

An Investigation on Aspects of Damage to Precast Concrete Piles Due to the 1995 Hyogoken-Nambu Earthquake

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Aspects of typical damage at deeper underground part of precast concrete piles, referred to as the K-shaped (in Japanese character pronounced "ku") failure in this paper, have been investigated. Such failures are attributed to lateral flow at liquefied reclaimed land, such as Port Island and Rokko Island in Kobe city, during the 1995 Hyogoken-Nambu earthquake. A simple static analysis proposed earlier by the first author and referred to as the load distribution method has been utilized to evaluate the horizontal resistance of piles and to investigate the cause of this type of failure. Results of the analysis show that bending stresses in piles exceed pile strength. In addition, the shearing stress at slip surface formed by liquefied soil layer is close to the ultimate strength. It is shown that the effects of lateral flow on piles may be effectively represented by the combination of distributed load arising from active earth pressure due to separation of frontward soil from piles and the concentrated load corresponding to the slip force in soil behind the piles.

INTRODUCTION

The 1995 Hyogoken-Nambu earthquake resulted in damage examples in no small numbers of not only superstructures but also pile foundations, especially precast concrete piles such as prestressed concrete (PC) piles, supporting residential buildings (AIJ, 1996). In general, the damage aspects may be classified into two categories, consisting of failure close to the pile top and at deeper parts of long piles. The former is the bending-shear or shear failure type of failure caused mainly by inertia force of superstructure, which the authors have pointed out repeatedly as the repetition of the nature of damage noted during the 1978 Miyagiken-oki earthquake (Sugimura et al, 1998, Ishii et al, 2002). The latter refers to damage or breaking of pile member at relatively deeper part usually due to lateral

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flow at the boundary or within a liquefied layer. This may be considered to be a damage aspect peculiar to the Hyogoken-Nambu earthquake, and is referred to as the K-shaped (in Japanese character pronounced "ku") failure. This paper is concerned with the investigation of this type of failure based on a newly proposed approach of hypothetical failure mechanism.

OUTLINE OF THE RESEARCH

Of the various research attempts to clarify the occurrence mechanism of K-shaped failure in piles, a committee report culminating in the symposium held in May 1998 by the Japanese Geotechnical Society concerning the flow and permanent deformation of the ground and soil structures during earthquakes is noteworthy. The initiative may be considered a concerted such effort, because it resulted in an inclusive summary (JGS, 1998). According to the report, methods of evaluating the effect of lateral flow of fluid ground on piles may be broadly classified into the following three groups: (i) earth pressure method, (ii) fluid pressure method and (iii) response displacement method.

Characteristically all the three methods above involve consideration of the effect of pile being pushed forward by the flowing ground mass behind the pile. However, close observation of actual cases such as shown in the photos in Figures 1 and 2 indicate cracks in the ground extending from the edge of quay wall to the toe of sloping ground. In fact, based on the simple experimental and numerical studies on another damage example of ground fissures radiating from the footing of a bridge pier, Tazo et al. (1999) have pointed out that the ground fissures appeared not because of the spreading ground pushing from behind the piles and footings, but by movement toward the river (or sea) of the ground behind the quay wall. The observation and explanations give the impression that the effect of lateral spreading emerged in a manner similar to what is known as slump in the field of landslide, as shown in Figure 3(a). The similarity of the image of the occurrence of lateral spreading reconstructed detailed field observation with the slump in landslide is seen to be remarkable in Figure 3. This interpretation that the fluid soil layer does not push the piles from the back but leaves from in front of the piles is worth noting as a unique idea. This in turn reminds one of Nojiri's work (1997) emphasizing the removal of earth support on the excavated side as the 'mechanism of unloading' in excavation process.

There is another research work drawing the interest of authors. Nakazawa et al. (1996) inferred the occurrence of a slip surface above which the lateral flow of the liquefied reclaimed layer had occurred, as shown in Figure 7(a). They have explained that failure of

piles under northern footings occurred due to inertia force of the superstructure while the failure of the piles under southern footings occurred due to slippage within the ground. Subsequently, Nakazawa et al. (1999) have presented a more detailed report on this building and ground conditions and reconfirmed the difference of failure type between northern and southern pile group noted earlier. That is, the damage to piles under southern footings was caused by lateral flow of the liquefied reclaimed layer, referred to as the K-shaped failure in this paper as an attempt to identify it as a new type of failure mode.



Figure 1. Photograph showing overall view of the building looking from the west side, with blocks A, B and C from left to right, where the damage to quay wall to the west can be noticed.



