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**A structural model of RC members subjected to severe
earthquake loads and its application to assess the seismic
performance of pile foundations**

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**Foundation Engineering Research Team,
Structures Research Group,
Public Works Research Institute**

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A structural model of RC members subjected to severe earthquake loads and its application to assess the seismic performance of pile foundations

by

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Synopsis: When pile foundations experience a severe earthquake, it is generally assumed that postpeak bending behavior emerges locally in the pile at the top and near the boundaries between different soil layers in the ultimate limit state. Experimental results for typical cast-in-place RC piles indicate that the postpeak behavior is mainly caused by the collapse of cover concrete and the local reinforcement swelling. In this study, we propose a simple constitutive model for fiber elements. The proposed model uses a plastic buckling model to describe the swelling of reinforcing bars. The plastic buckling length of the reinforcement bar is used as the element length. Numerical verification using this proposed model shows approximate agreement with experimental results for model piles under cyclic loading. Furthermore, a couple of examples are demonstrated. One is a comparison of toughness of RC piles in air and in ground. A model RC pile embedded in sand and subjected to lateral cyclic displacement at the top is computed using the proposed model with and without consideration of lateral confinement to the cover concrete of the pile due to the earth pressures of the surrounding soil. The other is a dynamic analysis using the proposed model of a bridge pile foundation. Finally, we propose design conditions and criteria for pile foundations likely to suffer from seismic-induced ground vibration.

Keywords: *pile foundation, postpeak behavior, swelling of reinforcing bar, kinematic interaction, seismic design*

強震動を受ける RC 棒部材の数値モデルと杭基礎の耐震性能評価への適用

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要 旨

繰返し曲げ荷重を受ける場所打ち杭の曲げ耐力の低下は、かぶりコンクリートのはく離、鉄筋のはらみ出しに起因することが実験から明らかになってきている。そこで、本資料では、杭のような RC 棒部材をファイバー要素を用いてモデル化することを前提に、はらみ出しを考慮した鉄筋の構成則とその構成則を用いる場合の要素長の設定法を提案する。ここに、はらみ出しは鉄筋の塑性座屈としてモデル化する。提案する構成則と要素長の設定方法を用いた有限要素解析は、場所打ち杭の正負交番荷重実験結果で観察された耐力低下挙動を再現することができた。また、この構成則を用いた基礎の耐震性能評価に関する 2 つの応用例を示す。一つは杭のじん性に与える地盤からの横拘束土圧の影響を定量的に評価することを試みた例である。もう一つは、地中深部において杭体が地盤の振動変位により大きな損傷を受ける場合の杭基礎・橋脚系の挙動を動的解析により把握した例である。これらの 2 つの応用例から、地盤の振動変位作用に対する杭基礎の耐震設計法を検討した。

キーワード：杭，耐力低下挙動，動的相互作用，耐震設計

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Contents

1	Introduction	1
2	Fundamental Limit of the Ability of Existing Constitutive Models in Prediction of Postpeak Behavior of Cast-in-place RC Piles	6
(1)	Experiment on a type-1 pile specimen subject to a horizontal reversed cyclic load with a constant axial load	7
(2)	Numerical models	9
a)	Modeling of the experiment	9
b)	Constitutive models of reinforcement	11
c)	Constitutive model of core concrete	11
d)	Constitutive model of cover concrete	14
(3)	Numerical examples	15
a)	Effect of the constitutive model of longitudinal reinforcement on the response of piles	15
b)	Effect of concrete softening parameters on the response of piles and the limit of the ability of typical models	17
(4)	Concluding Remarks	23
3	Modeling of Cast-in-place RC Piles Using Fiber Elements Considering Reinforcement Swelling	24
(1)	Proposal of a Constitutive Model of Longitudinal Reinforcement and a Length of Fiber Element	24

a)	Constitutive model without reinforcement swelling	24
b)	Criteria for initial swelling	26
c)	Constitutive model for swelled reinforcement bars and length of fiber elements with swelled reinforcement	27
d)	Some comments for constitutive models of concrete	31
(2)	Plastic Buckling Analysis of Reinforcing Bar	33
a)	Numerical model for plastic buckling analysis	33
b)	Determination of plastic buckling length of reinforcement bar	34
c)	Differences in reinforcement bar behavior due to variations in hoop resistance spring rigidity	37
(3)	Numerical Analysis of Cast-In-Place RC Pile Specimens Subjected to Re- versed Cyclic Load	38
a)	Experiment on the type-1 pile specimen subject to horizontal re- versed cyclic load with a constant axial load	39
b)	Experiment on type-2 pile specimen subject to horizontal reversed cyclic load and varying axial load	48
(4)	Concluding Remarks	56
4	Improvement in the Ductility Characteristic of the RC Pile in Soil Due to the Lateral Confinement Effect by Earth Pressures	58
(1)	Examined experiment	59
(2)	Enhancement of the toughness of cover concrete by a lateral confined effect due to earth pressures	63
(3)	Numerical calculations	68
a)	Computational cases and modeling	68
b)	Results and discussions	71
(4)	Concluding remarks	78
5	The Relationship in a Pile Foundation between Damage to Pile Members due to	

Kinematic Interaction and Stability of a Foundation as a Structural System	80
(1) Introduction	80
(2) Numerical settings	81
(3) Results of dynamic analyses	90
a) Dynamic behavior of ground	90
b) Response and damage to foundation in Case 1	93
c) Response and damage to foundation for Cases 2 and 3	99
(4) Discussion of seismic design of pile foundations including the effect of ground motion and Concluding Remarks	107
6 Conclusions	110
REFERENCES	112
A Finite Element Analysis of Plastic Buckling Behavior of Longitudinal Reinforcing Bars Constrained by Core Concrete and Hoop Bars	118
B Effect of Stiffness of Hoop Bars in the Post Buckling Behavior of a Longitudinal Reinforcing bar	121
(1) Numerical model	121
(2) Numerical results	122
a) Difference in the behavior of a reinforcing bar due to hardening ratio	122
b) Difference in behavior depending on the spacing and rigidity of hoops	123
(3) Recommendations for future research	126

List of Tables

2.1	Results of material element tests for the type-1 pile specimen	8
2.2	Computational cases with respect to the effect of constitutive models of longitudinal reinforcement	16
2.3	Computational cases with respect to the effect of the compressive softening behavior of concrete. (The same values are used in both cover and core concrete.)	18
2.4	Computational cases for various concrete behaviors (The confined effect is neglected on cover concrete.)	19
3.1	Results of material element tests for spp-1 pile specimen corresponding to the pure bending test	48
3.2	Results of material element tests of type-2 pile specimen	50
3.3	Concrete material parameters in numerical analysis of type-2 pile specimen	52
4.1	Strength data with respect to consolidation stress for the Kashima sand of $D_r = 30\%$	60
4.2	Results of material element tests for the RC pile specimen embedded in sand	61
4.3	Numerical cases	72
5.1	Ground profiles	83
5.2	Summary of computational cases	84
5.3	Arrangement of reinforcement in pile	88

List of Figures

2.1	Experiment on type-1 pile specimen subject to horizontal reversed cyclic loading	8
2.2	Horizontal load-displacement hysteresis at the load point in the experiment of type-1 pile specimen	10
2.3	Modified Muguruma model	12
2.4	Schematic outline of the constitutive model of concrete employed in this paper	13
2.5	Horizontal load-displacement curve at the loading point and stress-strain curve of a typical longitudinal reinforcing bar in the base element (Case 1, bi-linear model of longitudinal reinforcement, $E_1/E_0 = 0.1$)	16
2.6	Horizontal load-displacement loop at the loading point and stress-strain loop of a typical longitudinal reinforcing bar in the base element (Case 2, Ramberg-Osgood model of longitudinal reinforcement)	17
2.7	Horizontal load-displacement relationship at the loading point (Cases 3, 4, 5, and 6)	19
2.8	Horizontal load-displacement relationship at the loading point (Cases 7, 8, and 9)	20
2.9	A typical damage process in RC piles	21
3.1	Menegotto-Pinto model	26
3.2	Constitutive model including the swelling effect of reinforcement bars . . .	28
3.3	Numerical analysis of plastic buckling of reinforcement bar (for case $L = 4s$)	29
3.4	Behavior of a swelled reinforcement bar	30

3.5	Averaged strain derived by uniaxial compression test	32
3.6	Deformed shape after plastic buckling of longitudinal bar in the type-1 pile specimen	35
3.7	Normalized load-displacement relationship (type-1 pile specimen)	35
3.8	Normalized load-displacement relationship and plastic buckling deformation for case $L = 3s$ (type-2 pile specimen)	36
3.9	Difference in normalized load-displacement relationship caused by different spring constants for hoop resistance (type-1 and type-2 pile specimens, $L = 3s$)	38
3.10	Horizontal load-displacement hysteresis at the loading point (type-1 pile specimen)	40
3.11	Stress-strain hysteresis of typical outermost longitudinal reinforcement bar in the element at the bottom	41
3.12	Horizontal load-displacement hysteresis at the loading point for the case without considering the swelling of reinforcing bars (type-1 pile specimen)	42
3.13	Damage state immediately after the final $13\delta_y$ cycle	44
3.14	Difference in numerical results of horizontal load-displacement hysteresis at loading point depending on hoop spring modeling (type-1 pile specimen) .	45
3.15	Set up for a pure bending experiment of type-1 pile specimen	46
3.16	Load-displacement curve at a loading section, experimental and numerical results	46
3.17	Experiment of type-2 pile specimen	49
3.18	Pattern of varying axial load (dashed lines); tension is positive. Solid lines indicate the numerical results.	50
3.19	Horizontal load-horizontal displacement hysteresis at the loading point (type-2 pile specimen)	53
3.20	Estimated damage after final cycle	54

