results. The bearing capacity of the test piles were estimated by using the load-settlement curve from the loading tests and the Davison offset method (1973).

Pile #1 represents a case where a test results are identical to the prediction outputs. Pile #2 showed significantly stiff resistance in the skin friction region and showed a brittle yielding of the skin friction. Pile #3 showed normal skin friction resistance, but the end bearing resistance was severely low, causing a sudden settlement of the pile. Pile #4 showed extremely high skin friction and after the brittle yielding of the skin friction, sudden settlement was also observed.

The bearing capacity of the test piles also varied along with the behavior of the pile. While pile #1 showed relatively similar bearing capacity between predicted capacity and capacity based on field test. However, unlike pile #1, the bearing capacity of pile #2, #3 and #4 differs significantly with the predicted results.

By comparing the theoretical solution and the actual behavior based on the loading test, the gap between the academia and practice was clearly displayed. To clarify the cause of this difference, 2nd site investigation was conducted along with the borehole imaging profile system (BIPS).

2.4 2nd soil investigation after the loading test

After observing the considerable margin between the predicted load-settlement curve and the bearing capacity with the actual load-settlement curve and the bearing capacity, 2nd soil investigation was conducted right after the loading test.

The post-loading test soil investigation was carried out by boring the cement milk casted inside the steel pile. The boring process was continued until the boring machine reaches the toe of the pile, or meets an unpredicted layer - such as slime, soil plugged into the pile or a severely disturbed layer. After the boring process, the strength of the pile toe, or the unpredicted layer was estimated by measuring the N-value. Then the boring process was additionally carried out to measure the thickness of the unpredicted layer.

<table>
<thead>
<tr>
<th>Pile</th>
<th>#1</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
</tr>
</thead>
<tbody>
<tr>
<td>L (m)</td>
<td>10</td>
<td>10</td>
<td>11</td>
<td>9.6</td>
</tr>
<tr>
<td>End condition</td>
<td>W.R</td>
<td>W.R</td>
<td>Slime</td>
<td>Disturbed</td>
</tr>
<tr>
<td>N-Value</td>
<td>50/3</td>
<td>50/3</td>
<td>3/30</td>
<td>13/30</td>
</tr>
<tr>
<td>Slime layer (m)</td>
<td>-</td>
<td>-</td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Skin condition</td>
<td>Normal</td>
<td>Cementation</td>
<td>Normal</td>
<td>Cementation</td>
</tr>
</tbody>
</table>

![Figure 3. Actual load-settlement curve and bearing capacity](image-url)

![Figure 4. Schematic of the unpredicted layer](image-url)
2.5 Numerical analysis considering 2nd site investigation

After the 2nd site investigation, a modified analysis model considering the actual ground condition was modelled. The modified analysis model was modelled to consider the cement milk infiltrated into the surrounding ground and the unpredictable layers in the toe of the pile. In the shaft of the pile, the bonding between the pile and the cement milk layer was modelled as a “hard contact” based on the ABAQUS user-subroutine “FRIC”. The interface between the cement milk layer and the infiltrated cement milk layer was modelled as an interface t-z curve allowing the relative displacement between two layers and to simulate the brittle yielding of the skin friction using the “FRIC” function. However, in the case of normal shaft resistance, the t-z curve was modelled to consider the relative displacement only (figure 6 (a)). The maximum skin friction \( f_{\text{max}} \) is based on the following equation (1).

\[
 f_{\text{max}} = \mu' = \tan \delta \times K_0 \times \sigma_v'
\]  

The relative displacement (②) was set to 5~8mm for soils, and 2-5mm for weathered rock layer based on field load-transfer curve and literature reviews (Broms, 1979; Jeong et al., 2004). For the brittle skin friction failure condition, the peak resistance (③) was modelled as 50% of the shear strength of cement milk, and the residual strength after the brittle failure (④) was modelled identical to the maximum skin friction (①). The relative displacement of brittle yielding (④) was set to 1~3mm based on field load-transfer curve and literature reviews, and this is shown in figure 6 (Jeong et al., 2004; Jeong et al., 2010).

The end bearing condition of the modified model also considered the actual field condition based on the 2nd site investigation. Additional layer was modelled in the toe of the pile, and the properties reflecting the slime and the disturbed layer was considered.

Figure 7 shows the result of the numerical analysis using the modified model of four representative piles. The analysis results shows significantly different results compared to the previous analysis model, and converges with the actual loading tests. This results show that only by considering the actual field conditions rather than the theoretical or design condition, and by reflecting the unique interface behavior, the numerically analyzed results can converge with the actual loading test results.

3. CONCLUSION

In this paper, the gap between the theoretical estimation (academia part) and the actual field measurement (practice) was discussed based on a prebored and precast pile loading test results. Although the theoretical prediction can provide a guidance in the design process, the actual behavior of the pile in practice can show considerable gap due to disturbance during installation, uncertainties in site conditions and abnormalities due to unpredicted interactions. For this reason, to estimate the actual behavior of the prebored and precast pile it is necessary to consider the various unpredicted effects which can occur during installation.

Initiation of the designing and planning can be achieved based on the theoretical approach prior to the actual execution of the tests or construction. However, to accurately estimate and analyze the actual behavior, additional investigation of the changed field condition after the installation is necessary.