Figure 2. Photograph showing closer view of the south-west corner, where the widened gap due to bulging out of the quay wall towards the sea can be clearly seen.

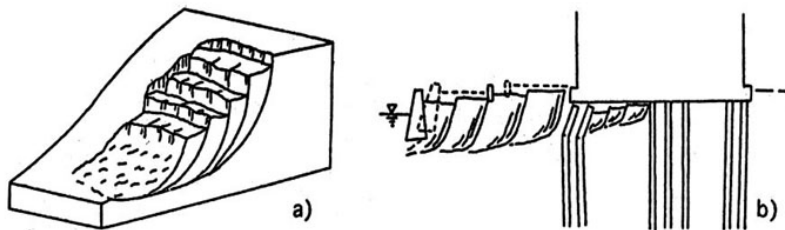


Figure 3. The schematic illustration of the K-shaped failure mechanism indicating the similarity of (a) slump during landslide with the (b) lateral spreading around quay wall due to liquefaction.

THE MECHANISM OF K-SHAPED FAILURE

The photograph in Figure 2 shows the bulging and southward movement of the quay wall. As a result, several cracks parallel to the building had appeared on the hardtop of parking area to the west and there were vertical cracks on the quay wall to the east as shown in the photographs in Figures 4 and 5 respectively. On the north side of the building, cracks on the ground parallel to the building and vertical gap at plinth level were observed. The photograph in Figure 6 is a closer view showing a large vertical gap under the north corridor and the crack parallel to the building about 5m in front. The interval of comparatively large cracks along the longitudinal direction of the building can be estimated to about 5 to 7m.



Figure 4. Photograph showing cracking at parking lot and fence parapet on the west side.



Figure 5. Photograph showing cracking on the quay wall to the east of the building.

From the basic observations described above concerning the behavior of building and ground, authors have attempted to develop a simple analytical technique to depict the occurrence mechanism by combining the ideas of Tazo et al. (1999) and Nakazawa et al. (1996). That is, in consideration of the development of slip surface as illustrated in Figure 7(a), the ground in front of the piles recedes towards the sea caused by movement quay wall. As a result, an unbalance of earth pressure between the front and the back of the pile

group occurs. It can be considered to resemble the situation of excavation in the front such that load distribution regarded as active earth pressure from the back may be assumed. However, the lateral pressure was considered instead of the active earth pressure, because the ground would have been in fluid state making it difficult to distinguish earth pressure from hydraulic pressure, and a coefficient of lateral pressure of 0.7 was assumed.



Figure 6. Photograph as seen from the north side of block A, where gap developed under the corridor and the crack parallel to the building about 5m in front may be noticed.

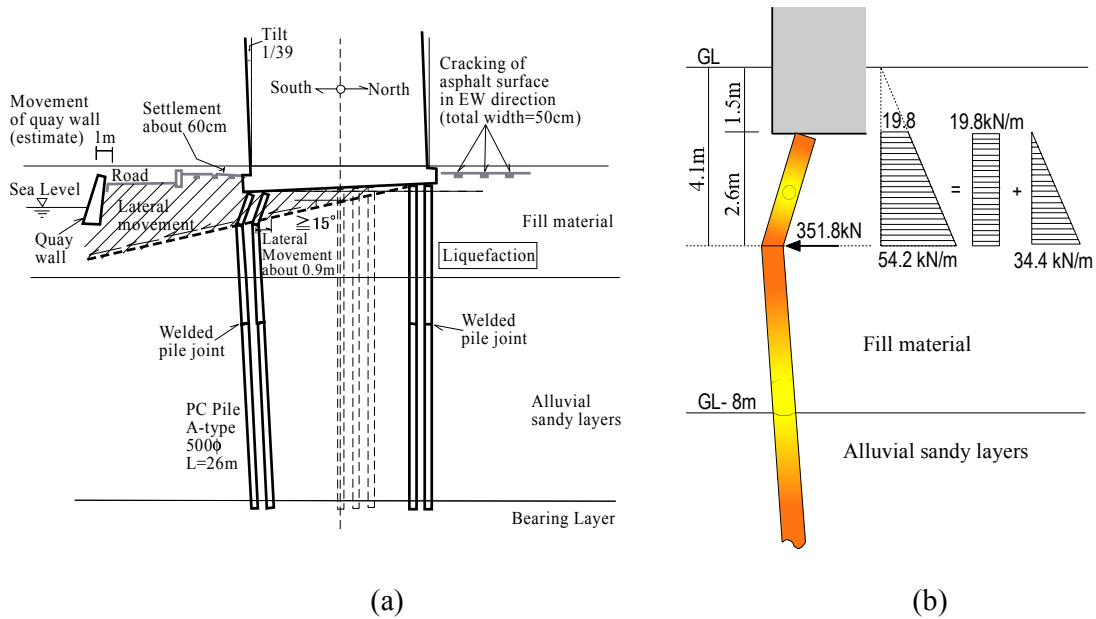


Figure 7. (a) Slip surface and displaced shape of the damaged pile based on the investigation by Nakazawa et al. (1996 & 1999) and (b) hypothetical load distribution considered in the analysis.