3.21	Histories of axial load, axial displacement, and horizontal displacement at the loading point: tension is positive for both axial load and displacement (type-2 pile specimen).	55
3.22	Hystereses of axial load, axial displacement, and horizontal displacement at the loading point: tension is positive for both axial load and displacement (type-2 pile specimen).	56
4.1	Test setup (unit: mm)	59
4.2	Load-displacement hysteresis at the load point	62
4.3	Schematic diagram of stress-strain model for concrete in compression proposed by Mander et al.	65
4.4	Comparison of stress-strain relationship for cover concrete among the numerical cases	66
4.5	Basic Hysteretic Mechanism.	69
4.6	Load-displacement response at the loading point (Case 0: No consideration of reinforcement swelling and lateral confinement of the earth pressure to the cover concrete)	73
4.7	Distributions of longitudinal strain in extreme reinforcement bars at the positive peak displacements of the first cycles (Case 0)	74
4.8	Load-displacement response at the loading point (Case 1: Including consideration of reinforcement swelling and without consideration of lateral confinement to the cover concrete by earth pressures)	74
4.9	Distributions of longitudinal strain in extreme reinforcement bars at the positive peak displacements of the first cycles (Case 1)	75
4.10	Load-displacement response at the loading point (Case 2: Including consideration of reinforcement swelling and lateral confinement to cover concrete by earth pressures)	76
4.11	Distributions of longitudinal strain in extreme reinforcement bars at the positive peak displacements of the first cycles (Case 2)	76

4.12	Load-displacement response at the loading point (Case 3: with considerations of both reinforcement swelling and lateral confinement to cover concrete by earth pressures)	77
4.13	Distributions of longitudinal strain in extreme reinforcement bars at the positive peak displacements of the first cycles (Case 3)	78
5.1	Simulated pier and finite element mesh	82
5.2	Numerical analysis of plastic buckling of longitudinal reinforcement bar (Arrangements 'c' and 'd')	88
5.3	Distribution of peak ground displacement	91
5.4	Distribution of peak shear strain in the ground	91
5.5	Distribution of peak ground acceleration	93
5.6	Time histories of: horizontal displacement at the ground surface on line D u_{gDs} ; displacement and lateral force coefficient of the upper structure u_u and k_{hu} ; lateral force coefficient of the pile cap k_{hf} ; bending moment of the pier at the bottom M_p ; and reaction force at the base of pile No. 4 P_{V4} (Case 1) .	95
5.7	Yield zone of longitudinal reinforcing bars in piles (Case 1)	96
5.8	Distribution of bending moment in each pile (Case 1); solid lines are the absolute maximum values, and dashed lines are the values when the compressive reaction at the base of the No. 4 pile reaches the maximum ($t = 4.9$ s)	98
5.9	Axial force distributions on piles when the reaction force at the base of the No. 4 pile reaches a maximum, $t = 4.9$ s (Case 1), where tension is positive	99
5.10	Time histories of: horizontal displacement of the ground at GL -16 m on line D u_{ggD} ; curvature ϕ ; bending moment M ; and axial force N of pile No. 4 at GL -16.25 m (Case 1)	100
5.11	Yield zone of piles (Case 2)	101
5.12	Stress-strain hysteresis curves of typical fiber element under swelling of the reinforcing bar	101

5.13	Bending moment (M -curvature ϕ hysteresis) of an element containing swelled reinforcements	102
5.14	Time history of axial strain of swelled reinforcement (3 to 8 s.)	102
5.15	Comparison of lateral force coefficients obtained in different cases (Cases 1, 2, and 3)	104
5.16	Time histories of: horizontal displacement of the ground surface on line D, u_{gsD} ; horizontal displacement of the superstructure, u_u ; lateral force coefficient of the superstructure and pile cap, k_{hu} and k_{hf} (Case 2)	104
5.17	Hysteresis loops of responding lateral force coefficient versus responding horizontal displacement (Case 1 and 2)	105
5.18	Comparison of rotation angles obtained in Cases 1 and 2	106
A.1	Comparison between the finite element analysis result and the prediction by the proposed constitutive model for the behavior of the swelled reinforcing bar (type-1 pile specimen)	119
A.2	Comparison between the finite element analysis result and the prediction by the proposed constitutive model for the behavior of swelled reinforcing bar (type-2 pile specimen)	120
B.1	Difference in deformed shape related to changes in the hardening ratio of the longitudinal reinforcing bar	123
B.2	Difference in normalized load-displacement relationship related to changes in the hardening ratio of the longitudinal reinforcing bar	124
B.3	Differences in normalized load-displacement relationships and plastic buckling shapes at an averaged strain of 5% depending on hoop resistance . . .	125
B.4	Differences in normalized load-displacement relationships and plastic buckling shapes at an averaged strain of 5% depending on different hoop spacing	125

List of Photos

2.1	Damage at bottom of the type-1 pile specimen after the experiment	10
3.1	Cracks developed in type-1 pile specimen after the pure bending experiment	47
3.2	Damage at bottom of type-2 pile specimen after the experiment (compression side when the pile was subject to compressive axial force)	51
4.1	State of the test ground after the experiment and dewatering	62
4.2	The final crack pattern of the tested pile in an experiment involving RC piles embedded in sand and subjected to a lateral cyclic load	63
4.3	Final damage to reinforcing bars in the test pile specimen	64

1 . Introduction

Seismic design methods and structural design codes have been developed based on the lessons learned from past earthquake experience. A recent trend is to clarify and verify the seismic performance requirements of structures for withstanding large earthquakes in order to prevent the collapse of structures or to control structural damage so that the structure quickly returns to service. Accordingly, seismic design has come to examine both the strength and ductility of structures against large earthquake motions, rather than only ensuring that structures behave elastically against small to medium earthquake motions, as was the case previously.

The Specifications for Highway Bridges,¹⁾⁻³⁾ which is the Japanese design code for highway bridges, require that the response of a foundation to very strong seismic motions of low probability during the service lifetime should be such that no excessive repair work is necessary, thus enabling a quick return to service, and the horizontal strength and ductility should be sufficient to facilitate repair of damage to the foundation if needed. Furthermore, the Specifications for Highway Bridges recommend a ductility design method as acceptable for verifying the requirements. This is a static design method, and lateral forces corresponding to the inertial forces of the first vibration mode of seismic response are applied, that is, a so-called 'pushover analysis' is employed. In the ductility design method of pier foundations, the lateral inertial forces arising from the motions of superstructure, pier, and pile cap are taken into account. It is verified that the maximum displacement at the foundation top does not reach the range in which the behavior of the foundation system is clearly nonlinear against the design earthquake. In addition, dynamic inelastic response is estimated if needed,

through applying a so-called energy conservation method to the result of the pushover analysis, under the assumption that the behavior of the structure can be modeled as a one-DOF system with an idealized bi-linear response. The specifications show a couple of examples of inevitable situations in order to consider the inelastic behavior of a foundation system. One is when a pier ends up possessing a large capacity due to factors outside the seismic design process. The other is when liquefaction of the subsoil layers occurs. These exceptions occur because excessively expensive and large-dimension design results would be achieved in the case of considering only the strength capacity.

However, since several disastrous earthquakes have occurred in various parts of the world in the last decades, engineers are seeking design methods that can provide a more realistic description of the seismic behavior of structures and assess the structural limit states of foundations during very strong earthquakes. One of the salient problems to be solved in order to satisfy the need of engineers is the prediction of the progressive structural inelastic behavior of foundations from the elastic state up to the postpeak region. In this study, our emphasis is placed on concrete piles, especially on cast-in-place RC piles, because these are the most popular type of structural piles in Japan. Major factors in the reduction of bending rigidity can be characterized by the onsets of concrete cracks and reinforcement yield. Moreover, the negative stiffness of bending rigidity can arise from the onsets of cover concrete spalling and reinforcement swelling. Since pile foundations are highly statically indeterminate as structural systems and consist of several piles and the surrounding soils, there is no large capacity loss until the foundation is severely damaged in several sections. Therefore, the postpeak behavior of the section must be considered in a numerical model of piles in order to predict the collapse behavior of the entire system of the grouped pile foundation.

However, numerical models used in the design of RC members do not consider such postpeak behavior and do not allow reliable estimates of the relationship between the pile foundation stability and the damage sustained by each pile. Accordingly, in order to quantitatively examine the stability of pile foundations subjected to inertial forces from the super-

structure and the damage sustained, and to develop the ductility design of pile foundations, Kimura et al.^{4),5)} performed a series of large-scale experiments of pile group structures in air and subjected to reversed cyclic horizontal and overturning moment loads. This is a milestone in the history of the development of the limit state design of pile foundations. Kimura et al. have proposed an allowable value for the displacement ductility factor of pile groups to ensure the seismic performance requirements for design against large earthquakes as specified in the Specifications for Highway Bridges. Although the recommended limit value of ductility factor is, deterministically, four and is considered to be independent of the number of rows of piles and the type of piles, the limit ductility factor must depend on these elements. The reason for the recommended allowable ductility factor being made constant is that the proposal is based only on a small number of experimental studies, and conventional numerical models and techniques cannot predict the postpeak behavior of piles and cannot estimate the relationship between the initiation of a large loss of horizontal capacity of a pile foundation and the progress of damage to the piles (in other words, the ultimate limit state of the horizontal deformation capacity cannot be determined).

