EVALUATION OF DUCTILITY FOR RC PILES SUBJECTED TO CONFINING PRESSURE IN THE GROUND

Toshinari IMAMURA¹, Yoshitaka MURONO² and Takuhiro NAGAO³

¹Member of JSCE, Researcher, Structures Technology Division, Railway Technical Research Institute
(2-8-38 Hikari-cho, Kokubunji-city, Tokyo 185-8540, Japan)
E-mail: imamura@rtri.or.jp

²Member of JSCE, Chief researcher, Structures Technology Division, Railway Technical Research Institute
(2-8-38 Hikari-cho, Kokubunji-city, Tokyo 185-8540, Japan)
E-mail: murono@rtri.or.jp

³Member of JSCE, Central Japan Railway Company
(Temporary Transferred to Railway Technical Research Institute)
(1545-33 Ohoyama, Komaki-city, Aichi 485-0801, Japan)
E-mail: nagao-t@jr-central.co.jp

In the seismic design codes for railway structures, a deformation capacity of RC piles is evaluated based on the experimental results, which have been obtained from alternate loading tests of RC columns under the atmospheric conditions. The actual behavior of piles, however, is likely to show a greater degree of ductility than these experimental results, because piles embedded in the ground are subject to confining pressure from the subgrade. In this study, we aimed to establish a rational evaluation method of seismic performance of pile foundations. A calculation method of ductility of RC piles is proposed, in which confining pressure by the subgrade reaction is converted into an equivalent passive confinement pressure from hoop reinforcements.

We have conducted alternate loading tests of RC columns supported by coil springs instead of the ground. We confirmed the applicability of the newly developed method by comparing these experimental results with numerical simulation results. In addition, we calculated the seismic response of railway structures with general shape and size using the proposed model, and confirmed the applicability of the proposed model to railway structures.

Key Words: reinforced concrete pile, moment-curvature relationship, confining pressure induced by subgrade reaction

1. INTRODUCTION

It is essential to evaluate the bending deformation characteristic of structural members appropriately in a seismic design of a large-scale earthquake like ‘Level 2’ earthquake. Various researchers¹-⁴ have performed many loading experiments with models of a full-scale or reduced-size-scale to study on the bending deformation characteristic of the reinforced concrete, and clarified the following. If lateral confining reinforcements are closely arranged, (1) the compressive buckling or swelling of longitudinal reinforcements are controlled and then (2) the ductility of the member is improved because of the confining effect on the concrete core. In addition, the following methods have been proposed to evaluate the bending deformation characteristic of structural members: (1) The member is modeled by the beam element, and its bending deformation characteristic is defined at the element level based on the result of the alternate loading experiment³. (2) The bending deformation characteristic of member is calculated with a fiber element or a finite element, giving the constitutive law to reinforcement and concrete⁵.

Most of the loading experiments, however, have been performed under atmospheric conditions. It is therefore necessary to examine the applicability of the bending deformation characteristic obtained by these studies to the structural members such as a pile,
which receives the confining pressure from the ground.

In addition, many loading experiments of piles embedded in the ground have been performed so far, and most of them have regarded the effect of a pile group or to a relative rigidity as an interaction problem between the pile and the ground. A few researches has been made to examine the bending deformation characteristic of RC piles in view of the confining effect induced by subgrade reaction. Then, the authors have executed alternate lateral loading experiments of RC piles supported by coil springs as a simulated ground, to grasp the influence of the confining effect induced by subgrade reaction on the bending deformation characteristic of RC piles. As a result, the RC piles subject to the confining pressure are demonstrated to have the greater ductility than the RC members under atmospheric conditions do 6).

In this paper, we made the following studies based on the above-mentioned experimental result: First, we proposed an evaluation method of the bending deformation characteristic of RC piles by converting the confining pressure induced by subgrade reaction into an equivalent passive confining pressure from hoop reinforcement. Next, the above-mentioned experimental result was likely to indicate the actual deformation characteristic of RC piles; therefore, we conducted numerical simulations and confirmed the validity of the proposed method. Eventually, the proposed model proved to be applicable to existing structures through a test design.