4. ACKNOWLEDGEMENTS

This work was supported by the National Research Foundation of Korea (NRF) grant funded by the Korea government (MSIP) (No. 2011-0030040) and the Korea Expressway Corporation Research Institute, and we express our gratitude for their support.
5. REFERENCES

ASTM D1143-87 (2009), Standard Test Method for Piles Under Static Axial Compressive Load


Korea Expressway Corporation. (2012), "Expressway construction guide specification".

Korean Land and Housing Corporation Structural Design Manual (2008), Korean Land and Housing Corporation, Jinju, Korea.


A simplified analysis of mega strip foundation on piles subjected to horizontal earthquake

UNE ANALYSE SIMPLIFIÉE DE LA BANDE DE FONDATION MÉGA SUR PILOTIS SOUMIS AU TREMBLEMENT DE TERRE horizontal

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ABSTRACT: A simplified analysis for seismic responses of mega strip foundation on piles under the horizontal earthquake excitations is suggested in this study. Ignoring vertical, rocking and torsional motions, one dimensional wave equation for horizontal structural responses of the strip foundation is solved using the finite difference formulas. The forces resulted from the superstructure, the piles and the soils are modeled using appropriate springs. The proposed solution was found agreeable with the three dimensional finite element analysis and its computational time is much less to provide efficiency in the preliminary design stage.

RÉSUMÉ: Une analyse simplifiée pour les réponses sismiques de la fondation filante sur pieux sous les excitations du séisme horizontal suggérées dans cette étude. En ignorant les mouvements verticaux, basculants et de torsion, une équation d’onde dimensionnelle des réponses structurales horizontales de la fondation filante est résolue en utilisant des formules de différence finie. Les forces résultent de la superstructure, les pieux et les sols sont modélisés en utilisant des ressorts appropriés. Avec l’analyse de méthode des éléments finis tridimensionnels et la réduction de son temps de calcul qui fournit l’efficacité dans la phase de conception préliminaire, la solution proposée était considérée acceptable.

KEYWORDS: strip foundation, piles, seismic responses, horizontal earthquake.

1 INTRODUCTION

The methods used in design and analysis of piled raft foundation have been suggested for years (Poulos, 1991 and 2001; Clancy and Randolph, 1993; Katzenbach, 1993; Randolph, 1994; Yamashita et al., 1994; Horikoshi and Randolph, 1996; Kobayashi et al., 2009). They were categorized as, 1. simplified calculation methods, 2. approxima computer-based methods, and 3. rigorous computer-based methods. All these methods can provide rational solutions to different levels of design requirements.

At modern time, three dimensional (3D) FEM analysis is the most rigorous approach. The approximate numerical analysis can provide effective solutions as well. Since the computational time of such type solution is much less than the 3D FEM analysis, the approximate computer-based method could provide a useful tool to the performance based design (PBD), in which a large amount of computations can be carried out on the variability of the design parameters. A series of study (Kitiyodom and Matsumoto, 2002; Kitiyodom et al., 2005) on 3D approximate computer-based methods has been suggested for static and dynamic analyses of piled raft foundation. These analyses were conducted solving the equations of motion at the nodes of the discrete raft. The piles and soils connecting to the slab were modeled by springs and dashpots. These analyses are applicable to the piled raft foundation where the loads are mounted on top of the foundation. For seismic responses of the foundation due the ground excitations, advanced solutions need to be made.

Therefore a simplified modeling on the horizontal seismic responses of piled raft foundation was suggested (Chang et al., 2016). The approximate analysis was suggested solving the differential equation derived from the force equilibrium of the raft with the central difference formulas. The transmitting loads from the underneath soil-pile elements due the ground motions and the superstructure on top of the raft were considered. The suggestion and an application is presented in this paper.

2 NUMERICAL MODELLING

Figure 1 shows the schematic layout of the uncoupled motions for the spread raft of a piled raft foundation. The displacements in x, y and z directions are denoted as u, v and w. Rotations along these axes are assumed negligible. For horizontal earthquake shaking in the x direction, the governing differential equation for the raft can be derived based on force equilibrium conditions of the raft.

![Figure 1](image-url)

Figure 1 Layout of the uncoupled motions of a piled raft foundation, (a) uncoupled motions and horizontal impact with a bevel angle (b) force equilibrium diagram

\[ E A \frac{d^2 y}{dx^2} dx = \rho A dx \frac{d^2 y}{dt^2} + k_{ep} (u - u_p) + k_{ep} (u - u_p) + k_{p} (1 - R) u + m_{p} R \frac{d^2 y}{dt^2} \]

where \( u \) = displacement of the raft; \( u_p \) = displacement of the ground soil underneath the raft; \( u_p \) = displacement of the equivalent pier (pile-soil-pile system) underneath the raft; \( E = \ldots \]
Young’s modulus of the raft; $A = \text{cross-section area of the raft}; \rho = \text{mass density of the raft}; k_{2D} = \text{spring constant of the soils underneath the raft (units in Force/Length)}; k_{eq} = \text{spring constant of pile-soil-pile system underneath the raft (units in Force/Length)}$ which can be calculated as $k_{eq} = k_{p}n + k_{s}s$, where $k_{p} = \text{stiffness of single pile}; k_{s} = \text{stiffness of the soils in equivalent pier}; n = \text{number of piles}; A = \text{area of the soils in equivalent pier}; k_{st} = \text{stiffness of the superstructure (units in Force/Length)}$; $R = \text{ratio of the superstructure displacement divided by the raft displacement (i.e., } R = u_{st}/u_{fd})$, $m_{st} = \text{mass of the superstructure}$. Notice that the ratios are assumed the same for the displacements and the accelerations.