Next, as a result of the development of slip surface in the ground behind pile group, certain addition demand on pile resistance may be expected. It is regarded as the slip force in this paper and represented by a concentrated load located at a depth where the slip surface interfaces the pile group. Based on the investigation of Nakazawa et al. (1999) as illustrated in Figure 7(a), the slip surface along the pile groups to the south occurs at a depth of about 4.1m below the ground surface. Accordingly, the slip force is assumed to

act behind the pile at a depth of 4.1m as shown in Figure 7(b). The magnitude of the slip force was estimated from the condition that the slippage occurs when the frictional resistance limit of slip surface is reached. Considering the possibility of slippage as a block of ground, three different slip lengths of 2m, 4m and 6.7m, with longest one corresponding to half width of the building, were considered for comparative analysis.

APPLICATION OF THE LOAD DISTRIBUTION METHOD

Authors have proposed and elaborated over the years the concept of load distribution method as a way of accounting for the response displacement effect in piles. They have also discussed comparison of such simple analytical results with more sophisticated analysis consisting of one and two dimensional time history analysis in accounting for the effects of ground response on stresses developed in piles during earthquakes (Sugimura et al, 1997, Karkee et al, 1999). Load distribution method is based on the fundamental equation of response displacement method as follows.

$$EI \frac{d^4 y}{dx^4} + k_h B \{y - f(x)\} = 0, \quad (1)$$

The notations used in equation 1 are as follows:

EI = Bending rigidity of pile

k_h = Coefficient of subgrade reaction

B = Breadth of pile

y = Horizontal displacement of pile

$f(x)$ = Horizontal displacement of the ground

x = Depth

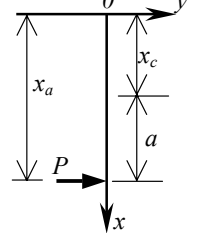
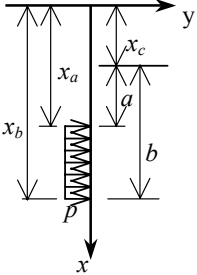
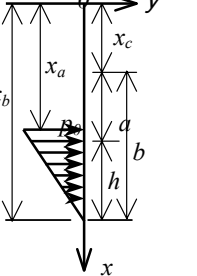
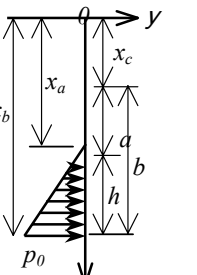
If we put $f(x)=0$ in equation 1, it becomes simply the ordinary fundamental equation representing horizontal resistance of pile subjected horizontal force at the top. Transforming equation 1, we have:

$$EI \frac{d^4 y}{dx^4} + k_h B y = k_h B f(x), \quad (2)$$

Because the term in the right hand side can be regarded as apparent external force, replacing $k_h B f(x)$ with an arbitrary external force distribution $p(x)$, we have:

$$EI \frac{d^4 y}{dx^4} + k_h B y = p(x), \quad (3)$$

Table 1. Theoretical solution for infinitely long pile under predefined lateral load distributions (Displacement y , rotation θ , bending moment M and shear force Q obtained by multiplying χ by μ .)

