Field Press-in Test of Tapered Steel Pipe and Timber Piles

Koji Tominaga*, Qunli Chen², Masahito Tamura¹ and Akihiko Wakai⁴

¹ Professor, IDEC, Hiroshima University, Japan
² Researcher, IDEC, Hiroshima University, Japan
³ Chief Research Engineer, Building Research Institute, Japan
⁴ Associate Professor, Faculty of Eng., Gunma University, Japan

Abstract
A relatively simple method was proposed in a previous study for predicting the vertical behavior of tapered piles penetrating from the ground surface. This method uses the radial cavity expansion theory to model the increase in side resistance due to the radial ground expansion as a tapered pile is being pressed, and applies the second theory of the ultimate point bearing capacity proposed by Takano N. for the point resistance. Field press-in tests were conducted on four steel sheet piles and nine timber piles at a site mainly composed of silty layers. This paper presents the test results of these four steel piles and three timber piles including one straight-sided steel pile and one straight-sided timber pile. Pile resistance obtained from field tests is compared with the predictions made by the proposed analytical method. As expected, the shaft resistance increased with the increase in taper angle and the tapered piles offered a larger resistance than the straight-sided piles. It was also found that, on sites consisting of extremely soft soil, the penetration force-displacement relationships are very sensitive to the internal frictional angle of the soil.

Keywords: tapered pile; shaft resistance; radial cavity expansion; force-displacement relationship

1. Introduction
First, tapered piles used in building foundations in soft ground have obvious advantages over straight-sided piles: lower negative skin friction along the shaft resulting from settlement of surrounding soil, less reduction in soil reaction under the raft of piled-raft foundations because piles follow the ground settlement better, and higher recyclability because they are easier to pull out of the ground. Moreover, tapered piles can be expected to provide higher resistance because of the increase in skin friction along the shaft due to the increase in lateral soil reaction caused by hole expansion resulting from the pile's penetration into the ground.

Second, thinning out excess trees is carried out worldwide to protect forests from over-growing. Effective use of thinned-out trees in various industries is recognized as a way to promote sustainable forest protection. In Japan, after the amending of the Basic Forestry Law, the use of timber products in buildings and infrastructures has been widely promoted.

Third, Japan is a country consisting of islands, and 67% of its land is occupied by mountains. Thus, the small plain area is efficiently used as the major base for building structures. However, the geotechnical layers near the ground surface are often soft, thus requiring deeper foundations even for small buildings. Foundations on the surface layers often fail to provide enough bearing capacity even for detached houses.

Thus, the idea of using forest thinnings for pile foundations for small-scale detached houses has naturally come into the researchers' minds. In general, tree trunks are naturally tapered from bottom to top. Therefore, timber piles for building foundations should be designed as tapered piles.

However, most design procedures and guidelines have been developed for straight-sided piles with little or no reference to tapered piles.

In recent decades, however, some researches have reported experimental and/or theoretical studies on vertical behaviour, bearing capacity and so on of tapered piles. Wei et al. (1998) conducted model tests on tapered piles in a laboratory setup. EI Naggar et al. (1999) described tests on three instrumented model steel piles with different degrees of taper in a large-scale laboratory setup under compressive and tensile loads. J.S. Horvath et al. (2004) reported the results of field tests conducted on the site of John F. Kennedy International Airport, and compared experimental and calculated results based on an analytical method developed by one of them.
The primary analytical method was published by Nordlund (1963), in which the benefit of taper was accounted for by the increased lateral earth pressure coefficient on the pile side that produces side friction, and Kodikara & Moore (1993) presented a breakthrough work demonstrating the pile capacity mechanism as cylindrical cavity expansion with consideration of the nonlinearity condition along the pile-soil interface.

The first two authors (2006) of this paper have proposed a method based on an analytical model similar to Kodikara & Moore’s, with pile and soil modeled as slices deforming in lateral planes. The method takes into account the increase in shaft resistance due to the additional lateral pressure of soil when in-plane cavity expansion occurs in soil as the tapered pile penetrates from the ground surface under vertical force.

Field press-in tests have been carried out in Saga prefecture, Japan, on closed-end steel shell piles and timber piles. This paper presents the test results of pile resistance and compares them with results predicted by the proposed method.

2. Analysis Method Review

As indicated in Fig.1., when slice elements of a pile penetrate into the ground one after another, the axial penetration force on the pile head is balanced by the vertical components of the ultimate shaft resistance and the ultimate point resistance.