In this study, $k_{p}$ is simply treated as shear spring constant, i.e., $G_{soil}$, where $G_{s} = \text{shear modulus of the soil}; A_{soil} = \text{contact area of the soils outside the pile-soil-pile elements}; l_{soil} = \text{thickness of the soil}$. The stiffness parameters $k_{p}$ and $k_{s}$ can be computed from shear springs too. In that case, $k_{p} = G_{soil}l_{soil}$ where $G_{p}$, $A_{p}$, and $l_{p}$ are the shear modulus, cross-section area, and the length of the pile, respectively; $k_{s} = G_{soil}l_{soil} + G_{soil}A_{soil}/l_{soil}$ where the subscripts $s$ and $l$ respectively denote for concrete structure and material inside the concrete structure. Notice that in Eq. 1, the viscous forces resulted from the superstructure and the soils underneath the raft are ignored. Using the central difference formulas, Eq. 1 can be solved easily by independent equations (Chang et al., 2016). Similarly, the differential equation for the motion (displacement of $v$) of the slab due to horizontal ground motion in $y$ direction can be presented in the same manner differentiating the variable $v$ with respect to $y$.

For lateral boundaries of the raft, free tractions were considered. Alternate equations can be achieved for the cases where no superstructure and no underneath pile and soil elements are encountered. To solve for the raft displacements, it was assumed that foundation is initially at rest. Time dependent raft displacements are thus obtained in an explicit manner. For horizontal seismic ground acceleration, $a(t)$ acting to the foundation with a bevel angle of $\theta$ as shown in Figure 1(a), the analysis can be conducted independently taking into account of the acceleration’s components $a_{x}$ and $a_{y}$ in each direction. The absolute displacements of the raft in the direction of the causative ground acceleration could be calculated as $(u_{st}+\omega v_{st})t^{3}$; displacement time history of the foundation can be also converted from the displacement components and averaging them to yield the solution. The above analysis was termed as EQPR (Earthquake analysis for Piled Raft foundation) (Chang et al., 2016).

2.2 Responses of pile-soil-pile elements

The time-dependent displacement functions of the pile-soil-pile elements underneath the raft due to horizontal ground motions can be analyzed using the EQWEAP procedure (Chang et al., 2014). In the first step, the linear and/or nonlinear free-field ground responses are able to obtain using the lumped mass analysis assuming that the ground is composed by horizontal soil layers. Secondly, the ground responses are applied to the discrete wave equations of the pile elements in order to solve for the corresponding pile displacements. Figure 2 illustrates the schematic layout of the EQWEAP procedure and the equilibriums of the pile segments used in the analysis. Notice that both the soil and pile nonlinearities can be modeled using proper material laws.

This solution was suggested in the past years and it was found reliable in comparison with the FEM analysis and pseudo static solution in matching the field observations (Chang et al., 2014; Chang et al., 2016). Although the EQWEAP analysis is suggested for single piles, with proper calculations of the load distributions while the effects of pile-to-pile interactions were included (Chang et al., 2009), this analysis can be used to monitor any single pile response within a pile group. In general, the piles were found moving accordingly with the ground motions. The differences between them are able to neglect. In applying the EQWEAP analysis into the piled raft foundation problem, it is suggested to obtain the response of the pile-soil-pile elements (or the equivalent pier), $a_{p}$ in connection with the raft, as $u_{eq} = (u_{st} \sum A_{p} + \omega v_{st} \sum A_{p})/(\sum A_{p} + \sum A_{s})$ in the above equation, $u_{eq}$ is time-dependent displacement function of the single piles; $u_{st}$ is free-field response function of the surface ground soil; $\sum A_{p}$ is the total cross-section area of the piles and $\sum A_{s}$ is the total area of surface soils in the pile-soil-pile elements. This would help to calculate the seismic ground force.

3 EXAMPLE AND VALIDATION

Assuming that a strip concrete slab with dimensions $L \times B \times H = 300m \times 60m \times 2m$ is allocated at the surface of a ground site consisting of soft soils whose thickness is $13m$ and underlain by gravels. Five massive superstructures are evenly mounted on the slab. For each one of them, 81 concrete piles with pile diameter of 2m and pile length of 28m, oriented in a ring shape with radial distance at 7, 14, 21 and 26 meters from the central pile (see Figure 3) are installed under the slab of $60m \times 60m$ to support the superstructures. In addition at each corner of the slab, three piles were seating in a triangular shape.

As a result, each superstructure is supported by 93 concrete piles under the slab. Total number of the piles would be 465. Material properties and model parameters used in the proposed analysis and the 3D FEM modeling using Midas-GTS program
Seismic accelerations recorded at the TAP052 station in EW direction during the 1999 Chi-Chi earthquake was taken as the input of ground motion. Figure 4 shows the acceleration records obtained by a calibrated one based upon the designed Peak Ground Acceleration (PGA) at 0.24g and the alternative one fitting acceleration record with the designed spectrum under the same level of PGA. It can be seen that although the time-dependent accelerations are in similar forms, the resulting response spectra are very different. The formation of an artificial earthquake is very important. Figure 4(b) and 4(c) were obtained after baseline corrections.