Several studies have reported numerical simulations that trace the postpeak behavior of RC members,⁶⁾ and these studies have indicated that reinforcement swelling must be properly modeled.^{7)–11)} The fiber model is considered to be suitable because we can set material nonlinearities and kinematic reinforcement softening phenomena due to swelling as constitutive relations in each fiber and because changes in the applied axial load in each pile of a pile foundation that depends on the horizontal displacement of the superstructure strongly affect the nonlinear characteristics of the piles. On the other hand, some consideration is necessary in order to avoid mesh size dependency of the numerical result when a softening-type constitutive model is used in a finite element analysis.^{12),13)} For general RC members, however, the problems of how to model the swelling behavior of the reinforcement and how to set the element length when using a softening constitutive model considering the presence of swelling have not yet been resolved.

This study is organized as follows:

- In Chapter 2, we conduct numerical analyses of an RC pile subjected to lateral cyclic loading in air, and demonstrate how the stress-strain relations of concrete and reinforcement affect the numerical behavior of RC piles. We summarize some important items to be considered in modeling of RC piles with fiber elements to predict their postpeak behavior. Finally, we clarify the application limitations of existing constitutive models.
- In Chapter 3, a uniaxial constitutive model for longitudinal reinforcement, including kinematic softening due to swelling behavior, is proposed to trace the behavior of RC beams from the elastic region to the postpeak region. The proposed model is to be implemented into fiber elements. The proposed model treats the swelling of a reinforcement bar as a plastic buckling phenomenon that occurs after compressive collapse of the cover concrete. This model is combined with an ordinary constitutive model with no swelling effect. In addition, a method to determine the length of an element to which the proposed constitutive model is applied is also proposed in order to avoid the mesh dependency of the finite element result due to the softening of elements. The mesh dependency can be avoided by introducing the plastic buckling length as the characteristic element length. The accuracy and characteristics of this constitutive model are checked through calculations using the experimental results of RC piles subjected to lateral reversed cyclic loading.
- An application of the proposed model is demonstrated. In Chapter 4, the ductile behavior observed in an experiment involving an RC pile embedded in sand and subjected to a lateral reversed cyclic loading at the top is numerically examined. Recently, the use of extended RC pile shafts in single and multicolumn bents is expected as a result of increased construction of overpasses in urban areas, because there is less space for the construction of foundations and such structures do not require pile caps. An extended pile shaft is a structure in which the pile shaft is continued over the ground level as a column of almost the same diameter. In these structures, the plastic hinge zone which

would dominate the ultimate limit state of the bridge as a system is formulated as being located under the ground. Accordingly, it is anticipated that the seismic response of the bridge as a system and the distributions of sectional forces of column bents as well as pile shafts depend on the location and ductility capacity of the plastic hinge zone. Therefore, in this chapter, we examine the difference in toughness of cast-in-place RC piles located above and below the ground, respectively. In particular, we look at the lateral confinement effect to the cover concrete of the pile due to earth pressure from the surrounding soils. We numerically estimate such effects on the ductility of piles through parametric simulations using the proposed model.

- Finally, in Chapter 5, an example of the seismic behavior simulation of a grouped pile foundation consisting of cast-in-place RC piles is shown, using the proposed modeling of RC piles. According to the concept of performance-based design code, the seismic design will capture the seismic collapse behavior of pile foundation systems and define the limit states to be considered, that is, the relationship between the possible damage state and the stability of the foundation. In effect, Chapter 5 shows an example of a future computational seismic performance assessment of a pile foundation subjected to a severe earthquake, based on the performance-design concept. Recent reports of earthquake damage such as that in the Hyogoken-Nanbu earthquake (Kobe Earthquake) of 1995 reveal that pile foundations are likely to sustain damage near the boundaries between soft and hard subsoil layers due to the kinematic interactions during a large earthquake. Many studies have attempted to develop prediction methods for such damage, which are suitable for practical design.^{14)–20)} Nevertheless, a quantitative assessment of the effects of local damage or local loss of the strength of the piles on the foundation system has not been fully investigated.²¹⁾ We also discuss a seismic design method for pile foundations, which takes into account the kinematic effect from seismic-induced ground vibrations, for the special case in which the damaged portions in the piles are deeper compared to the area greatly affected by the inertial interaction.

2 . Fundamental Limit of the Ability of Existing Constitutive Models in Prediction of Postpeak Behavior of Cast-in-place RC Piles

A fiber model (or fiber element) is a kind of beam element used in the finite element method. A fiber model consists of several subdivided fibers in the sectional area of a beam. Each fiber is given a uniaxial constitutive model for uniaxial tension and compression in the fiber, the sectional bending rigidity and sectional forces are derived by the integration of corresponding physical values of the fibers, respectively. We can consider the nonlinear properties of RC members due to the material nonlinearities of concrete and reinforcements.

Meanwhile, moment-curvature relationships are widely-accepted in modeling to simulate the nonlinear bending behavior of RC beams, because the handling is much easier in the computation rather than when using the fiber model. However, pile foundations, which are emphasized in the present study, are composed of several piles and are frame-type structures. When a pile foundation is subject to horizontal loads or overturning moment loads, the axial force in each pile varies and the nonlinear property for bending in each pile also changes along with the variation of the axial force. Namely, the moment-curvature relationship of each pile is not maintained during the action of horizontal loads or overturning moment loads to a foundation system. Consequently, the applicability of moment-curvature relationships in nonlinear analyses of pile foundations should be considered to be limited. This has been also shown by the authors^{21),22)} through a numerical simulation of a model group RC pile subjected to cyclic lateral and moment loads, simultaneously.

On the other hand, the fiber model automatically takes the effect of the variation of axial force into account, since the bending rigidity of each pile is updated in each loading step along with the change in its axial force at every section during a simulation. Therefore, the fiber model is greatly advantageous in the nonlinear analysis of pile foundations.

In this chapter, as an initial step in the study of the modeling of cast-in-place RC pile members, the effects of the constitutive relations of concrete and reinforcement on the nonlinear response of piles are examined, based on typical constitutive relations.

(1) Experiment on a type-1 pile specimen subject to a horizontal reversed cyclic load with a constant axial load

We examine an experiment performed by Kimura et al.^{4),5)} The specimens used are summarized in **Figure 2.1**, and the corresponding material properties, which were obtained from standard element tests, are listed in **Table 2.1**. We hereafter refer to this specimen as the type-1 pile specimen. The axial load was first applied to the top of the specimen up to 147 kN and was maintained there throughout the experiment, where the axial compressive stress was 7.7 % of σ_{ck} . The horizontal reversed cyclic load was then applied. This cyclic loading was controlled by displacement at the horizontal loading point of the specimen. The amplitude of the specified displacement was gradually increased from δ_y to $n \times \delta_y$ ($n = 2, 3, 4, \dots, 16$), where the yield displacement δ_y is defined as the displacement at which an extreme longitudinal reinforcement bar first yields. The amplitude of the specified displacement was eventually determined to be 3.57 mm in this specimen. At each displacement, three cycles were repeated.

Figure 2.2 shows the horizontal load-displacement history measured at the loading point in the experiment. Noteworthy characteristics of this history are summarized as follows:

- Horizontal cracks appeared when the horizontal load reached 24.5 kN;

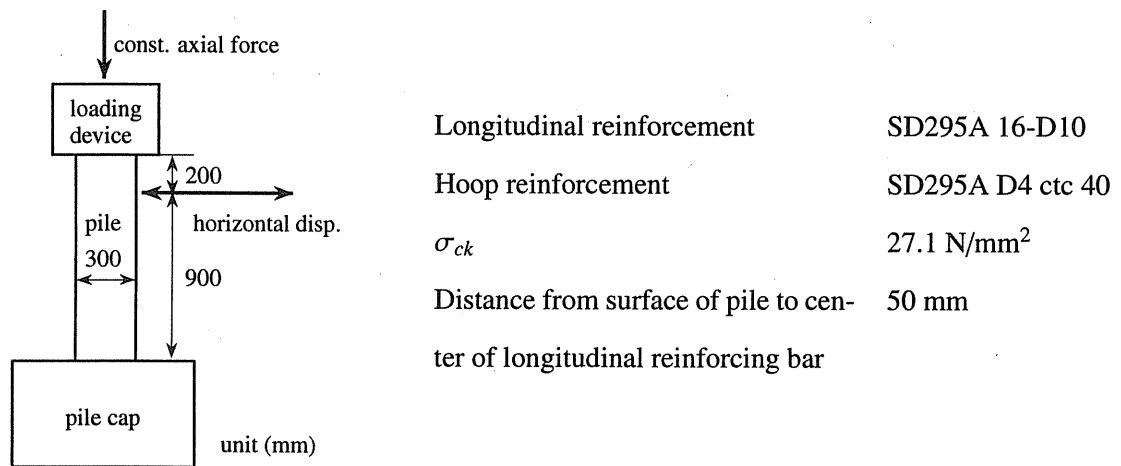


Figure 2.1 Experiment on type-1 pile specimen subject to horizontal reversed cyclic loading

Table 2.1 Results of material element tests for the type-1 pile specimen

Reinforcement

Reinforcement	Yield stress (N/mm ²)	Tensile strength (N/mm ²)	Young's modulus (kN/mm ²)
Longitudinal	348	478	175
Hoop	389	554	213

Concrete

Age (days)	Compressive strength (N/mm ²)	Young's modulus (kN/mm ²)	Splitting strength (N/mm ²)
40	27.1	26.5	2.72

- The maximum horizontal load, 55 kN, was attained when the horizontal displacement became $4\delta_y$ (14.3 mm);
- During the $5\delta_y$ cycle, the surface concrete of the pile slightly collapsed near the joint between the pile and the footing;
- Although the horizontal load fell temporarily during the $7\delta_y$ cycle (25.0 mm), it was maintained afterwards in cycles $7\delta_y$ through $10\delta_y$;
- However, loss of horizontal load started again at cycle $11\delta_y$ (49.3 mm), and damage to the pile became noticeable;
- Eventually, at the third cycle of $13\delta_y$, several large pieces of the cover concrete in the compression side near the joint suddenly dropped away.
- Thereafter, a reinforcing bar fractured at $14\delta_y$;
- Further reinforcement bars began to fracture after the first cycle of $16\delta_y$;
- The experiment was terminated after the $16\delta_y$ cycle (57.1 mm).