2.1 Assumptions of the analysis model

The following assumptions are made:
1) The pile-soil system comprises slices of thickness \( dz \) and there is no interaction between slices;
2) The pile is elastic with length of \( L \), head radius \( r_0 \) and tip radius \( r_b \). The taper angle \( \theta_T = \left( r_0 - r_b \right)/L \) is small (\( \theta_T \leq 1/20 \)).
3) Soil at the pile-soil interface yields, fulfilling the Coulomb criterion, as the pile penetrates into the ground. For each pile slice, as shown in Fig.2. (a), the following equation can be derived:

\[
\tau_a = \sigma_a \tan \left( \phi_a + \frac{\theta_T}{2} \right) + \frac{c_a \sec \theta_T}{\left( 1 - \tan \phi_a \tan \theta_T \right)}
\]  

(1)

where \( q_a \) and \( c_a \) are the friction angle and adhesion on the pile-soil interface, respectively. Then, the pile shaft resistance can be expressed as:

\[
F(z_p) = \int 2\pi r_0 \tau_a \, dz
\]  

(2)

where \( z_p \) denotes the displacement of the pile being pressed into the ground and \( r_0 \) is the pile radius at depth \( z \), as indicated in Fig.2. (b).

4) Let \( dU_z \) be the radial soil deformation along the pile surface at depth \( z \) due to pile penetration, and \( dp_{uw} \) be the relative radial soil reaction, or the additional soil pressure (evenly distributed along slice depth \( dz \)), so that

\[
\sigma_a = K_0 \gamma z + dp_{uw}
\]  

(3)

where \( K_0 \) is the coefficient of earth pressure at rest and \( \gamma \) is the unit weight of soil. The soil reaction \( dp_{uw} \) is evaluated from cavity expansion theory for 2-dimensional elastic (or elasto-plastic) soil on the assumption that the influence of layers outside the evaluated layer can be neglected and the small element of \( dz \) at \( z \) deforms horizontally.

5) The point resistance can be evaluated using the 2nd bearing capacity expression proposed by Takano N. (1981) for displacement pile.

2.2 Additional horizontal soil pressure

2.2.1 Elastic state

Fig.3. (a) shows the elastic analysis model of soil expansion around a pile based on assumption 4). Under internal pressure \( \left( dp_{uw} + K_0 \gamma z \right) \) acting at \( R_z \) \( \left( R_z = r_z + dU_z \right) \), the radial and tangential stresses in the soil at radius \( r \) \( (r \geq R_z) \), \( \sigma_r \) and \( \sigma_\theta \), can be expressed as:

\[
\sigma_r = \frac{A}{r^2} + B
\]

(4)

\[
\sigma_\theta = \frac{A}{r^2} + B
\]

(5)

When \( r \to \infty \), \( \sigma_r = \sigma_\theta = K_0 \gamma z \). The radial deformation \( dU_z \) can be expressed as the integral of strain from \( R_z \) to \( \infty \). Thus, the additional soil pressure \( dp_{uw} \) can be derived for \( \sigma_{\theta|r=r_0} = dp_{uw} + K_0 \gamma z \) as:

\[
\begin{align*}
\sigma_{\theta|r=r_0} &= dp_{uw} + K_0 \gamma z \\
&= dp_{uw} + \int_{R_z}^{\infty} \frac{A}{r^2} + B \
&= dp_{uw} + \left[ \frac{A}{r} + B \right]_{R_z}^{\infty} \\
&= dp_{uw} + \left( \frac{A}{\sqrt{R_z}} - \frac{A}{\infty} \right) + B (\infty - R_z) \\
&= dp_{uw} + A \left( \frac{1}{\sqrt{R_z}} - \frac{1}{\infty} \right) + B (\infty - R_z)
\end{align*}
\]

(a)Stresses on Pile Shaft (b) Radial Soil Reaction

Fig.2. Penetration and Lateral Soil Pressure
where $E$ and $\nu$ are the elastic modulus and Poisson's Ratio of soil, respectively.

2.2.2 Elasto-plastic state
When the radial displacement $dU_z$ increases, the soil near the pile enters the plastic state. As indicated in Fig.3. (b), when soil to radius $R_z$ is in plasticity, it is assumed that there is no volumetric change in the plastic zone. That is, soil volume change from $r_z$ to $R_z$ is equal to the volume change due to horizontal displacement $dU_p$ according to the elastic expansion of soil outside the elastic-plastic interface.