Concentrated Load P	Coefficient Values χ	Multiplier μ		Explanations
		$x_c < x_a$	$x_c \geq x_a$	
 k_h : Coefficient of subgrade reaction	$y_c = \frac{P}{8EI\beta^3}$	B_{3a}	B_{3a}	$\beta = \sqrt[4]{\frac{k_h D}{4EI}}$ $a = x_c - x_a $ $b = x_c - x_b $ $B_{1a} = e^{-\beta a} \cos(\beta a)$ $B_{1b} = e^{-\beta b} \cos(\beta b)$ $B_{2a} = e^{-\beta a} \sin(\beta a)$ $B_{2b} = e^{-\beta b} \sin(\beta b)$ $B_{3a} = e^{-\beta a} \{\cos(\beta a) + \sin(\beta a)\}$ $B_{3b} = e^{-\beta b} \{\cos(\beta b) + \sin(\beta b)\}$ $B_{4a} = e^{-\beta a} \{\cos(\beta a) - \sin(\beta a)\}$ $B_{4b} = e^{-\beta b} \{\cos(\beta b) - \sin(\beta b)\}$
	$\theta_c = \frac{P}{4EI\beta^2}$	B_{2a}	$-B_{2a}$	
	$M_c = \frac{P}{4\beta}$	B_{4a}	B_{4a}	
	$Q_c = \frac{P}{2}$	B_{1a}	$-B_{1a}$	
Uniform Load p	Coefficient Values χ	Multiplier μ		
		$x_c < x_a$	$x_a \leq x_c \leq x_b$	$x_c > x_b$
	$y_c = \frac{p}{8EI\beta^4}$	$B_{1a} - B_{1b}$	$2 - B_{1a} - B_{1b}$	$B_{1b} - B_{1a}$
	$\theta_c = \frac{p}{8EI\beta^3}$	$B_{3a} - B_{3b}$	$B_{3a} - B_{3b}$	$B_{3a} - B_{3b}$
	$M_c = \frac{p}{4\beta^2}$	$B_{2b} - B_{2a}$	$B_{2a} + B_{2b}$	$B_{2a} - B_{2b}$
	$Q_c = \frac{p}{4\beta}$	$B_{4a} - B_{4b}$	$B_{4a} - B_{4b}$	$B_{4a} - B_{4b}$
Inverse triangular Load p_0	Coefficient Values χ	Multiplier μ		
		$x_c < x_a$	$x_a \leq x_c \leq x_b$	$x_c > x_b$
	$y_c = \frac{p_0}{16EI\beta^5 h}$	$B_{4b} - B_{4a} + 2\beta h B_{1a}$	$B_{4b} - B_{4a} - 2\beta h B_{1a} + 4\beta b$	$B_{4b} - B_{4a} - 2\beta h B_{1a}$
	$\theta_c = \frac{p_0}{8EI\beta^4 h}$	$B_{1b} - B_{1a} + \beta h B_{3b}$	$B_{1a} + B_{1b} + \beta h B_{3a} - 2$	$B_{1a} - B_{1b} + \beta h B_{3b}$
	$M_c = \frac{p_0}{8\beta^3 h}$	$B_{3a} - B_{3b} - 2\beta h B_{2a}$	$B_{3a} - B_{3b} + 2\beta h B_{2a}$	$B_{3a} - B_{3b} + 2\beta h B_{2a}$
$Q_c = \frac{p_0}{4\beta^2 h}$	$B_{2a} - B_{2b} + \beta h B_{4a}$	$-B_{2a} - B_{2b} + \beta h B_{4a}$	$B_{2b} - B_{2a} + \beta h B_{4a}$	
Triangular Load p_0	Coefficient Values χ	Multiplier μ		
		$x_c < x_a$	$x_a \leq x_c \leq x_b$	$x_c > x_b$
	$y_c = \frac{p_0}{16EI\beta^5 h}$	$B_{4a} - B_{4b} - 2\beta h B_{1b}$	$B_{4a} - B_{4b} - 2\beta h B_{1b} + 4\beta a$	$B_{4a} - B_{4b} + 2\beta h B_{1b}$
	$\theta_c = \frac{p_0}{8EI\beta^4 h}$	$B_{1a} - B_{1b} - \beta h B_{3a}$	$2 - B_{1a} - B_{1b} - \beta h B_{3a}$	$B_{1b} - B_{1a} - \beta h B_{3a}$
	$M_c = \frac{p_0}{8\beta^3 h}$	$B_{3b} - B_{3a} + 2\beta h B_{2b}$	$B_{3b} - B_{3a} + 2\beta h B_{2b}$	$B_{3b} - B_{3a} - 2\beta h B_{2b}$
$Q_c = \frac{p_0}{4\beta^2 h}$	$B_{2b} - B_{2a} - \beta h B_{4b}$	$B_{2a} + B_{2b} - \beta h B_{4b}$	$B_{2a} - B_{2b} - \beta h B_{4b}$	

While the external force distribution restricted to $k_h Bf(x)$ in equation 2, the term $p(x)$ in equation 3 may be selected arbitrarily, such that any general load condition, such as shown in Figure 7(b), may be considered in arriving at the solution.