Damage observed near the bottom of the specimen is shown in **Photo 2.1**. The longitudinal reinforcing bars were deformed into the form of a sinusoidal wave with a wavelength approximately three times as long as the hoop spacing of 40 mm, i.e. 120 mm.

(2) Numerical models

Piles are considered to be Bernoulli-Euler and are presented as fiber elements in the present study.

a) Modeling of the experiment

A two-dimensional finite element analysis is employed within the framework of the infinitesimal displacement theory. The connection between the pile and the footing is considered as a rigid embedded end. The same vertical load as in the experiment is first applied

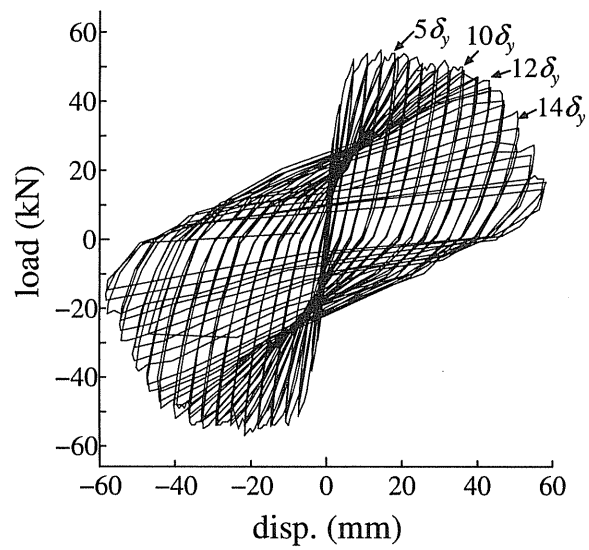


Figure 2.2 Horizontal load-displacement hysteresis at the load point in the experiment of type-1 pile specimen

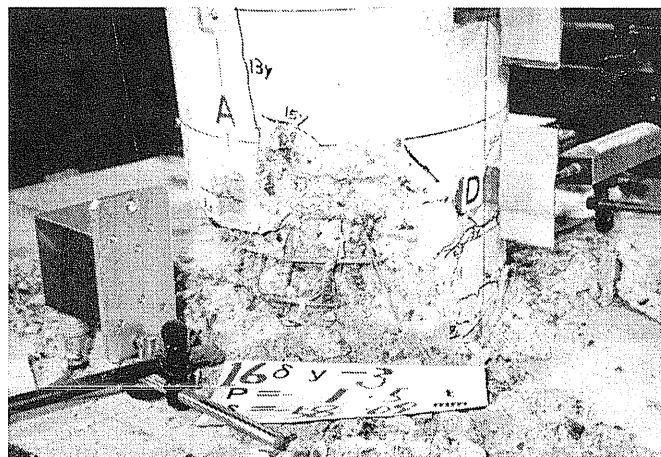


Photo 2.1 Damage at bottom of the type-1 pile specimen after the experiment

to the top of the specimen, and then the horizontal displacement is input at a height of 900 mm, which corresponds to the loading point in the experiment.

Since we employ a softening type of constitutive relation for concrete, as described below, the finite element mesh dependency of numerical results is anticipated. In order to avoid this, we fix the element length as 150 mm, which is a half of the pile diameter, based on the experimental observation and the specifications of the seismic design of the piers.¹⁾ Next, two types of nonlinear constitutive relation for reinforcing bars and several patterns of the values of the degradation gradient parameters of concrete are tested in this section under the condition of constant element length.

b) Constitutive models of reinforcement

Two representative constitutive models are examined in this chapter: the bi-linear model and the Ramberg-Osgood model. The Ramberg-Osgood model has an exponential function type relation and is employed in a constitutive relation of soils that are subject to simple shear deformation, although the model was originally proposed for metals.²³⁾ Details of the Ramberg-Osgood model are presented in Chapter 5 (2).

c) Constitutive model of core concrete

Since the postpeak behavior of piles subject to cyclic loading is considered, we employ the modified Muguruma model, as shown in **Figure 2.3**, as proposed by Ristic et al.,²⁴⁾ which can also handle softening behavior due to plastic deformation.

This model is composed of nine hysteresis rules. The skeleton (envelope) curve is constructed using an ascending compression branch of a second-order parabola (O-CC), the sustaining branch of the maximum compressive strength (CC-R), and two linear falling branches (R-U, U-L), where the point 'L' is the compressive failure point. Internal curves consist of: when stress reverses from the compression loading envelope to an unloading path, a second-order parabola crossing zero stress at one-half strain of the reversed point strain. When the stress changes back to a compression reloading path, the branch goes straight through the

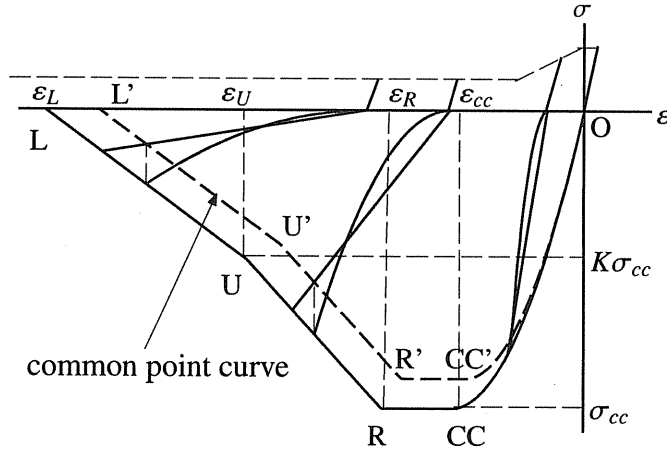


Figure 2.3 Modified Muguruma model

common point of the same strain as the preceding reversal point on the compression loading envelope. The common point curve, as shown in **Figure 2.3**, is defined as the curve scaled to 0.9 times a skeleton curve, and this is required so as to specify reloading curves. Stress-strain branches of concrete subjected to tension stress are linear, and the values of tension failure stress σ_t and gradient E_t are dependent on the maximum compressive strain in the history, ε_{cmax} . In addition, in the strain region exceeding tension or compression failure strains, the stress and strain paths run along $\sigma = 0$.

We further modify this model as follows in order to include the confined effect on core concrete by hoop reinforcement, and construct a skeleton curve by referring to Hoshikuma et al.,²⁵⁾ which is adopted in the Japanese Specifications for Highway Bridges.¹⁾ This modification alters the skeleton curve of the modified Muguruma model such that it no longer possesses a sustaining branch of peak strength, as illustrated in **Figure 2.4**. The maximum compressive stress point (CC) is evaluated by

$$\sigma_{cc} = \sigma_{c0} + 3.8\rho_s\sigma_y, \quad (2.1)$$

$$\varepsilon_{cc} = 0.002 + 0.033 \frac{\rho_s\sigma_y}{\sigma_{c0}}, \quad (2.2)$$

for the solid circular section, where σ_{c0} is the unconfined compression strength of concrete obtained by a standard material element test, σ_y is the yield strength of the lateral confinement bars (i.e. hoops or spirals in the case of piles), ρ_s is the volume ratio of the lateral

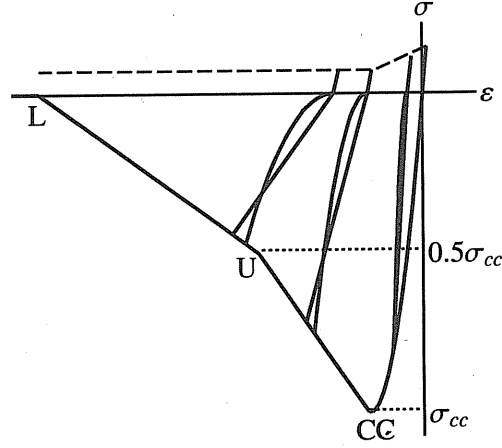


Figure 2.4 Schematic outline of the constitutive model of concrete employed in this paper

confinement bars and is defined by

$$\rho_s = \frac{4A_h}{sd}, \quad (2.3)$$

where d is the diameter of the hoop or spiral, which has a bar area A_h , and s is the longitudinal spacing of the hoop or spiral. Hoshikuma et al. have recommended that the value of ρ_s should be given as 0.018 when Eq. (2.3) provides a value larger than 0.018 from the scope of their experimental parameters. The degrading gradient between CC-U is given by the following equation:

$$E_{\text{des}} = -11.2 \frac{\sigma_0^2}{\rho_s \sigma_y}, \quad (2.4)$$

where E_{des} is a descending gradient between CC-U. While Hoshikuma et al. used a hyperbolic function rather than a simple second parabolic function to define the ascending branch, a second-order parabola is basically adopted in the present study following the definition of a skeleton curve of the modified Muguruma model, because the numerical results for the bending behavior of piles are considered to show little discrepancy.