On the basis of the force equilibrium and yield criterion in the plastic zone, the additional soil pressure $dp_{aw}$ can be expressed as:

$$ dp_{aw} = (K_0 \gamma z + c \cot \phi)(1 + \sin \phi) \left\{ \left[ \frac{G}{K_0 \gamma z + c \cot \phi} \right]^\sin \phi \phi \right\} - c \cot \phi - K_0 \gamma z $$

A plastic soil zone exists when:

$$ dU_z \geq \frac{1}{r_z} \left[ 1 - (K_0 \gamma z + c \cot \phi) \sin \phi / G \right]^{1/2} - 1 $$

In the above equations, $c$, $\phi$ and $G$ are the cohesion, internal frictional angle and shear modulus of soil, respectively.

2.3 Point resistance
The 2nd bearing capacity expression proposed by Takano N. is expressed as:

$$ q_{az} = \alpha_1 p_m N_{q2} + \alpha_2 c N_{c2} $$

where $p_m$ is the effective mean confining stress at pile tip depth $z_p$, $p_m = (1+2K_0)p_v/3$, $p_v$ is the effective vertical soil pressure; and $N_{q2}$ and $N_{c2}$ are the bearing capacity factors expressed as follows:

$$ N_{q2} = (1 + \tan \phi \tan \psi) \frac{3(1 + \sin \phi)}{2(1 + \nu)} I_{s2} $$

$$ N_{c2} = (1 + \tan \phi \tan \psi) e^{(1-2\nu)\tan \phi} \left[ \frac{\sin \phi}{(1 + \nu) I_{s2}} - 1 \right] $$

+ ($\cot \phi + \tan \psi) e^{(1-2\nu)\tan \phi} - \cot \phi

Here, $\psi$ is the angle between the horizontal and the face of the soil wedge formed beneath the pile tip when the pile is being pressed in. In this study, $\psi = \phi$, assuming that pile and soil are in complete frictional contact.

Thus, the balance between axial penetration force $P$ and pile resistance (ultimate point resistance at pile tip and ultimate frictional resistance along the shaft) can be expressed as functions of penetration displacement $z_p$.

3. Outline of Field Test
3.1 Soil and pile conditions
The field press-in test was carried out in the vicinity of Ariake Bay, Higashi-Yoga, Saga prefecture. The soil profile of this site is shown in Fig.4. The subsurface layers are composed of extremely soft silt (with SPT $N$-values nearly zero, where SPT means Standard Penetration Test), except two sandwiched sand layers, which have SPT $N$-values $\approx 2$ at GL-2.5m to 3.0m and $\approx 6-7$ at GL-4.0m to 6.3m, respectively.

Field tests were conducted on Sep. 25-26, 2006 on four closed-end tapered steel sheet piles and nine timber piles, whose data are listed in Table 1. Press-in construction data were not recorded for pile No. 5 and pile Nos. 9-13 were pressed using vibration. Thus, in this study, only steel piles SS140, ST200, ST260 and ST380, and timber piles W3-140, W3-215 and W3-265 are discussed.

Here, the man-made steel piles are of equal length (3.0m) and tip diameter (140mm), but different head diameters (200mm, 260mm and 380mm), that is, different taper angles: 1/100, 2/100 and 4/100.

3.2 Testing procedure
For the closed-end steel sheet piles, 4 strain gauges were installed on the inside pile wall at each of 3 sections (near the head, in the middle and near the tip,
respectively. Because the diameters of the tapered piles vary along their lengths, the axial force-strain relationships at each pile section were first calibrated. The timber piles with solid sections had no installed strain gauges. As shown in Photo 1., the timber piles were sharpened into wedges of about 75° to horizontal to the pile tip.

In all cases, the piles were first pressed about 50 cm into the ground, and then the test started. During the test, the penetration force through the load cell placed at the pile head and the axial strains at each section were measured for every 10 cm of penetration until 30 cm of pile head was left above ground (see Photo 2.). For the timber pile, however, only penetration force at the pile head was measured.

3.3 Test results
For the steel piles, the penetration force and point resistance according to the pile-tip depth from the ground surface, that is, displacement, were as shown in Fig. 5., where the point resistances are converted from the average strains at the pile-tip based on the calibration of force-strain relationships, and the penetration forces were measured through the load cell. The difference between the penetration force and the point resistance was the total friction mobilized along the pile shaft.