Assuming the pile to have infinite length, general solutions for displacement or stresses for predefined load distributions, such as concentrated load, uniform load, triangular load, can be readily obtained from Hetenyi (1946) as shown in Table 1. It may be noted from the table that when the solutions for the cases of triangular and inverse triangular load distributions are added, the result logically corresponds to the solution for uniform load. Authors have also utilized this approach and prior examples of application can be found in Sugimura and Karkee (2001) and Karkee and Sugimura (2002). In order to consider the situation of the nonexistence of subgrade reaction at the upper part above the slip surface, as may be considered to be the actual condition, the effect of drastic reduction in the coefficient of subgrade reaction above the slip surface was evaluated.

ANALYTICAL EXAMPLE

Nakazawa et al. (1999) have provided details of the building and foundation system. According to them, the building consisted of 11 storied SRC structure with span between columns in longitudinal direction of 6.3m. Accordingly, the width of the slip surface corresponding to a pile group was considered to be 6.3m. The foundation beams were 0.4m in width and 1.8m in depth, 1.5m of which is embedded. There were six prestressed concrete (PC) piles of grade A (effective prestress $\sigma_e=3.92\text{N/mm}^2$) in the group supporting the target foundation slab of thickness equal to depth of foundation beam. The PC piles had outer diameter of 50cm, wall thickness of 9cm and length of 26m consisting of two 13m segments joined together. The design bearing capacity of single piles was 764kN.

The reclaimed layer (fill material) in Figure 7 consists of sandy material containing some gravel, the unit weight of which is assumed as $\gamma=18\text{kN/m}^3$. Based on the available information, the average value of standard penetration test blow count N is estimated to be about 15. The shear strength s of the fill material may then be evaluated as 50kN/m^2 based on the empirical relation in Japanese practice of $s=10N/3$ (kN/m^2). As mentioned above, the shear resistance of sandy material may be considered as resisting slip, and when the slip occurs slip force equivalent to the frictional resistance over the slip surface may be regarded to have mobilized. Based on AIJ (2001), the coefficient of subgrade reaction may be estimated from the following equation:

$$k_{h0} = \alpha \xi E_0 B^{-3/4} \quad (4)$$

Here,

k_{h0} = Normalized coefficient of subgrade reaction (kN/m^3)

E_0 = Modulus of deformation (kN/m^2), which for the N-value works out to be

$$E_0 = 700 \text{ (the coefficient } \alpha = 80 \text{ in this case)}$$

ξ = Influence factor for pile group = $0.15\kappa + 0.10$ for the case $\kappa \leq 6$

κ = Ratio of center to center interval of piles to pile diameter (2.5 in this case, such that $\xi = 0.475$)

B = Non-dimensional coefficient of pile diameter (50 in this case)

Substituting appropriate values in equation 4, the value of $k_{h0} = 2.122 \times 10^4$ (kN/m^3) was obtained. Calculation of response under trapezoidal lateral and the concentrated force at the slip surface acting on the upper part of pile as shown in Figure 7 can be undertaken. The trapezoid load can be divided into uniform and triangular distributions as illustrated in Figure 7(b). Displacement and stresses in pile were calculated at the 86 points from pile top at GL-1.5m to the depth of GL-10m at 0.1m interval and the results are shown in Figures 8 and 9.

DISCUSSION OF THE ANALYTICAL RESULTS

Analytical results shown in Figure 8 include displacement, bending moment and shear force in piles for various lengths of slip surface, where the concentrated load P of 105.0kN, 210.0kN and 351.8kN correspond to slip surface lengths of 2m, 4m and 6.7m respectively. Similarly, Figure 9 shows the case of $P=351.8\text{kN}$ in Figure 7(b) with the coefficient of subgrade reaction reduced to 10% and 1% of the normal value obtained from equation 4 for the portion of fill material above the slip surface. As mentioned above, the drastic reduction attempts provide analogy to the case of practically nonexistent subgrade reaction due to liquefaction.

It may be noted that all the Figures 8 and 9 show the respective response curves corresponding to concentrated load, distributed load, and triangular load, separately as well as the synthesis (total) of those denoted by the thicker solid line. For comparison, the ultimate bending resistance M_u (kNm) and ultimate shear resistance Q_u (kN) of the pile member is shown by vertical dashed lines in the respective diagrams in Figures 8 and 9. Ultimate bending moment M_u was determined from the relation of the ultimate axial force and bending moment ($M-N$ interaction curve) corresponding to the design bearing capacity of piles, which is 764kN as noted above. Q_u was determined by comparing the results by equation 5 (AIJ, 1990) and equation 6 proposed by Kishida et al. (2000).