The analysis is then conducted with the input seismic motions using the first method. Horizontal seismic motion is assumed independently in the longitudinal and transverse directions of the foundation whereas the bevel angle is kept at 0° and 90°. For analysis in x direction, seven nodes along the raft are analyzed. For analysis in y direction, three nodes are computed. Notice that time increment used in the proposed analysis is 0.0005 sec to ensure stability of the solutions (the original data has time increment of 0.005 sec). The discrete model used in 3D FEM modeling is shown in Figure 5. Convergence and stability of the FEM solutions were ensured varying the types of elements, discrete mesh, and boundary conditions.

Seismic responses of strip raft foundation on piles obtained from Midas and EQPR analysis, horizontal ground motions at (a) x direction (b) y direction (c) bending moment (d) shear force

It is interesting to learn that the ground motions acting in x direction (longitudinal direction of the slab) will yield very small difference than those acting in y direction. Figure 6(c) and 6(d) depicted the internal bending moments and shear forces at the time (55.8 sec) when the maximum displacements occurred. The FEM and simplification solutions have some disagreements since the nonlinearities of the structure were captured by different material models. Various stress conditions at the pile heads will also affect the results. More comparisons on the internal stresses of the piles from different numerical modeling can be found in Hong (2016). Parametric studies on the influence factors have been reported (Chang et al., 2016).

(Midas, 2012) are tabulated in Table 1.

<table>
<thead>
<tr>
<th>Method</th>
<th>Material Properties</th>
<th>Model Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>EQPR analysis</td>
<td>Piles and raft : E= 3x10^9 MPa; γ = 24kN/m³; ξ = 0.02; ν = 0.1; 3D Midas-GTS analysis</td>
<td>Piles and raft : Linearly elastic</td>
</tr>
<tr>
<td></td>
<td>Soft soils: E= 137.4 MPa; Vₙ= 180 m/sec, γ = 14 kN/m³; ξ = 16 kN/m³; ξ = 0.05; ν = 0.3</td>
<td>Soft soils: Modified Cam Clay model</td>
</tr>
<tr>
<td></td>
<td>Gravel: E=1582.4 MPa; Vₙ=560m/sec; γ =20kN/m³; ξ =22kN/m³; ξ = 0.05; ν = 0.25</td>
<td>Gravel: Mohr Coulomb model</td>
</tr>
<tr>
<td></td>
<td></td>
<td>c = 0 kPa; φ = 36°; tₙ = 0.41</td>
</tr>
</tbody>
</table>

Table 1 Material properties and parameters used in the analyses

Figure 3 Numerical model for strip raft foundation on piles.

Figure 4 Seismic Inputs for the analyses

Figure 5 3D DEM model for the validation of the simplified analysis
To reveal the influence of displacement factor $R$, the superstructure was taken as a single degree of freedom (SDOF) system mounting on the raft. The resolved foundation time-dependent accelerations can be treated as the base motions to solve for the associated motions of the superstructure. Time histories of the relative displacements and the absolute displacements as well as the time-dependent ratio $R$ can be shown in Figure 7(a), 7(b), and 7(c). $R$ was found oscillating with time in between 0.5–1.2.

Figure 7 SDOF motions of the superstructure subjected to the foundation shaking (a) relative displacement time-history (b) absolute displacements time-history (c) displacement ratio $R$ calculated as a time-dependent function.

Referring to the response of a single pile solution under the earthquake obtained earlier by Chang et al. (2016), it seems that the responses of piles and raft of the foundation will be governed by the ground motions. From the 3D FEM modeling, the pile responses were found also dominated by the ground motions. Table 2 shows the required time for computations, the EQPR analysis seems to be a very efficient solution to the preliminatory design. It will make the Performance Based Seismic Design (PBSD) much easier for the piled raft foundations.

Table 2 Computation time of the numerical analyses

<table>
<thead>
<tr>
<th>Method</th>
<th>Computer features</th>
<th>Computation time (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EQPR analysis</td>
<td>CPU: Intel Xeon</td>
<td>60 sec based on Δt of</td>
</tr>
<tr>
<td></td>
<td>E3-1231v3</td>
<td>0.0005sec (computations)</td>
</tr>
<tr>
<td></td>
<td>RAM: 16GB</td>
<td>required for EQWEAP</td>
</tr>
<tr>
<td>3D Midas</td>
<td></td>
<td>analysis is included</td>
</tr>
<tr>
<td>GTS analysis</td>
<td></td>
<td>9hr 25min 10sec for 174780</td>
</tr>
<tr>
<td></td>
<td></td>
<td>elements based on Δt of 0.02</td>
</tr>
</tbody>
</table>

4 CONCLUSIONS

A simplified analysis called EQPR was suggested to monitor the seismic responses of a strip foundation on piles subjected to horizontal earthquake motions. Finite difference formulas were used to discretize the governing differential equation of the strip foundation. The simplified analysis is validated with 3D finite element analysis for pile responses subjected to seismic horizontal ground motions. EQPR analysis seems to be a very efficient solution to the problem.