2. CYCLIC LOADING EXPERIMENTS ON THE SIMULATED GROUND OF COIL SPRINGS

(1) Experimental summary

In order to clarify a bending deformation characteristic of RC piles embedded in the ground, the authors have developed loading tests apparatus, in which the simulated ground of coil springs support the RC specimens. We conducted a series of alternate cyclic loading experiments with this apparatus. Here, we introduce the experimental summary of H13-UNIT2, which shows a flexural failure mode.

The test specimen is of a square section of 300 mm × 300mm, and the length is 4.3 m from the loading point (the pile top) to the pile tip. Longitudinal reinforcements are 16-D13, and hoop reinforcements are D10-ctc75mm. Area ratio of tensile longitudinal reinforcement is 0.81%, and area ratio of hoop reinforcement is 0.63%. The reinforcement ratios are set in covering the bending deformation characteristic of RC piles ruled in the current “The Seismic Design Standard for Railway Structures in Japan”. Table 1 and Table 2 indicate material properties of both reinforcement and concrete used, respectively.

Twelve coil springs are installed on the surface of the test specimen from a point 450 mm away from the pile top to the pile tip at intervals of 300 mm. The stiffness coefficient of coil springs is 2.5 kN/mm, and is set in such a manner to behave within the range of linear elasticity.

The amplitude of the specified displacement at the loading point is gradually increased from $\delta_y$ to $n \times \delta_y$ (n = 0.5, 1.0, 1.5 successively); where $\delta_y$ is the displacement of the loading point ($\delta_y$ = 29mm), when a strain of the longitudinal reinforcement arranged at the outside edge of RC piles has reached to yield strain. Three cycles are repeated at each specified displacement in positive and negative sides.

(2) Experimental results

Fig.1 indicates a load versus displacement response at the loading point. A solid line in this figure indicates the experimental result. A dotted line (conventional model) shows the result of numerical simulation, using the bending deformation characteristic of RC piles as stipulated by the current “The Seismic Design Standard for Railway Structures”7). An alternate long and short dash line (proposed model) indicates the analytical result based on the

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First, we regarded the experimental results. When the load has reached 108 kN, longitudinal reinforcement on the outside edge yielded, and the bearing strength of the RC piles has risen monotonically as displacements increased up to $4.0\delta_y$. We confirmed the collapse of concrete core and the buckling tendency of the longitudinal reinforcement, depending on the crack observation on the surface of concrete, under the first cycle of $4.5\delta_y$, likewise under the third cycle, the cover concrete spalled, and the longitudinal reinforcements buckled. Finally, at the first cycle of $5\delta_y$, the cover concrete fell off as large slabs, and in the second cycles the longitudinal reinforcements in the compression side fractured. Then, we observed a large and abrupt decrease in the strength. Therefore, we judged that this test specimen indicated the maximum bearing strength at $4.5\delta_y$ and ultimate strength at $5.0\delta_y$.

Secondly, we remarked the result of numerical simulation based on the conventional model as shown with dotted line in this figure. We confirmed that an analytical value underestimates the experimental result by about 20% in displacement at the maximum bearing strength. Thus, it is understandable that RC piles subjected to confining pressure have a greater ductility than that calculated according to the current “The Seismic Design Standard for Railway Structures”.

3. PROPOSED MODEL FOR A BENDING DEFORMATION CHARACTERISTIC OF RC PILES SUBJECTED TO CONFINING PRESSURE

(1) Basic concept

In this Chapter, the authors have proposed a method to convert the confining pressure by subgrade reaction into the equivalent passive confining pressure from hoop reinforcements, with respect to the confining pressure of concrete core.

First, we considered how hoop reinforcements confine the concrete core. In the RC column where hoop reinforcements are arranged, concrete core starts to expand horizontally when the compression stress induced by the bending deformation acts. At this time, hoop reinforcements are subject to tensile stress on the circumferential direction, and concrete core receives confining pressure $f'_c$ on the radial direction from the hoop reinforcement as a reaction.

Thus, hoop reinforcements have a capacity to confine the concrete core as a reaction when the bending deformation occurred. Hereinafter, this confining pressure is referred to “confining pressure from hoop reinforcement”. On the other hand, the RC column under the ground receives confining pressure induced by the surrounding soils, and the confining pressure always acts regardless of the deformation of RC columns as referred to “confining pressure by earth pressure”.