$$Q_u = \frac{2tI}{S_0} \frac{1}{2} \sqrt{(\sigma_g + 2\phi\sigma_t)^2 - \sigma_g^2} \quad (5)$$

$$\sigma_g = \sigma_e + \frac{N}{A_e}$$

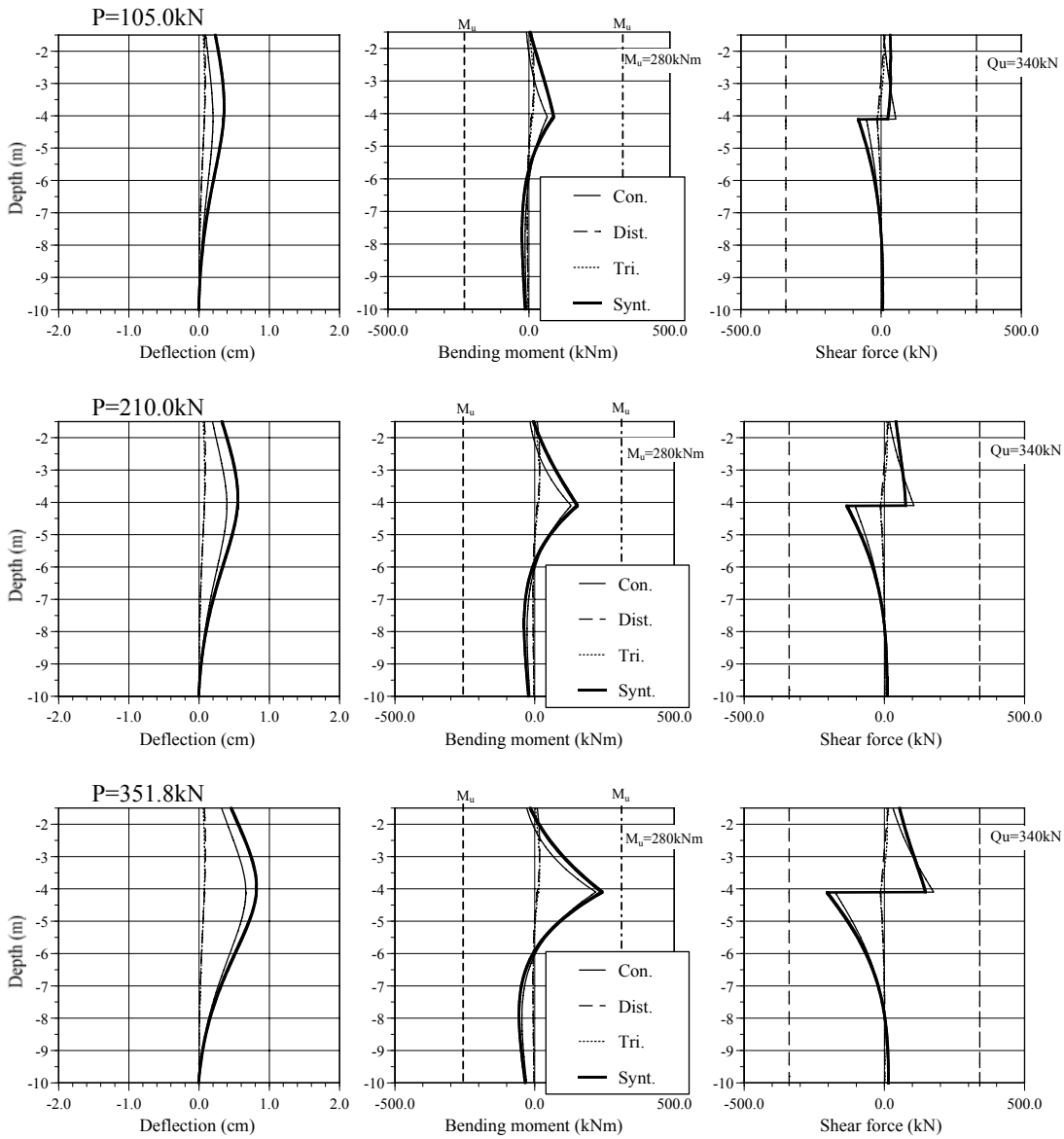


Figure 8. Variation of the extent of pile response obtained from the load distribution method depending on changes in length of slip surface, and hence the magnitude of concentrated load P.