3. The observations have limitations based on the usage of shear springs. Seismic design of the piled raft foundation needs to check carefully the internal stresses of the piles. The foundation displacements should be used only for explicit comparisons.

5 ACKNOWLEDGEMENTS

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6 REFERENCES


Shaking table test and numerical simulation on seismic performance of a bridge column integrated by multiple steel pipes with directly-connected piles

Essai à la table vibrante et simulation numérique de la performance sismique d'une colonne de pont intégrée par de multiples tuyaux en acier avec des pieux directement connectées.

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ABSTRACT: A bridge column integrated by multiple steel pipes and connected directly to piles without a footing has been proposed to design a rational foundation of the column. Based on the past achievements, the proposed substructure possess an excellent advantage of reduction of strain at the column through strain decentralization at footing point. In addition, reduction in footing weight contributes to decrease pile strain. On the other hand, the proposed substructure has some disadvantages i.e. increase in strain and displacement of piles. But it has been revealed that the strain generated at piles can be minimized by using a beam in the ground. This paper, the seismic performance of the bridge column structure grounded in liquefiable sand is evaluated based on the large-scale shaking table tests using a bridge column model with the scale of 1/20. Subsequently, a soil-water coupled FE analysis is conducted to reproduce the experimental results and confirm the seismic performance and the detail of the mechanism.

RÉSUMÉ : Une colonne de pont intégrée par de multiples tuyaux en acier et directement connectée à des pieux sans semelle a été proposée pour concevoir une fondation rationnelle de la colonne. Sur la base des réalisations passées, la sous-structure proposée possède un excellent avantage de la réduction de la déformation de la colonne par décentralisation de la déformation au point de la semelle. En outre, la réduction du poids d’une semelle contribue à diminuer la déformation d’un pieu. D’autre part, la sous-structure proposée présente certains inconvénients, c’est-à-dire une augmentation de la déformation et du déplacement des pieux. Mais il a été révélé que la déformation générée aux pieux peut être minimisée en utilisant une poutre dans le sol. Dans cet article, la performance sismique de la structure de la colonne du pont fondée dans du sable liquéifiable est évaluée en fonction des essais à la table vibrante à grande échelle de secousses à l’aide d’un modèle de colonne de pont avec une échelle de 1/20. Par la suite, une analyse couplée EF et sol-eau est effectuée pour reproduire les résultats expérimentaux et confirmer la performance sismique et le détail du mécanisme.

KEYWORDS: damage-controlled structure, pile foundation, footing-less, liquefaction, shaking table test, FE analysis.

1 INTRODUCTION, FIRST LEVEL HEADING

A bridge column integrated by multiple steel pipes and multiple shear panels interconnecting the pipes (Fig. 1) has been proposed and put into practical use (Shinohara et al., 2012). The bridge column is designed based on damage-control concept, in which the vertical load such as dead load and traffic load is supported by multiple steel pipes, and lateral load such as seismic load is adjunctively supported by shear panels made of low yield steel. Seismic damage will be aggregated on only the shear panels so that it enables early recovery by replacing only the shear panels after an earthquake.

Subsequently, a multiple steel pipes bridge pier integrated by pile foundation without a footing has been proposed to design a more rational foundation (Shinohara et al., 2013). Based on the past achievements, the proposed substructure has advantage of strain reduction at the column by strain decentralization at footing point. In addition, reduction in footing weight contributes to decrease pile strain. On the other hand, the proposed substructure has some disadvantages i.e. increase in strain and displacement of piles. But it has been revealed that strain of piles could be decreased by using a beam in the ground.

In the previous research (Isobe et al., 2017), the seismic performance of the structure in dry and liquefiable sand is evaluated based on the large-scale shaking table tests using a bridge column model with the scale of 1/20, comparing to a conventional pile foundation using a footing. In this paper, after mentioning the outline of shaking table tests, the outline and results of the soil-water coupled FE analysis to reproduce the experimental results and confirm the seismic performance and the detail of the mechanism is shown.

![Figure 1. A bridge column integrated by multiple steel pipes and multiple shear panels interconnecting the pipes.](https://example.com/figure1.png)
2 OUTLINE OF SHAKING TABLE TEST

Fig. 2 shows the bridge column and foundation model with the scale of 1/20 used in the shaking table tests. Two types of models are used; one has a group of pile foundation (8 piles) with a footing and the other has directly connected piles (4 piles) without a footing. In this paper, each model is called F-type and S-type, respectively. The integrated bridge column consists of 4 steel tubes (STK 400) and three-layered shear links (LY225) to interconnect the pipes. In S-type, the underground beam is used to interconnect the piles around the pile head for the purpose of reducing displacement of the pile head. The pile and pipe spacing is set to be 2.5Dp (Dp: pile diameter). A weight of 52.6 kN is applied at the pier top.

Fig. 3 shows the schematic view of the shaking table tests. The soil chamber, which measured 4.0 m in length, 1.0 m in width and 2.0 m in depth, has cushioning material made of foam rubber on the sidewall surface to reduce the influence of the rigid sidewall during shaking. In the case of dry sand, the ground is modeled by Tohoku silica sand #6 with a relative density (Dr) of 80% using the tamping method. In the case of liquefiable sand, the liquefiable ground is modeled by pluviation in water using Tohoku silica sand #6 with Dr of 40%. The non-liquefiable ground is modeled in the same manner as the case of dry sand. Ground water level in the liquefiable sand cases is set up to G.L. -0.2 m. The ground configuration in detail is described in Fig. 3. The soil physical properties (Isobe et al.) are shown in Figs. 4 and 5.