It can be seen from this figure that all the piles maintained an approximately constant penetration force with displacement increase until almost the same depth of about 1.5 m, then increased abruptly to a peak value at about 2.0 m and then decreased as displacement increased. It can also be seen that the penetration force at a certain pile-tip depth increased with the increase in pile-head diameter, and that the difference between penetration force and point resistance, that is, pile shaft resistance, of all the tapered piles was much greater than that of the cylindrical piles and this difference increased with the increase in taper angle. Furthermore, the peak values of point resistance appeared at a shallower depth than those of the penetration force.

More detailed observation shows that the penetration forces of ST260 were very close to those of ST380, although their taper angles were very different. It can also be seen that, although the point resistances for
ST200 and ST260 were almost the same as that of SS140, that of ST380 was much less and its shape was different from those of SS140, ST200 and ST380. This might be because the soil conditions of ST380 were a little different from those of the others.

Variations of penetration force with displacement of pile-tip for the timber piles are shown in Fig.6. From this figure, it can be seen that the measurements for the timber piles showed a similar tendency to those of the steel piles, which showed an almost constant penetration force until 1.5m depth, a peak value at about 2.1m depth and after that a decrease. However, the peak penetration forces of W3-265 were a little smaller than those of W3-215, although its average diameter was larger. This might also be because there was some difference between the soil conditions of W3-265 and W3-215.

4. Analytical Presentation and Discussion of Results

4.1 Analysis conditions

The distribution of equivalent N-value shown in Fig.4 (b) was obtained from Swedish Weight Sounding (SWS) conducted near the area of the press-in tests. The ground level for SWS was about 50cm lower than that for SPT. Before the start of the press-in test, the site was further excavated by about 25cm and leveled. Thus, in this analysis, the soil layers from GL-0.75m to GL-2.25m in Fig.7, it was assumed that the soil cohesion was cohesionless, that is, \(C_a=0.0 \text{kN/m}^2\), and that the elastic modulus and internal frictional angle can be evaluated on the basis of the empirical equations: \(E_s=1400 \text{kN/m}^2\) and \(\phi=20 \text{°}\) (\(q_o=3.5\), \(3.5 \leq N_i \leq 20\)), respectively, (Architectural Institute of Japan (2001)). In the equation for \(q_o\), \(N_i\) is the revised N-value according to effective overburden pressure \(\sigma_{vo}\): that is, \(N_i = N(98/\sigma_{vo})^{0.5}\). To avoid over estimation of the soil resistance of layers very near the ground surface as described in the reference, \(N_i\) was assumed to be equal to \(N\) and the effect of the overburden pressure was considered in \(q_o\) in this paper. The value of \(q_o\) was backfigured from the test results of SS140 as shown later. Poisson's ratios for the silt and sand layers were also assumed to be 0.4 and 0.3, respectively.

The compressibility of a pile was considered with elastic modulus \(E_s=2.0*10^3 \text{kN/m}^2\) for steel piles and \(E_s=1.5*10^3 \text{kN/m}^2\) for timber piles.

4.2 Analysis results compared with test results

The sharpened wedge of the timber pile tip did not fulfill the assumption of taper angle less than 1/20. Therefore, because the proposed model could not be directly applied to the timber pile, the analysis was carried out for steel piles first. Then, on the basis of those results, analysis was performed for timber piles.

4.2.1 Results for steel pipe piles

Calculations based on the above parameters gave peak penetration forces much less than the test results for both cylindrical and tapered piles. Hence, the analytical parameters of the cylindrical straight pile SS140 should be examined without the influence of taper. The trials led to the conclusion that small changes of \(K_o\), \(C_o/C_a\) and \(E_s\) did not have much influence on the calculation results, while they were very sensitive to the above internal frictional angle of soil. Here, applying \(q_o=25^\circ\) to \(q=20^\circ\), the calculation results indicated fairly good agreement with test results.
agreement with the peak penetration force obtained from the test, as shown in Fig.8. For the sand layer at the test site, $\phi_0=25^\circ$ is considered as appropriate.

Fig.9. compares total and point resistances obtained from the tests on all the steel pipe piles and the calculation results with $\phi_0=25^\circ$. It is found that the predictions had the same tendency as the measurements for the cylindrical pile and tapered piles, and that for both penetration forces and point resistances, the predictions were in good agreement with the measurements. Moreover, except for pile ST380, whose predicted peak value was larger than the measured one, the prediction gave almost the same peak values as the tests. The frictional resistances along the pile shafts are shown in Fig.10. With the exception of pile ST380, the predictions of the proposed method are in fairly good agreement with the test results.