In equation 5, the notations are as follows:

t = Thickness of the hollow circular pile section

I = Second moment of area of the section

S_0 = First moment of area of the section

σ_e = Effective prestress

σ_t = Tensile strength of concrete (about equal to 7 % of compressive strength)

N = Axial load

A_e = Equivalent transformed section area of concrete

ϕ = Correction coefficient corresponding to experimental results (usually taken as 0.5)

$$Q_u = \tau_{\max} b_e j \quad (6)$$

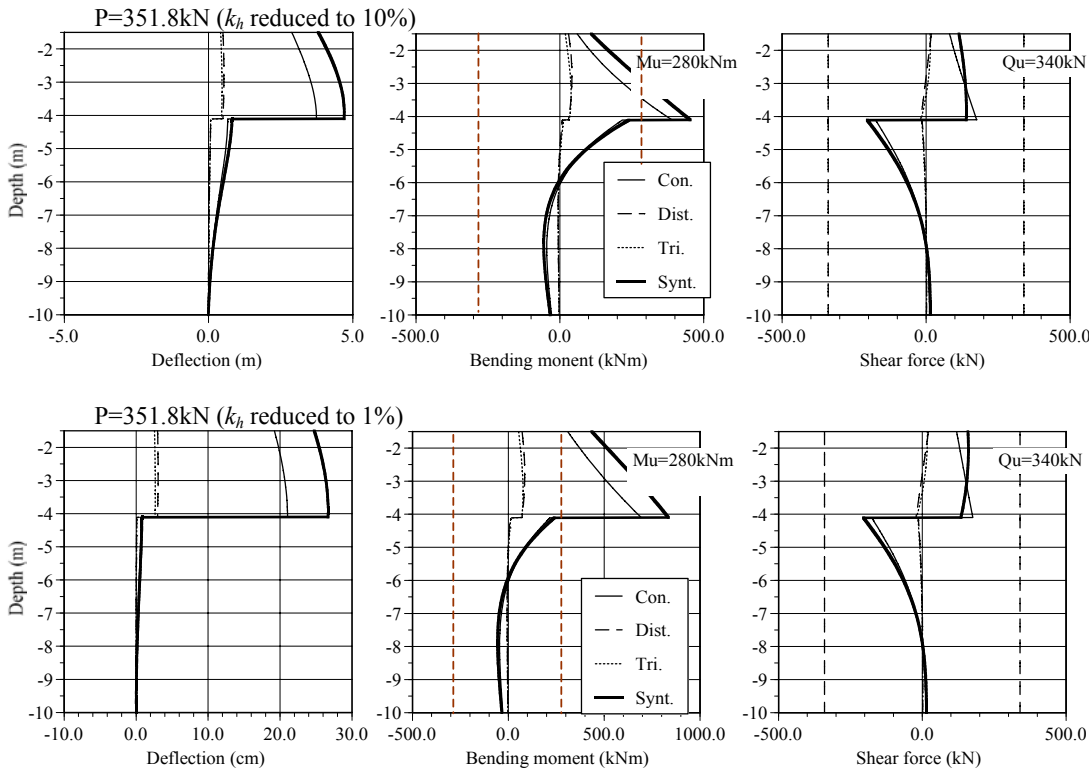


Figure 9. The effect of drastic reduction in coefficient of subgrade reaction above the slip surface

In equation 6, the notations are as follows:

$$b_e = \{-1.24(t/D) + 1.19\} A_c / D$$

$$j = \frac{7}{8} d = \frac{7}{8} \left\{ D - \frac{t}{2} \right\}$$

$$\tau_{\max} = \tau_1 + \tau_2 + \tau_3 \quad (\text{For design of pile members})$$

$$\tau_1 = \frac{0.115 k_u k_p (\sigma_B + 17.7)}{(M/Qd) + 0.115}$$

$$\tau_2 = 0.657(0.785p_w \cdot_w \sigma_y) \quad (0.785p_w \cdot_w \sigma_y \leq 7.4) \text{ (N/mm}^2\text{)}$$

$$\tau_3 = 0.102(\sigma_e + \sigma'_0) = 0.102 \left\{ \sigma_e + \frac{N'}{b_e j} \right\} \quad (\sigma_e + \sigma'_0 \leq 27.4) \text{ (N/mm}^2\text{)}$$

Where,

D = Outer diameter of pile (mm)

$A_c = \pi(D - t)$ = Section area of pile (mm²)

τ_1 = Shear stress shared by concrete (N/mm²)

τ_2 = Shear stress shared by spiral hoop (N/mm²)

τ_3 = Shear stress shared by axial force (N/mm²)

σ_c = Compressive strength of concrete (N/mm²)

N' = Axial force (N)

d = Effective depth (mm)