The layout of measuring instruments such as displacement gauges, accelerometers and pore water pressure gauges are also shown in Fig. 3. The strain gauges are attached on the column, piles and shear panels. As input, 20 cycles of a 2 Hz sinusoidal tapered wave are used, varying the target acceleration amplitude from 0.62 m/s² to 5.25 m/s² in a step-by-step manner for the dry sand case and setting it up as 2.0 m/s² for the liquefiable sand case. Table 2 shows the test cases.

3 OUTLINE OF NUMERICAL SIMULATION

The DBLEAVES soil-water coupling FE analysis code (Ye et al., 2007) was used in the simulation. The Cyclic Mobility model developed by Zhang et al. (2007), which incorporates the concepts of subloading and superloading as described by Hashiguchi and Ueno (1977) and Asaoka et al. (2002), was used as the constitutive model. The soil parameters are determined based on the results of isotropic consolidation test, triaxial tests and cyclic triaxial tests to the specimens with Dr of 40% and 80%. The parameters for which detailed information is unavailable, the properties of Toyoura sand has been used which has a similar grain size distribution as Tohoku sand. The soil parameters are shown in Table 2. The cushion attached on the side wall is also modeled by elastic solid elements (E = 0.05 GPa, ν = 0.49, ρ = 0.07 g/cm³).

In the analysis, the integrated column, the underground beam and the shear panels are simply modeled by the bi-linear type of elasto-plastic beam and spring elements, respectively. The steel pile in the ground is modeled by the hybrid element which can consider the soil-pile interaction adequately (Zhang et al., 2000). The parameters are shown in Tables 3 and 4. The weight (52.6 kN) fixed on the structure is modeled by a single mass.

By considering symmetry of geometrical and loading conditions, only half of the domain is used in the analysis. Fig. 6 shows the finite element mesh used in the simulation. The boundary conditions are as follows: (a) the bottom of the ground is fixed, (b) the vertical boundaries parallel to the XOZ plane are fixed in the y direction and free in the x and z directions, (c) the vertical boundaries parallel to the YOZ plane are fixed in the x direction and free in the y and z directions and (d) the ground surface above the water table is set with a drainage condition, while the other surfaces are impermeable.

(d) the ground surface above the water table is set with a drainage condition, while the other surfaces are impermeable.
Table 1. Test cases.

<table>
<thead>
<tr>
<th>Material</th>
<th>F-type</th>
<th>S-type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry sand</td>
<td>D-F</td>
<td>D-S</td>
</tr>
<tr>
<td>Liquefiable sand</td>
<td>D-F</td>
<td>D-S</td>
</tr>
</tbody>
</table>

Table 2. Soil parameters.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Dr 40%</th>
<th>Dr 80%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression index å</td>
<td>0.036</td>
<td>0.036</td>
</tr>
<tr>
<td>Swelling index x</td>
<td>0.058</td>
<td>0.005</td>
</tr>
<tr>
<td>Stress ratio at critical state K_c</td>
<td>3.924</td>
<td>4.812</td>
</tr>
<tr>
<td>Void ratio (p_c = 98 kPa on NCL) N</td>
<td>0.700</td>
<td>0.700</td>
</tr>
<tr>
<td>Poisson’s ratio ν</td>
<td>0.300</td>
<td>0.300</td>
</tr>
<tr>
<td>Degradation parameter of overconsolidation state m</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>Degradation parameter of structure a</td>
<td>2.200</td>
<td>2.200</td>
</tr>
<tr>
<td>Evolution parameter of anisotropy b_e</td>
<td>1.500</td>
<td>1.500</td>
</tr>
<tr>
<td>Initial mean effective stress [kPa] p_0</td>
<td>0.48–3.61</td>
<td>0.46–19.1</td>
</tr>
<tr>
<td>Initial degree of structure K_s</td>
<td>0.400</td>
<td>0.900</td>
</tr>
<tr>
<td>Initial degree of overconsolidation s</td>
<td>0.004</td>
<td>vary according to p_0</td>
</tr>
<tr>
<td>Initial anisotropy δ_0</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Permeability [m/sec] k</td>
<td>1.0E-04</td>
<td>1.0E-04</td>
</tr>
<tr>
<td>Dry unit weight [kN/m^3] γ_d</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td>Saturated unit weight [kN/m^3] γ_s</td>
<td>19.3</td>
<td>20</td>
</tr>
<tr>
<td>Unit weight under water [kN/m^3] γ'_w</td>
<td>9.5</td>
<td>10.2</td>
</tr>
</tbody>
</table>

Table 3. Parameters of the column and the pile.

<table>
<thead>
<tr>
<th>Column</th>
<th>Pile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
<td>STK400</td>
</tr>
<tr>
<td>Model type</td>
<td>Beam</td>
</tr>
<tr>
<td>Diameter (mm)</td>
<td>89.1</td>
</tr>
<tr>
<td>Thickness (mm)</td>
<td>2.8</td>
</tr>
<tr>
<td>Cross section stiffness (kN)</td>
<td>1.5x10^6</td>
</tr>
<tr>
<td>Flexural rigidity (kN-m)</td>
<td>1.4x10^6</td>
</tr>
<tr>
<td>Yield moment (kN-m)</td>
<td>3.73</td>
</tr>
</tbody>
</table>

Table 4. Parameters of the panel and the underground beam.