Since the descending gradient introduced by Hoshikuma et al. is determined through a regression analysis of the experimental results obtained for uniaxial compression tests using large-scale specimens and since the regression analysis evaluates the range from maximum compressive strength σ_{cc} to $0.5\sigma_{cc}$, this regression analysis cannot be applied directly to the modified Muguruma model. Accordingly, we derive the first descending gradient, which

is valid in the range from σ_{cc} to $0.5\sigma_{cc}$ (the branch CC-U) from Hoshikuma et al., and, in the following section, we set the second descending gradient in the range below $0.5\sigma_{cc}$ (the branch U-L) through a parametric study of the simulation of piles subjected to cyclic lateral load. The specimens used in the uniaxial compression element test of Hoshikuma et al. had volumetric ratios of hoop reinforcement of 0.19–4.66 %, and both the volumetric ratios of hoops used in test pile specimens analyzed numerically herein and those of the piles commonly used in Japan are of the same order.

d) Constitutive model of cover concrete

The cover concrete usually fails in brittle mode, and sudden compressive crushing at the surface and then spalling off of small or large pieces of cover concrete are observed, whereas damage to core concrete confined by hoops is progressive. In most RC beams or columns, since the ratio of the depth of cover concrete to the effective depth in a section is very small, cover concrete is usually neglected in the estimation of the strength capacity of RC members. In addition, few studies have examined one-dimensional constitutive models of cover concrete for use in fiber beam elements. However, the cover concrete of cast-in-place RC piles is relatively thick because these piles are constructed in the ground and have a larger uncertainty of execution. The modeling of cover concrete greatly influences numerical results for cast-in-place RC piles.

Thus, we do not expect any confined effect in the determination of the material parameters in the constitutive model of cover concrete, whereas the mathematical expression of the constitutive model of cover concrete is the same as that of confined concrete. We will give the stress and strain at the maximum compressive stress point CC as follows: the maximum stress is assumed to be equal to the value obtained in a standard element test, and the corresponding strain is assumed to be 0.002, in other words, these values are derived using Eqs. (2.1) and (2.2), respectively, with $\rho_s = 0$. The estimation of the values of the descending gradients of the skeleton curve is important but very difficult. Consequently, these values will also be examined through parametric simulations of an experiment involving the type-1

pile specimen in the following section.

(3) Numerical examples

a) Effect of the constitutive model of longitudinal reinforcement on the response of piles

First, we examine the effect of the constitutive model of longitudinal reinforcement on the numerical results of pile behavior. The computational cases are tabulated in **Table 2.2**. For the bi-linear model, the initial rigidity E_0 and the initial yield strain ε_y are given by the material element test results, and 1/10 the Young's modulus is used as the hardening rigidity E_1 . For the Ramberg-Osgood model, the values of E_0 and ε_y are also given by the material element test results, but the parameters for nonlinearity α and β in Eq. (5.1) are determined by trial and error via numerical analyses for the experiment of the type-1 pile specimen. The most suitable values from the trial-and-error analyses are obtained as $\alpha = 4$ and $\beta = 5$, whereas the obtained numerical results are not sensitive to the values of the nonlinearity parameters α and β very well. For concrete, in Cases 1 and 2, we neglect the confined effect on both core and cover concrete and employ the modified Muguruma model of **Figure 2.3** with respect to both the skeleton and inner curves, because the purpose in these cases is just to examine the differences due to the constitutive relations of the reinforcing bar. In other words, we assume $\sigma_{cc} = \sigma_{c0}$ and $\varepsilon_{cc} = 0.002$. Other parameters are temporarily assumed to be $\varepsilon_R = 0.003$, $\sigma_U = 0.5\sigma_{cc}$, $\varepsilon_U = 0.005$, and $\varepsilon_L = 0.008$, because we are now interested in only the differences in numerical results due to the differences in the constitutive relations of the bilinear model and the Ramberg-Osgood model.

We conduct numerical analyses only for the 2 and 3 δ_y cycles, because it is not preferable that the softening of concrete largely affects the numerical behavior of pile and, for other cycles, the softening of concrete could greatly affect the numerical behavior of the pile. **Figures 2.5** and **2.6** depict horizontal load-displacement curves at the loading point

Table 2.2 Computational cases with respect to the effect of constitutive models of longitudinal reinforcement

Case	Types of constitutive rel.	Parameters
1	Bi-linear model	$E_1/E_0 = 0.1$
2	Ramberg-Osgood model	$\alpha = 4, \beta = 5$

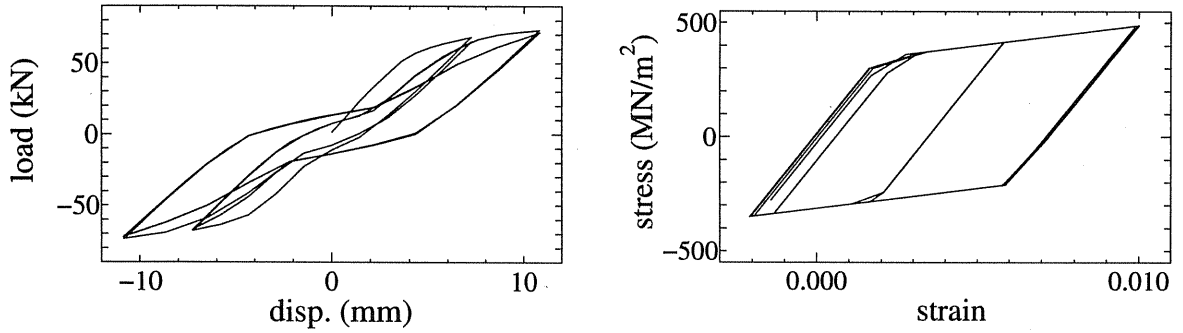


Figure 2.5 Horizontal load-displacement curve at the loading point and stress-strain curve of a typical longitudinal reinforcing bar in the base element (Case 1, bi-linear model of longitudinal reinforcement, $E_1/E_0 = 0.1$)

and stress-strain hystereses of a typical longitudinal reinforcing bar in the base element. The skeleton curves of the horizontal load-displacement loops of these cases are similar, but the internal curves are different. The shapes of the internal curves are highly associated with those of the stress-strain loops of the reinforcement, and the decrease in the gradient of each unloading path of the load-displacement loops is small in the bi-linear model result compared with the results for the other model. While the bi-linear model has a constant rigidity of internal curves, the Ramberg-Osgood model comprises smooth internal stress-strain curves, and correspondingly, smoother unloading paths and reduction in rigidities in unloading paths of the load-displacement loops are achieved.

As a result, a constitutive model with a smooth curve in the unloading rule is more appropriate for numerical analyses with respect to RC members subjected to cyclic loading when we employ the fiber model for RC members, because the degradation of unloading

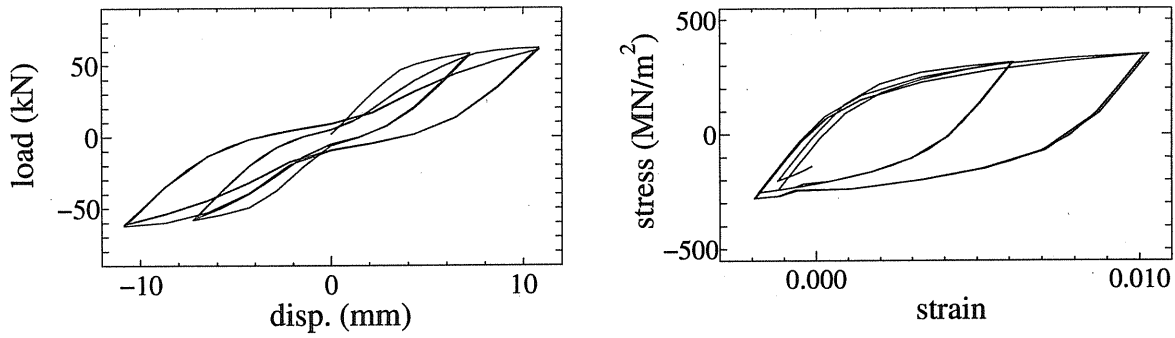


Figure 2.6 Horizontal load-displacement loop at the loading point and stress-strain loop of a typical longitudinal reinforcing bar in the base element (Case 2, Ramberg-Osgood model of longitudinal reinforcement)

gradient in the load-displacement curve is commonly observed in RC members.

b) Effect of concrete softening parameters on the response of piles and the limit of the ability of typical models

Next, we examine how the softening of concrete affects the numerical behavior of RC members under the condition that the values of parameters of the Ramberg-Osgood model for reinforcement are maintained. The loading is monotonic to one-direction in the numerical analyses here (i.e. pushover analysis). The parameters of the Ramberg-Osgood model are set as follows: initial rigidity E_0 and initial yield strain ε_y are determined by the values of material element test results, and the nonlinearity parameters α and β are again set as 4 and 5, respectively.

Initially, in this calculation, the concrete parameters are given as shown in **Table 2.3**, where the first and second descending gradients indicate the gradients of the CC-U branch and U-L branch in **Figure 2.4**, respectively. E_{des} indicates the first descending gradient yielded by Eq. (2.4). The second descending gradient is given as half of E_{des} , indicating a more gentle gradient, since confined concrete generally has good toughness and the load resistance capacity is not reduced significantly, even under large axial strain of greater than 3–5%, as demonstrated in the experiments conducted by e.g. Hoshikuma et al.²⁵⁾ and Mander

Table 2.3 Computational cases with respect to the effect of the compressive softening behavior of concrete. (The same values are used in both cover and core concrete.)