${}_w \sigma_y$ = Strength at yield point (N/mm²)

k_u = Modification factor for section size ($k_u = 0.72$ in this case)

k_p = Modification factor depending on tension reinforcement p_t as shown in the following equation,

$$k_p = 0.82(100p_t)^{0.23}, \quad p_t = p_g/4 \quad (7)$$

p_g = Main reinforcement ratio ($p_g = A_s/b_e j$)

p_w = Spiral hoop ratio ($p_w = a_w/b_e s$)

A_s = Total area of main reinforcement (mm²)

a_w = Area of spiral shear reinforcement (mm²)

s = Interval of spiral hoop (mm)

M/Q_d = Shear span ratio (Since based on the experience of authors (Sugimuta et al., 1984), it is clarified that a shear span ratio of nearly equal 1.0 lies in the border between bending failure and shear failure of PHC piles type A, the value of 1.0 was used in the analysis.)

Considering the axial force to be equal to design bearing capacity of single pile (764kN), the value of Q_u were obtained as 277kN and 340kN respectively from equation 5 and equation 6. The result seems to correspond well with the recent thinking that equation 5 agrees with shear crack capacity while equation 6 tends to conform to the ultimate shear capacity. Accordingly, equation 6 was adopted for estimation of Q_u for the pile member.

Figure 8 shows the effect of concentrated load dominates the response of pile. As such the bending moment as well as the shear force at the slip surface at GL-4.1m increases with the length of slip surface. The ratio of the maximum load effect to the ultimate capacity is about 80% in bending moment and about 60% in shear force.

On the other hand, the situation is seen to change completely in Figure 9. When the coefficient of subgrade reaction is reduced to 10% for the case of slip surface length of 6.7m, the maximum shear force becomes a little larger and distribution above the slip surface changes a little, but the change is limited over a small range on the whole. Distribution of bending moment at the slip surface, however, becomes much larger and the maximum bending moment exceeds the ultimate capacity. And displacement also becomes discontinuous in distribution at the slip surface, which is extremely different from usual case. When the coefficient of subgrade reaction reduced further to 1%, these tendencies become more prominent.

Based on the analytical results, damage features observed in piles may be explained analogically as follows. The piles are subjected to forced deformation, which tends to be discontinuous at the slip surface. Accordingly, not only the bending moment reaches easily the ultimate capacity but also the effect of direct shear failure contributes to overall response. The authors are aware that the several assumptions considered in the process of analysis by load distribution method may not provide a solid basis for quantitative aspects of the problem, such as the aspects of whether the stresses in pile exceed ultimate capacity or not. However, the authors wish to emphasis that the consideration of comparatively unique loading condition, such as the introduction of a combination of concentrated and distributed load discussed above, is effective and necessary to introduce in order to clarify the aspects of damage actually observed. Alternatively, while attempting to investigate pile behavior by response displacement method, it is necessary to consider the condition of deformation discontinuity at locations where a slip surface may be expected to develop in the ground under the action of severe earthquakes.

CONCLUSIONS AND FUTURE RESEARCH

Attempt was made to clarify the mechanism of K-shaped failure at deeper part of piles, observed during the 1995 Hyogoken-Nambu earthquake. This involved review of existing relevant reports and selection of a particular structure for analytical interpretation by applying the load distribution method proposed earlier by the authors. The analysis was based on the hypothesis that damage of the ground due to lateral flow resembles occurrence of slump in landslide. That is, considering the effect of unbalanced horizontal stress due to lateral flow of liquefied soil layer, while the horizontal earth pressure on one side of the pile is absent as if removed by excavation. The analysis is based on the assumption of an infinitely long pile subjected to a trapezoidal load representing lateral pressure and the concentrated load representing the action at the slip surface. The analysis also include the cases with drastically reduced coefficient of subgrade reaction for the part of soil above the slip surface as a way to analogically represent the nonexistence of subgrade reaction in liquefied soil.

The analytical results indicate general tendency that the pile would be subjected to displacement discontinuity at the slip surface. Bending moment easily reaches ultimate capacity of pile member, particularly at the slip surface. In addition, the effect of relatively large shear force is likely to contribute to failure. In conclusion, it may be emphasized that the consideration of a comparatively unique loading condition to represent ground condition and its possible failure mechanism is necessary to realistically represent K-shaped failure of piles. In addition, consideration of such unique loading condition is expected to be an effective approach in design practice.

In order to represent the actual condition more realistically, it is necessary to introduce the boundary condition at pile top, effect of axial force, reduction in strength of the ground due to liquefaction, the elasto-plastic theory or large deformation theory of piles etc. in the analysis. These aspects remain to be investigated and evaluated in future.

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