<table>
<thead>
<tr>
<th>Panel</th>
<th>Underground beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
<td>LY225</td>
</tr>
<tr>
<td>Model type</td>
<td>Spring</td>
</tr>
<tr>
<td>Size (mm)</td>
<td>61.0 x 61.0</td>
</tr>
<tr>
<td>Thickness (mm)</td>
<td>11.1</td>
</tr>
<tr>
<td>Shear stiffness (kN/m^2)</td>
<td>7.7x10^7</td>
</tr>
<tr>
<td>Yield shear stress (kN/m^2)</td>
<td>1.2x10^7</td>
</tr>
</tbody>
</table>

Figure 6. Finite element mesh used in the simulation (L-F).

Figure 7. Relationship between response acceleration and lateral displacement at the pier top of the structure.

4 RESULTS AND DISCUSSION

Due to space limitations, only results for the dry sand cases are shown here. Fig. 7 shows the relationship between response acceleration and lateral displacement at the pier top obtained from the model test and numerical simulation. The simulation results reproduce that both response acceleration and lateral displacement for F-type are less than those for S-type in the first and second steps. The initial stiffness for both cases obtained from simulation results is almost the same as those from test results. On the other hands, in the third step the trend is reversed, which indicates that the bigger response acceleration and displacement for F-type are larger than those for S-type. The simulation results reproduce such a behavior somewhat adequate although it overestimates the history loop for S-type.

Fig. 8 shows the relationship between response acceleration at the pier top and shear strain generated on the upper shear panel obtained from the model test and numerical simulation. Although they slightly overestimate the shear strain for both cases, the simulation results reproduce the following qualitative features: (i) in the first and second steps, the shear strain for S-type is larger than that for F-type, (ii) in the first step the shear strain for S-type exceeds the yielding strain, (iii) in the second step the shear strain for both types exceed the yielding strain, (iv) in the third step the magnitude relationship between F-type and S-type is reversed. These results are coincident with the trends of lateral displacement mentioned above.
In both results, the strain reduction at the footing point for S-type structure is seen. Also, the strain generated on the piles for S-type is larger than that for F-type due to the difference of the rigidity of the foundation although the distribution curve is slightly different. Fig. 10 shows the lateral displacement of the structures in depth while the maximum lateral displacement at the pier top for the third step. The simulation results reproduce the features that the lateral displacement of the pier top for S-type is smaller than that for F-type although the lateral displacement of the piles for S-type is larger than that of F-type, which indicates S-type has high seismic performance and high toughness against large-scale ground motions.

5 CONCLUSIONS

A bridge column integrated by multiple steel pipes and connected directly to piles without a footing is proposed as the reasonable damage-controlled structure. In this paper, the seismic performance of the structure in dry sand is evaluated based on large-scale shaking table tests and a soil-water coupled FE analysis. As with the past achievements, the proposed substructures have advantages of strain reduction of column by strain decentralization at footing point. It is recognized that it has high seismic performance and high toughness if the conditions are right in view of the fact that the main member (columns and piles) holds a large residual strength after yielding of the shear panels.

6 REFERENCES


End bearing capacity of embedded ringed piles with different tip shapes.

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Kyeong-Han Jeong  
The Dream ENC, Republic of Korea

ABSTRACT: Embedded piles are widely used because of low noise and vibration. Several studies have been made on embedded pile method to increase end bearing capacity. For the purpose of decreasing the use of piles to reduce cost and period of construction, large-diameter plate or steel pipe is attached to the piles. In this study MAX Pile, which inclined steel ring is attached at the tip is developed. The test construction and dynamic pile tests are carried out and analytical study is conducted for verifying increase in end bearing capacity of MAX Pile. This research shows that the end bearing capacity of MAX Pile is bigger than that of the pile without steel ring and the end bearing capacity is bigger when the inclination of ring is bigger. By using MAX Pile, the number of piles needed is reduced so that the cost and construction period is reduced as well.

KEYWORDS: analytical study; embedded piles; end bearing capacity; field test.

1 INTRODUCTION

Recently, embedded piles are widely constructed because of low noise and vibration. Since noise and vibration control act was released, construction of embedded piles rapidly increased and therefore the embedded pile method has been applied to most construction sites (Cho 2010). The procedure of construction of embedded piles as follows: excavation to depth of weathered rock or soft rock ground, penetration of pile and finish with blow and so on (Im et al. 2001).

On the purpose of checking bearing capacity of embedded pile, pile load test is conducted alike the case of driven pile. Dynamic pile test is widely used because of its convenience and affordability. The end bearing capacity of pile is estimated using the results of end of initial driving (EOID).

In an attempt to reduce cost in construction, many studies of increasing end bearing capacity of embedded piles so that number of piles that are constructed are done to reduce expense and period on construction (Paik and Yang 2013). The most representative cases are the method that increases the area of pile tip by attaching large-diameter plate or thickening piles (Yoo et al. 2007) and the method that increases depth of penetration by attaching steel pipe at the tip of piles.