Although such a difference between both confining pressures is visible in affecting, no difference is admitted in the sense that both pressures can also confine the concrete core when the plastic hinge is formed. Moreover, if the amplitude of both confining pressures is the same, it is reasonable to assume that the ductility of RC columns is also the same. Then, assuming the confining pressure acted on concrete core to be an index of improvement of the ductility, a method to convert the confining pressure induced by subgrade reaction into the pressure from hoop reinforcements is devised.

In converting the confining pressure as explained above, it is necessary to note that there is an area of ineffectively confined concrete core between adjacent hoop reinforcements, because the hoop reinforcement is discretely arranged at specified intervals as shown in Fig.2. To take account of this aspect, the concept of confinement effectiveness coefficient $k_e$ that Mander et al. proposed was intro-
duced. Here, the confinement effectiveness coefficient $k_e$ is a ratio of an area of concrete core enclosed by hoop reinforcement $A_{cc}$ (strictly, one that the sectional area of longitudinal reinforcement was subtracted) to area of concrete core actually confined $A_c$, a confined area as shown in Fig.2, calculated by $k_e = A_c / A_{cc}$.

Concerning a circular cross section, if the stress distribution along the axis of members is likely to occur in a form of the second-degree parabola, which starts from the position of arranged hoop reinforcements with an initial tangent slope of 45 degrees, a confinement effectiveness coefficient $k_e$ is obtained by

$$k_e = \left(1 - \frac{s'}{2d_c}\right)^2 \sqrt{\frac{1 - \rho_{cc}}{1 - \rho_{cc}}}, \quad (1)$$

Where $s' = $ clear vertical spacing between adjacent hoop reinforcements; $d_c = $ diameter of hoop reinforcement between bar centers; and $\rho_{cc} = $ ratio of area of longitudinal reinforcement to area of core of section.

In the case of the rectangular section, it is noticeable that a decrease in the stress occurs not only vertically between layers of hoop reinforcements but also horizontally center part of sides of hoop reinforcement. Thus, confinement effectiveness coefficient $k_e$ is expressed by

$$k_e = \left(1 - \frac{w'_c}{6 \cdot b_c \cdot d_c}\right) \left(1 - \frac{s'}{2d_c}\right) \left(1 - \frac{s'}{2d_c}\right) \left(\frac{1 - \rho_{cc}}{1 - \rho_{cc}}\right), \quad (2)$$

Where $w'_c = $ clear distance between adjacent longitudinal reinforcements; $b_c$ and $d_c = $ core dimensions to centerlines of perimeter hoop in $x$ and $y$ directions, respectively, where $b_c \geq d_c$.

(2) Equivalent area ratio of hoop reinforcement

Based on the above-mentioned discussions, the confining pressure induced by subgrade reaction is converted into the equivalent confining pressure from the hoop reinforcement.

If the concrete core enclosed in hoop reinforcement does not have a loose region, (i.e. the full cross section is confined), the equilibrium of forces between concrete core and hoop reinforcement is as shown in Fig.3. In the case of the circular cross section, the confining pressure of concrete core $f_i$ is obtained by

$$f_i = \frac{2 f_{sh} \cdot A_{hp}}{s \cdot d_c}, \quad (3)$$

Where $f_{sh} = $ yield strength of the hoop reinforcement; $A_{hp} = $ area of hoop reinforcement; and $s = $ center-to-center spacing of circular hoop. By multiplying this by confinement effectiveness coefficient $k_e$, the effective confining pressure of concrete core $f'_i$ that considers loosening region along the axis of member can be calculated. From the viewpoint of improvement of ductility, it is reasonable to assume that the confining pressure calculated thus $f'_i$ is equal to the confining pressure by earth pressure $p$. Thus, Eq. (4) is approved.

$$p = f'_i \quad (4)$$

Because the area ratio of hoop reinforcement $p'_c$ is defined as $p'_c = (2 \cdot A_{hp})/(D \cdot s)$, the equivalent area ratio of hoop reinforcement $p'_e$ that gives the same confining pressure as subgrade reaction is obtained by

$$p'_e = \frac{p \cdot d_c}{k_e \cdot f_{sh} \cdot D}, \quad (5)$$

Where $D = $ a diameter of pile.

Similarly, in the case of the rectangular section, the equivalent area ratio of hoop reinforcement is expressed by

$$p'_e = \frac{p \cdot b_c}{k_e \cdot f_{sh} \cdot B}, \quad (6)$$

Where $B = $ the length of the long side in rectangular cross section.