Case	First descending gradient (CC-U)	Second descending gradient (U-L)
3	E_{des}	$0.5E_{\text{des}}$
4	E_{des}	E_{des}
5	$0.5E_{\text{des}}$	$0.5E_{\text{des}}$
6	$2E_{\text{des}}$	$2E_{\text{des}}$

et al.²⁶⁾ In Cases 3 and 4, we consider two different second descending gradients. Moreover, in Cases 5 and 6, we deal with two different first descending gradients from the normal E_{des} in order to ascertain variations of pile behavior depending on the first descending gradient. For Cases 3 through to 6, the parameters for cover concrete and core concrete are set equal for the sake of simplicity.

Figure 2.7 shows the results of the pushover analyses. The load-displacement curves coincide with each other up to 10 mm. The maximum strength and the corresponding displacement increase as the gradients of softening parameters of concrete become milder. This is due to the difference in collapse strains of cover concrete. Moreover, the degree of reduction of load in postpeak is proportional to the values of the softening parameters of concrete. However, while the experiment exhibits noteworthy load-resistance reductions at 6 and $7\delta_y$ after the peak strength, and the resistance is stable between 7 and $10\delta_y$, the numerical results do not show such a tendency. Case 3 shows a tendency that is somewhat similar, due to the dual-stage softening behavior of concrete.

In order to improve on this, we distinguish unconfined cover concrete from confined core concrete in setting the constitutive parameters. Three cases listed in **Table 2.4** are tested. Only Case 7 includes a dual-stage descending curve in the concrete constitutive relation. In **Figure 2.4** of the core concrete, the values of σ_{cc} and ε_{cc} at the maximum strength point CC are obtained using Eqs. (2.1) and (2.2). The first descending gradient E_{des} in each case

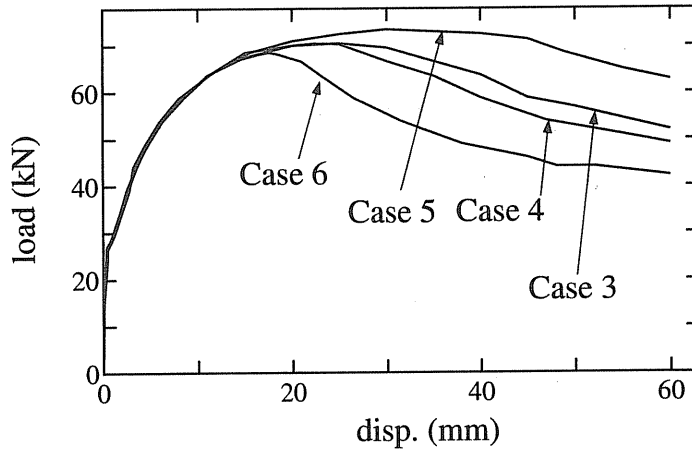


Figure 2.7 Horizontal load-displacement relationship at the loading point (Cases 3, 4, 5, and 6)

Table 2.4 Computational cases for various concrete behaviors (The confined effect is neglected on cover concrete.)

Case	Cover concrete			Core concrete		
	σ_{cc} (MN/m ²)	ε_{cc}	Descending gradient	σ_{cc} (MN/m ²)	ε_{cc}	Descending gradient
7	27.1	0.002	$1.5E_{des} \rightarrow 0.75E_{des}$	36.5	0.005	$E_{des} \rightarrow 0.5E_{des}$
8	27.1	0.002	$1.5E_{des}$	36.5	0.005	E_{des}
9	27.1	0.002	$2E_{des}$	36.5	0.005	E_{des}

is yielded by Eq. (2.4). The second descending gradient is gentler than the first descending gradient in Case 7. Cases 8 and 9 have single-stage descending gradients in the softening region. Cases 8 and 9 facilitate observation of the effects due to the different softening parameters for core concrete and cover concrete.

For the maximum strength point of cover concrete, the compression strength σ_{cc} is the same as that obtained by the material element test, and the corresponding strain ε_{cc} is given as the first term of Eq. (2.4) with no consideration of the confined effect. The values of the descending gradient are assumed to be of two cases, 1.5 times and 2.0 times as steep as E_{des} of the core concrete. However, no effective strategy is available for the determination of the

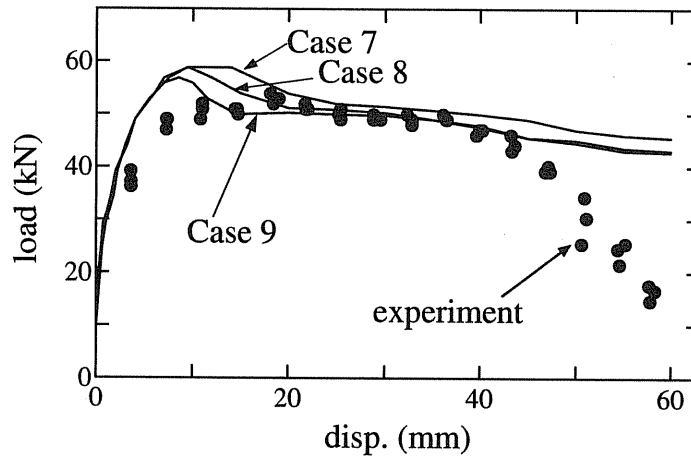


Figure 2.8 Horizontal load-displacement relationship at the loading point (Cases 7, 8, and 9)

descending gradient of unconfined concrete.

Presented in **Figure 2.8** are the results of pushover analyses with respect to horizontal load versus horizontal displacement relationships at the loading point, and the black circles in **Figure 2.8** show the experimental results for the horizontal loads and displacements at each $n\delta_y$. The distinction between cover and core concrete in the setting of the parameters of constitutive relation brings about a sudden decrease in the load after the peak strength and the subsequent load held compared with **Figure 2.7**. In addition, the prediction of load-displacement relationships is quite good up to $10\delta_y$ (35.7 mm). Ultimately, we can state that softening parameters of concrete affect predictions about the maximum strength point and degree of postpeak reduction of a pile and that it is better to set the values of the parameters of constitutive relation of confined concrete and unconfined concrete separately. As a result, herein, Case 7 is considered to be the standard setting for concrete parameters.

However, several problems appear. In particular, the numerical results continue to retain the horizontal resistance capacity and do not demonstrate its rapid loss after $10\delta_y$ (35.7 mm). The numerical simulations cannot predict the rapid loss of horizontal resistance capacity because the swelling of reinforcement, which is observed in the experiment, is not taken into account in the numerical models used herein. Swelling of reinforcement is

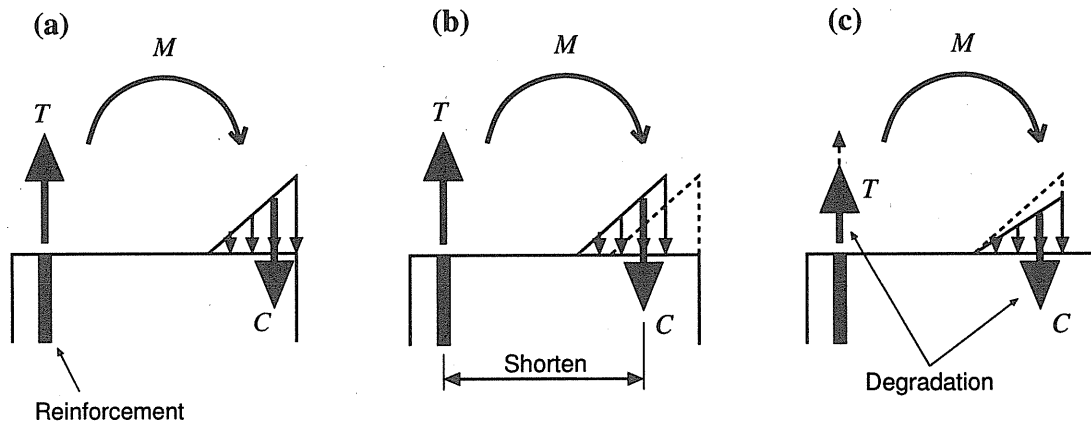


Figure 2.9 A typical damage process in RC piles

commonly observed in postpeak behavior of RC members subjected to cyclic lateral loads, and swelled reinforcement causes tensile resistance reduction of longitudinal reinforcement rather than the straight state and plays a crucial role in the loss of effective section modulus of RC members.

A typical collapse process of RC members subjected to cyclic bending deformation is illustrated in **Figure 2.9**. Before reaching the peak bending strength, the condition of sectional force can be modeled as (a) in **Figure 2.9**, where M is the sectional bending moment, T is the tensile force carried by reinforcement, and C is the compression force carried by concrete. The compressive damage to the concrete occurs from the edge of the section, and the distribution of the compressive force carried by the concrete is shifted inward, as shown in (b) of **Figure 2.9**, and then the moment arm is shortened. On the other hand, well-confined core concrete has usually a large ductility. Accordingly, the degradation of bending moment capacity is mild.

Reinforcement swelling occurs following the compressive collapse of the concrete around the reinforcement, and the tensile force carried by the swelled reinforcement decreases beyond that carried by the non-swelled reinforcement, as illustrated in (c) of **Figure 2.9**. The reduction in tensile strength of the reinforcement is a critical factor in the loss of bending capacity, and as a result, sudden and rapid bending capacity loss emerges.