In this study MAX Pile, which inclined steel ring is affixed at the tip for the purpose of increasing the end bearing capacity, is developed. The concept picture of MAX pile is in Figure 1. As a way of testifying increase in bearing capacity of MAX Pile, dynamic load tests are conducted after construction of test piles. The experiment shows that bearing capacity of MAX Pile is bigger than that of pile without steel ring and the bigger angle of MAX Pile is, the bigger bearing capacity is. Analytical studies are also conducted to establish the cause of increase in end bearing capacity.

2 FIELD TEST

2.1 Experimental setup

For the purpose of confirming increase in the end bearing capacity, three MAX Piles which inclination of steel rings is 30°, 45° and 60° each and a pile without ring were constructed in field. Right after construction dynamic pile test was conducted and as the result of the test end bearing capacity of each pile was estimated because no skin friction exists before grouting curing. All piles were embedded in weathered rock contiguously by SDA(Seperated Doughnut Auger) method. The piles are 500mm in diameter and 12m in length and the diameter of steel rings is 560mm.

2.2 Experimental results

The measured end bearing capacities and calculated increments are in Table 1.

<table>
<thead>
<tr>
<th>Case</th>
<th>End bearing capacity [ton]</th>
<th>Increment [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without tip</td>
<td>282.3</td>
<td>-</td>
</tr>
<tr>
<td>30° tip</td>
<td>367.7</td>
<td>30</td>
</tr>
<tr>
<td>45° tip</td>
<td>410.0</td>
<td>45</td>
</tr>
<tr>
<td>60° tip</td>
<td>465.8</td>
<td>65</td>
</tr>
</tbody>
</table>

As shown in Table 1, the end bearing capacity of MAX Pile is bigger than that of the pile without steel ring and the end bearing capacity is bigger when the angle is bigger.

3 ANALYTICAL STUDY

3.1 Effective bearing area

Considering the effective bearing area of each case is a simplistic approach to analyze the end bearing capacity. The increment of end bearing capacity is same as that of end bearing area. The calculated values of increment are in Table 2.
Table 2. End bearing capacity of each case.

<table>
<thead>
<tr>
<th>Case</th>
<th>Increment [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without tip</td>
<td>-</td>
</tr>
<tr>
<td>30° tip</td>
<td>26</td>
</tr>
<tr>
<td>45° tip</td>
<td>30</td>
</tr>
<tr>
<td>60° tip</td>
<td>42</td>
</tr>
</tbody>
</table>

The calculated values in Table 2 are smaller than values in Table 1 because the results came from the simply calculated end bearing area.

3.2 Equilibrium analysis

Terzaghi (1943) proposed the theory to explain bearing capacity of shallow foundation and Vesic (1977), Meyerhof (1976), Coyle and Castello (1981) improved the theory to apply to deep foundation (Das 1997).

In equilibrium analyses ultimate unit bearing capacity of pile foundation is calculated as

\[ q_u = c'N_c + q'N_q + \gamma DN_{\gamma} \]  

where \( q_u \) is the unit bearing capacity, \( c' \) is cohesion of soil, \( q' \) is effective vertical stress at end bearing of pile, \( D \) is diameter of pile and \( \gamma \) is unit weight of soil. \( N_{c'} \), \( N_{q'} \), and \( N_{\gamma} \) is the coefficients of bearing capacity.

When considering \( \gamma DN_{\gamma} \) part, unit bearing capacity of pile is proportional to the diameter of pile. As shown Figure 2, the diameter of MAX Pile, \( D' \) is calculated as Eq. 2.

\[ D' = D + 2(0.06D + 0.06D \frac{\tan \omega}{\tan \alpha}) \]  

where \( D \) is the diameter of the pile without rings, \( \Theta \) is the inclination of steel rings, and \( \alpha \) is the coefficient of soil and 60° in this study.

End bearing capacity is the multiplication of unit bearing capacity and area. The calculated values of increment of the end bearing capacity of MAX Pile is in table.

Table 3. End bearing capacity of each case.

<table>
<thead>
<tr>
<th>Case</th>
<th>Increment [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without tip</td>
<td>-</td>
</tr>
<tr>
<td>30° tip</td>
<td>35</td>
</tr>
<tr>
<td>4</td>
<td>42</td>
</tr>
<tr>
<td>60° tip</td>
<td>54</td>
</tr>
</tbody>
</table>

The calculated values in Table 3 are more similar the values of experimental results compared to that of effective bearing area.

3.3 Comparison and analysis

In this study, experimental verification and theoretical approach about increase in the bearing capacity of MAX Pile were conducted. In case of 60° MAX Pile, the bearing capacity increase by 65% compared to the pile without ring for the result of dynamic pile test. Also, the bearing capacity of 60° MAX Pile is bigger than the general pile by 42~54%.

The reasons for increase in bearing capacity are the increase in pile tip area and densification of ground around the ring. Firstly, the tip area of MAX Pile is larger than that of the pile without ring by 25%. The bigger tip area is concluded the bigger bearing capacity. For the second reason, during penetration of MAX Pile, the ground near the tip is densified because of inclined shape of ring of MAX Pile. This densification causes the increase in elastic modulus and strength of the ground.

4 ACKNOWLEDGEMENTS

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5 REFERENCES