(3) Moment-curvature relationship

In designing a structure using the proposed moment-curvature relationship, it is important that: (1) the proposed model has a representation ability to increase the ductility smoothly and continuously
from the state of confining pressure zero, and that (2) it can be simply calculated. Then it is decided that the moment-curvature relationship of RC piles subjected to confining pressure is expressed based on the tetra-linear model provided for the current “The Seismic Design Standard for Railway Structures”7). Then, the rotational angle in the plastic hinge is calculated by adding the equivalent ratio of hoop reinforcement obtained by Eq. (5) or Eq. (6) to the ratio of hoop reinforcement actually arranged.

4. VERIFICATION OF EFFECTIVENESS

(1) Summary of examination
In this Chapter, we verify the validity of the proposed model by executing the numerical simulation of the experimental results of the model pile as described in Chapter 2, where the results are likely to indicate the actual behavior of RC piles.

The equivalent area ratio of hoop reinforcement is obtained from Eq. (2) and Eq. (6), because its cross section is square. Then, the bending deformation characteristic is calculated. The confining pressure $p$ from the surrounding soils is obtained by dividing the maximum value of the coil spring’s reaction force in both the width (0.3m) of a model pile and the interval (0.3m) of coil springs. Fig.4 shows an example of the calculated moment-curvature relationship. From the figure, it is understandable that the curvature at the maximum strength estimated by the proposed model has improved by 40% compared with that calculated by the conventional model.

(2) Analysis results
We regarded the load versus displacement response at the loading point, and made a comparison between the conventional model and the proposed model as referred in Fig.1. The displacement at the maximum strength obtained by the experiment is $4.5\delta_y$. Thus, $3.7\delta_y$ that is the displacement given by the conventional model undervalues the experimental value. On the other hand, $4.2\delta_y$ by the proposed model can roughly explain the experimental result.

Fig.5 describes the bending moment diagram. The bending moment obtained by the conventional model indicates almost zero near the maximum strength (at $4.2\delta_y$), because the decrease in strength occurred in the early stages. On the other hand, the proposed model can express the experimental result satisfactorily until the loading displacement reaches $4.2\delta_y$.

The plastic hinge formed in part ‘a’ of Fig.5 in both the experiment and the analysis. Fig.6 shows moment-curvature relationship around this part. It is at the first cycle of $4.5\delta_y$ that the collapse of concrete core and the buckling tendency of longitudinal reinforcements were observed as explained earlier. In addition, the bending moment and curvature at the maximum strength (hereinafter, M-point) obtained by the proposed model are roughly corresponding to the experimental result. Further, moment-curvature response obtained by the proposed model reaches M-point when displacement at the loading point is
Thus the proposed model can also be expressed the experimental result in view of the progress of deformation.

**Fig.7** depicts the curvature diagram (at $4.2\delta$). The solid lines in this figure indicate the experimental result. The dotted lines show the result of numerical simulation obtained by the conventional model. The alternate long and short dash lines indicate the result of analysis by the proposed model. Added to these lines, the limit curvature corresponding to ‘damage level 2’ (i.e. curvature at the maximum strength, $\phi_m$), which is obtained by the proposed model, is also represented in this figure. It is confirmed that (1) the limit curvature corresponding to ‘damage level 2’ can express the generated curvature when the longitudinal reinforcements are buckled, and that (2) the distribution of the curvature obtained by the proposed model can express the experimental result satisfactorily, contrary to that the damage region is localized at the limited area in the conventional model.

Therefore, the validity of the proposed model can be verified.

### 5. APPLICATION TO EXISTING STRUCTURES

#### (1) Examination summary

In this Chapter, the application of the proposed model to the existing structure is verified to execute a test design.

**Fig.8** shows an analytical model for the examination of structure. The structure is supported by a group-pile foundation made of $2 \times 2$ piles, and the dimensions of piles are 1300 mm in diameter and 22 m in length. Its column is of the rectangular section of 5000 mm x 1800 mm. The compressive strength of concrete is 24 N/mm² for the column and the footing, and is 30 N/mm² for the pile. The material property of the reinforcement is SD390.