Using fiber models, we generally consider this shortening phenomenon to be due to

compressive concrete collapse. However, an essential limit exists for conventional models that do not consider the effects of swelling of reinforcements in predicting the postpeak behavior of RC members. A pile foundation usually consists of a pile-group. Accordingly, the limit states of a pile foundation system cannot be assessed without predicting the postpeak behavior of each pile composing the foundation system. Therefore, the postpeak prediction caused by the swelling is the most important problem to be solved.

Needless to say, fractures of the reinforcements lead to strength and deformation capacity losses in the experiment. However, the failure of longitudinal reinforcements could be caused by fatigue or localized and accumulated damage due to a large number of cycles of alternating swelling and tensioning, rather than pure tension. The average strain of reinforcement bars predicted in the simulation is approximately five percent, which is considered to be smaller than the usual fracture strain observed in element tests. In addition, a high degree of spalling off of cover concrete was observed in the $13\delta_y$ cycle (46.4 mm cycle), while the first longitudinal reinforcement failure was observed in $14\delta_y$ cycle (50.0 mm cycle). This is too early if the first swelling is assumed to develop in $13\delta_y$ cycle. Therefore, it is inferred that the swelling of longitudinal reinforcement occurred before the $13\delta_y$ cycle in the experiment, and the first problem to be solved is the effects of swelling of reinforcements.

Another problem is that the numerical results behave more stiffly than as observed in the experiment before the peak strength points. This may be due to pull-out of longitudinal reinforcing bars of the pile in the anchored region in the pile cap or to inhomogeneous tensile strain distributions in longitudinal reinforcing bars of the pile on a micro-scopic scale caused by distributed cracks in the cover concrete. The latter may lead to a reduction in average initial yield strain in the longitudinal reinforcement bars.^{27),28)} However, we decided to concentrate on the postpeak behavior and ultimate limit state of cast-in-place piles in the present study.

(4) Concluding Remarks

Based on the parametric simulations herein, the fiber model is a powerful tool for modeling the nonlinear behavior of RC members, including cast-in-place RC piles. The typical constitutive models of reinforcement and concrete can trace the ductile behavior of cast-in-place RC piles, but cannot predict the ultimate limit state of piles, in other words, the beginning of rapid reduction of the lateral capacity of piles cannot be predicted and simulation of the postpeak behavior is not possible. Based on previous experiments on RC members subjected to lateral cyclic loads and past seismic damage case histories of RC members, the reinforcement swells to the outside of the section soon after drop-away of the cover concrete, and ultimately this causes a decrease in the tension stress sharing ability of the reinforcement. Therefore, we conclude that the swelling of reinforcing bars must be modeled in order to predict the postpeak behavior of cast-in-place RC piles.

3 . Modeling of Cast-in-place RC Piles Using Fiber Elements Considering Reinforcement Swelling

Although the process of reinforcement swelling and the decrease in the bending resistance of piles could be simulated by a three-dimensional modeling of the pile itself, this is cumbersome. We herein propose a uniaxial constitutive relation of a reinforcement bar considering both swelling and the setting of the fiber element length when applying the proposed model in fiber elements.

(1) Proposal of a Constitutive Model of Longitudinal Reinforcement and a Length of Fiber Element

a) Constitutive model without reinforcement swelling

We employ the Menegotto-Pinto model^{24),29)} as a constitutive model for the longitudinal reinforcement, as proposed for the reinforcing bars in RC members subject to cyclic loading. The Menegotto-Pinto model is specified as a bi-linear curve with initial rigidity E_0 and hardening rigidity E_1 , plus a series of asymptotic curves representing these bi-linear relations as shown in **Figure 3.1**. These are given by

$$\sigma^* = b\varepsilon^* + \frac{(1-b)\varepsilon^*}{(1+\varepsilon^{*R})^{1/R}} \quad (3.1)$$

with definitions

$$\sigma^* \equiv \frac{\sigma - \sigma_r}{\sigma_0 - \sigma_r}, \quad \varepsilon^* \equiv \frac{\varepsilon - \varepsilon_r}{\varepsilon_0 - \varepsilon_r}, \quad (3.2)$$

where σ is stress; ε is strain; $(\sigma_r, \varepsilon_r)$ is the last strain reversal point, and $(\sigma_0, \varepsilon_0)$ is the intersection of the two envelopes when unloading and loading occurs from point $(\sigma_r, \varepsilon_r)$, as illustrated in **Figure 3.1**. Parameter b is a hardening ratio defined by the initial rigidity E_0 and the hardening rigidity E_1 as follows:

$$b \equiv E_1/E_0. \quad (3.3)$$

Parameter R represents the deviation of a stress-strain branch from the bi-linear curve and is defined as

$$R = R_0 - \frac{a_1 \xi}{a_2 + \xi}, \quad (3.4)$$

where R_0 , a_1 , and a_2 can be regarded as material constants, and R_0 is the value of R at initial yield. We herein use the values of R_0 , a_1 , and a_2 used in the numerical analysis of an RC pier by Ristic et al.,²⁴⁾ because no experimental data by which to determine these material parameters is available, where $b = 0.02$, $R_0 = 20$, $a_1 = 18.5$, and $a_2 = 0.15$. The parameter ξ is defined as

$$\xi \equiv |\varepsilon_0 - \varepsilon_r'|/\varepsilon_y = |e_0 - e_r'|, \quad (3.5)$$

where ε_r' represents the preceding maximum or minimum strain reversal point with respect to $(\sigma_r, \varepsilon_r)$. This parameter represents the absolute value of plastic strain developed from $(\sigma_r, \varepsilon_r)$ to $(\sigma_r', \varepsilon_r')$, and is updated at every unloading and loading step, where σ_r' is the corresponding stress to ε_r' . In **Figure 3.1**, we use

$$s \equiv \sigma/\sigma_y, \quad e \equiv \varepsilon/\varepsilon_y, \quad (3.6)$$

where σ_y and ε_y are the initial yield stress and strain, respectively. In the application of Eqs. (3.4) and (3.5), if partial unloading and reloading occurs, we calculate the value of ξ using the strain reversal points during the unloading and reloading branch. Note that the value of ξ for the path following the first strain reversal is estimated as illustrated as ξ_0 in **Figure 3.1**.

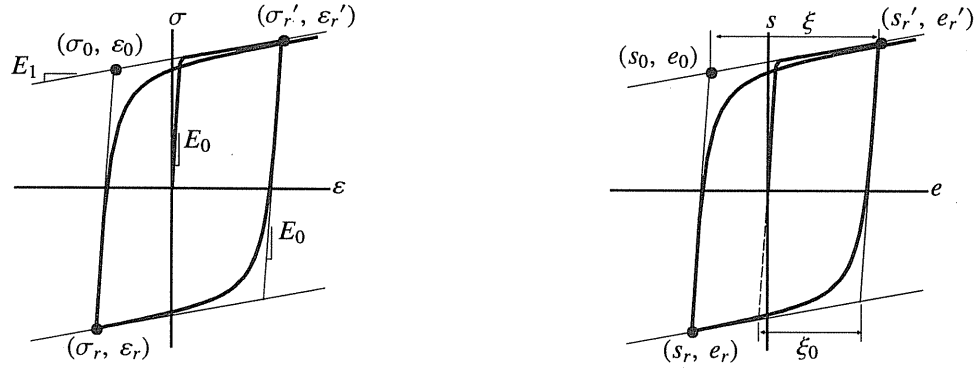


Figure 3.1 Menegotto-Pinto model

b) Criteria for initial swelling

There exist some interactions between reinforcement swelling and the collapse of the cover concrete. For example, Suda et al.^{7),8)} have generalized the mechanism of the initiation of reinforcement swelling and have proposed a criterion for the appearance of swelling considering such an interaction.

However, the results of existing loading tests on model cast-in-place RC piles^{4),5),30)} suggest that the cover concrete is either crushed or falls apart when the capacity of the piles begins to decrease significantly owing to bending. Therefore, we consider the use of a simple assumption, whereby the reinforcement swelling does not occur until the cover concrete at the position of the reinforcement bar collapses, to be appropriate. Specifically, swelling of a reinforcement bar becomes possible if the following conditions are satisfied consecutively:

1. the reinforcement bar strain must exceed the compressive failure strain of the cover concrete,
2. the reinforcement bar must then undergo unloading (and tensile loading), and
3. the reinforcement bar must once again undergo compressive loading.

That is, swelling of reinforcement is never initiated under monotonic loading.

c) Constitutive model for swelled reinforcement bars and length of fiber elements with swelled reinforcement

In piles subject to cyclic bending loads, when the axial fiber stress of longitudinal reinforcing bars reverses to compression after undergoing tension yielding, the reinforcing bars are no longer compressed along a straight line because the residual curvature of the reinforcing bars remains during the tension yielding process. Consequently, the accumulation of residual curvature can be considered to cause swelling of the longitudinal reinforcement bars. In addition, the swelled reinforcement bars behave as plastic buckled reinforcement bars. In addition, as has already been clarified via the numerical analyses in the previous chapter, the skeleton curve of the constitutive relation of reinforcement bars affects the envelop of load-displacement loops of piles and the definition of the internal curves thereof affects the form of horizontal load-displacement loop of piles. Thus, in order to trace the postpeak region of the horizontal load-displacement relationship of RC beams, the present study involves the following two factors:

- The reduction in compressive fiber stress of a longitudinal reinforcement bar caused by the swelling is modeled as plastic buckling behavior of the longitudinal reinforcement bar.
- The reinforcement behavior in tension after compressive swelled deformation is considered so that the tension resistance of the reinforcement and the effective bending rigidity of the pile will not be overestimated.