Except for pile ST380, however, there are sudden decreases in penetration force beyond the peak value in the calculations, which are especially obvious for point resistance. This may be because the sudden change of soil from sand to silt was input for the analysis, but the boundaries of the natural soil layers were not so clear.

4.2.2 Results of timber piles

Considering the effect of the sharpened wedge at the pile tip of the timber piles, the angle $\psi$ in the 2nd bearing capacity expression proposed by Takano (1981) were assumed to be the same as that of the pile wedge, $\psi=75^\circ$, instead of $\psi=\phi$ used for the steel piles.

Fig.11. compares the relationships between the penetration force and the displacement obtained on the basis of this analytical model. This figure indicates that the proposed analysis method showed the same tendency as the measurements. In a similar fashion to the steel piles, the calculation results of the proposed method showed the same tendency as the measurements, i.e., the penetration force was almost constant up to 1.5 m depth, then showed a peak value at about 2.0 m depth and then decreased.

However, the predictions gave larger peak values than the tests. This may be due to the above simple assumption made for the soil wedge angle $\psi$ in the 2nd bearing capacity expression proposed by Takano (1981). It might underestimate the effect of the sharpened part of the pile tip in reducing the point resistance, because the friction at the wood-soil interface was different from that inside the soil. Therefore, we intend to carry out further study concerning the mechanism of point resistance under the sharpened pile tip in the future.

5. Conclusions

Field press-in tests were conducted on four closed-end steel sheet piles and nine timber piles at a site...
with the layers consisting of extremely soft silt and silty sand. The purpose of these tests was to clarify the ultimate resistance mechanism during penetration of tapered piles from the ground surface and the difference between that of straight-sided piles.

This paper presents the results of these tests and examinations, and an analytical method for predicting the behavior of pressed-in piles from the ground surface, which is made up of the in-plane cavity expansion model and the 2nd bearing capacity theory (Takano N. (1981)). From this study, the following conclusions are drawn:

1) The shaft resistance increased with the increase in taper angle and tapered piles offer a larger resistance than straight-sided piles.
2) At the test site, consisting of extremely soft soils, the prediction of relationship between the penetration force and the displacement is sensitive to the internal frictional angle of the soil.
3) Using appropriate soil parameters for test data, the proposed method can give fairly precise estimation of pile resistance.
4) With the same parameters, the predicted peak penetration forces are almost the same as the test results for steel piles but mostly larger for timber piles. This might be because the angle of the sharpened part under the timber pile tip was used as the angle $\psi$ in the 2nd bearing capacity expression for the point resistance. In order to clarify this point, we intend to carry out further study on the mechanism of pile point resistance in the future.

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**References**

1) AIJ (2001) Recommendations for Design of Building Foundations. 2nd ed. Tokyo: Architectural Institute of Japan.
2) EI Naggar, M.H. and Wei, J.Q. (1999) Axial capacity of tapered piles established from model tests. Canadian Geotechnical Journal, 36(6), pp.1185-1194.
3) Horvath, J.S. and Trochalides, T. (2004) A half century of tapered-pile usage at the Kennedy International Airport. Proceedings: Fifth International Conference on Case Histories in Geotechnical Engineering, New York, NY.
4) Kodikara, K.K. and Moore, I.D. (1993) Axial response of tapered piles in cohesive frictional ground. Journal of Geotechnical Engineering, American Society of Civil Engineering, 119, pp.675-693.
5) Norlund, R.L. (1963) Bearing capacity of piles in cohesionless soils. Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineering, 89(SM3), pp.1-34.
6) Shirashi, S. (1969) Soil Mechanics and Foundation Engineering Library -5: Application of Soil Mechanics in Building Construction. Tokyo: The Japanese Geotechnical Society.
7) Takano, N. (1981) The base resistance of a non-displacement pile in sands (Doctoral Thesis). Tokyo Institute of Technology.
8) Tominaga, K. and Chen, Q.L. (2006) An analysis for relationships between vertical load and settlement on a tapered pile. Journal of Structural and Construction Engineering, 603, pp.77-83.
9) Wei, J.Q. and EI Naggar, M.H. (1998) Experimental study of axial behavior of tapered piles. Canadian Geotechnical Journal, 35(4), pp.641-654.

![Fig.11. Analysis Results Compared with Test Results for Timber Piles](image-url)