**Fig.8** also shows the ground condition; the surface layer is an ordinary ground which is mainly composed of a sand layer with N-value of approximately 20 (the predominant period of the ground motion is 0.26 sec, and ‘G3 ground’ according to the classification of the current “The Seismic Design Standard for Railway Structures”[7]). The bedrock is a diluvial gravelly layer with N-value of 50. No consideration is given to liquefaction in this case. A nonlinear response spectrum method was adopted as an analytical technique. As an earthquake load, the ‘Level 2’ earthquake (especially, ‘spectrum II’ wave) was assumed which is stipulated by the current “The Seismic Design Standard for Railway Structures”[7].

Where the proposed model adopted as a pile-bending characteristic, we discussed how much subgrade reaction would act on RC piles. Considering that concrete core expands horizontally and that the longitudinal reinforcements swell or buckle on the compression edge of cross section, it may be reasonable to estimate the passive earth pressure as the confining pressure. However, no verification of the fact has been made with loading tests for underground piles; under such circumstances, it may be appropriate to assume the confining pressure as a
mean value of the earth pressure that acts on the front and backsides of piles.

In addition, occurrence of a gap between the pile and the ground is likely within a range of $1/\beta$ from the pile top during an earthquake. Therefore, we have not considered the confining pressure in the moment-curvature relationship of piles in this region. We also made comparisons on the conventional model in accordance with the current seismic analysis.

(2) Results of numerical simulation

Fig. 9 shows the load versus displacement curve in the top of the structure, which is calculated based on the conventional model and the proposed model. Fig. 10 likewise shows the moment-curvature relationship in element 30, where the plastic deformation is remarkably visible. The structure objective for the examination is of a wall type pier. The flexural strength of the column is so large that the response seismic coefficient of the full system of the structure considerably rises. Then, variable components of the axial force of piles grow so large that a compressive axial force acted on the pile in compressive side exceeds the balanced axial force, hereinafter referred to a high compressive axial force member. The moment-curvature relationship of such a high compressive axial force member is expressed as the tri-linear model which runs through the origin. As shown in Fig. 10, a C-point where a crack generates, a CU-point where concrete shows a compression failure as soon as the longitudinal reinforcements are buckled, and a CN-point where a large volume of cover concrete falls off, the compression buckling of the longitudinal reinforcements occur, and then a rapid decrease of the flexural strength is caused.

As shown in Fig. 9, the load versus displacement curve obtained by the proposed model is completely corresponding to that given by the conventional model. There is no difference between both of them in the behavior of the structure. It is conceivable that the cause includes the following aspects: (1) the structure for the examination is of a wall type pier, and the area of damage is limited to piles, (2) the pile in compressive side becomes a high compressive axial force member. Then, the skeleton curve given by the proposed model is the same as that by the conventional model, because the stiffness coefficient after the CU-point becomes zero in a high compressive axial force member.

We remarked the limit curvature ($\phi_{\text{cnd}}$) corresponding to ‘damage level 2’. As shown in Fig. 10, it is confirmed that the limit curvature obtained by the proposed model is larger than that by the conventional model. On the other hand, the value of the response curvature obtained by the proposed model is the same as that given by the conventional model, because the skeleton curves used in both of analyses are the same. If the indexes of safety verification ($\gamma_i \cdot \phi_i / \phi_{\text{cnd}}$) obtained by both of the models are compared, it is understandable that the value by the conventional model becomes 0.93 and that the value by the proposed model becomes 0.84. Therefore, it is confirmed that the proposed model can bring an economic design of about 10% in view of the index of safety verification. It is also confirmed that the subgrade reaction that acts on element 30 reaches the effective subgrade reaction. Namely, it is confirmed that the precondition, which is assumed in the proposed model, is satisfied.

6. CONCLUSIONS

The bending deformation characteristic of RC member (pile) embedded in the ground is likely to indicate a greater ductility than that obtained under the atmospheric conditions. The authors regarded
such a fact, proposed the evaluation method, and examined the applicability to existing structures. The findings obtained under the studies comprise the following:

(1) The method of converting the confining pressure induced by subgrade reaction into the equivalent confining pressure from the hoop reinforcements was proposed, and the validity was verifiable.

(2) The test design was conducted using the proposed model. It was evident that the ductility improved approximately by 10%, when a-half of the passive earth pressure was considered as the confining effect by the ground.

The authors will execute continuously the alternate lateral loading experiments of piles in the ground, and make an endeavor to deepen the examination.

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