However, the softening type constitutive relations show a mesh dependency in the finite element results. Accordingly, in the present model,

- A characteristic length is introduced into the modeling of the constitutive relation in which the swelling effect is taken into account.

Figure 3.2 illustrates the proposed constitutive model taking swelling into account. This model consists of two parts, the Menegotto-Pinto model without swelling (curve ABC),

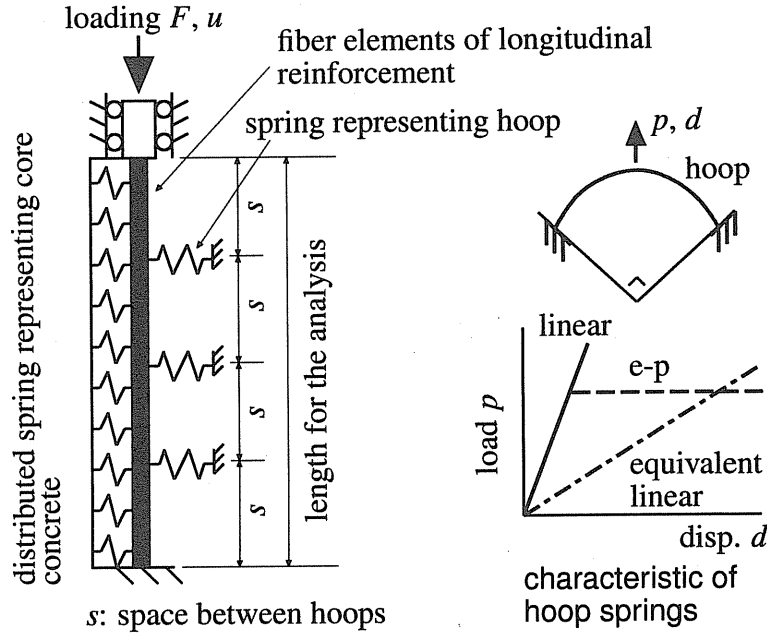


Figure 3.3 Numerical analysis of plastic buckling of reinforcement bar (for case $L = 4s$)

elastic foundation is employed, but the corresponding flexural rigidity is modified step-by-step depending on the degree of development of the plastic region under a certain loading.

In contrast, we herein propose a new method that is more suitable for practical implementation. First, we perform several cases of numerical plastic buckling analysis on reinforcement bars of lengths $L = ms$, where m is an integer to be controlled and s is the spacing between two adjacent hoops. Then we determine the critical m_0 . That is, the characteristic plastic buckling length $L_0 = m_0s$ at which the most drastic decrease in load is observed after initiation of plastic buckling among many trials with different m 's. It should be noted that there exist $(m - 1)$ sets of springs representing hoops for a bar of length $L = ms$. Dividing the obtained load F by the cross sectional area of the reinforcing bar and also dividing the displacement u by the characteristic plastic buckling length $L_0 = m_0s$, we obtain the averaged stress-strain relation. Finally, this averaged stress-strain relationship is used to express the compressive branch of the stress-strain relation curve PQBR in **Figure 3.2** by referring to the method employed by Gomes and Appleton.¹¹⁾ Point 'P' is the point at which the elastic unloading path of rigidity E_0 from the maximum tensile strain point 'A' in the history at that time crosses the zero stress level. Note that, in the analyses of pile behavior

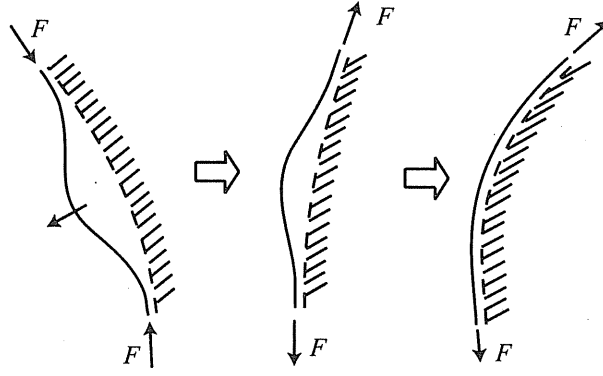


Figure 3.4 Behavior of a swelled reinforcement bar

presented hereinafter, we will assume piecewise linear load-displacement relationships for buckled longitudinal reinforcement and interpolate the discretized load-displacement data obtained from numerical analysis of plastic buckling of the reinforcing bar.

Swelling is assumed to occur when compressive stress σ_m derived from the Menegotto-Pinto model and σ_b obtained from the curve PQBR become coincident at a certain strain level. Thus, the stress-strain response is chosen as curve B-R not B-C as in the Menegotto-Pinto model.

When strain reverses after the post-buckling branch and expansion occurs, the stress and strain response begins to tend toward the maximum tensile strain point up until that time. This implies that the tensile resistance of longitudinal reinforcement bars that have experienced swelling under compression is smaller than in the state without any swelling, as indicated in **Figure 3.4**. We set the value of E_0 to E' , which is the instantaneous modulus between 'R' and 'A' (i.e. between the strain reversal point in the compression branch and the maximum tensile strain point up until that time), and the curvature parameter is set to $R = R_0$ in the Menegotto-Pinto model. A comparison of the averaged stress-strain hystereses given by the model proposed herein and a 2-dimensional finite element analysis for a reinforcing bar subjected to reversed cyclic axial loading is presented in Appendix A.

The length of the fiber element to which the proposed constitutive model is applied must be made equal to the characteristic plastic buckling length L_0 , as used in the setting of

the curve PQBR. This is because we want to reflect the physical phenomena onto the element length through averaging the deformation within the element.

From the results of past experimental research,^{4),5),30)} we can expect the length of the swelled portion of the reinforcement in standard arrangements to be equal to approximately one half the diameter of pile. Based on these results, axial extension and compression will not occur simultaneously in one particular fiber of an element as long as the element length is of the same order, so the proposed method is considered to be applicable. Since, in general, real piles are in practice approximately ten diameters in length or more, sufficient accuracy is possible from the viewpoint of convergence in the FEM analysis. Note that it is possible to apply the proposed model to any constitutive model as long as it is described within the framework of deformation theory.

d) Some comments for constitutive models of concrete

Since the constitutive relation of concrete considered in this study also includes softening, a similar treatment needs to be applied to the constitutive relation of concrete in order to avoid mesh size dependency in finite element results. Although there are many reports on this subject, there is no well-established method of solving this problem.¹²⁾

In addition, most constitutive relations of confined concrete are based on averaged longitudinal stress-strain relationships observed in uniaxial compression tests. Let us consider that, in uniaxial compression tests, localized deformation zones are formulated in the test specimens as illustrated in **Figure 3.5**. We assume the test specimens to be identical without the overall lengths and the lengths of the damaged regions to be same. In typical element tests, the axial strain is estimated by use of the longitudinal displacement at the end and the overall length of the specimen. In such cases, the averaged axial strains over the specimens become different, although the energy absorptions by plastic deformation are considered to be equivalent in both the specimens. In other words, there is no objective measurement of the averaged strain of softening concrete specimens. The present study deals with the postpeak behavior up to the loss of strength, and the determination of softening

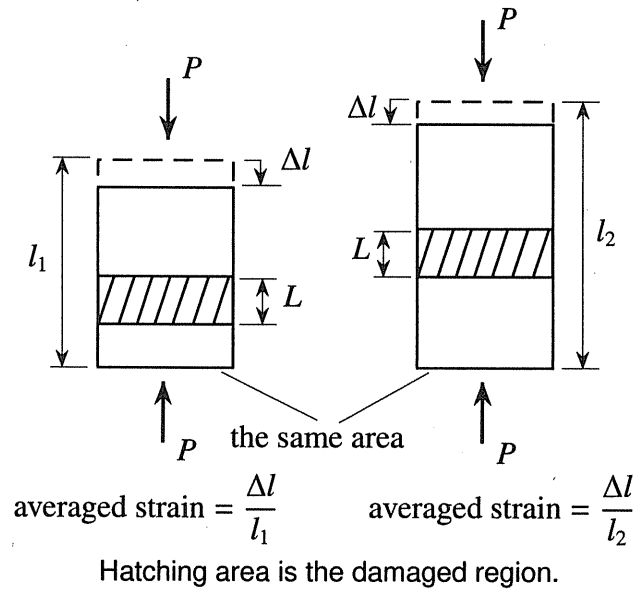


Figure 3.5 Averaged strain derived by uniaxial compression test

concrete resistance curve is significant.

One of the ideas to resolve this problem is to introduce a concept of control volume. The stress-strain relation averaged using a length should be applied in the elements whose lengths are longer than the length used in averaging. Many constitutive relations have been proposed based on specimens of approximately 20 to 30 cm in height. The parameters of the concrete constitutive relation determined in the previous section are also based on the fixed fiber element length of 15 cm. Therefore, it is considered that the constitutive relation proposed in the present study can be applied as long as we set the fiber element lengths to approximately 15 cm or more, and we here employ the model proposed by Hoshikuma et al. Note that the swelling length of longitudinal reinforcing bar can usually be assumed to be approximately 45 cm in the usual arrangements of reinforcements of cast-in-place RC piles. For example, in the pile foundation examined in Chapter 5.

(2) Plastic Buckling Analysis of Reinforcing Bar

a) Numerical model for plastic buckling analysis

Next, we carry out a series of plastic buckling analyses of reinforcing bars constrained by core concrete and hoops, as illustrated in **Figure 3.3**. The 2D finite element method is employed within a framework of finite displacement. A reinforcing bar is expressed by a group of fiber elements; that is, the cross section is divided into a number of layer-like elements.

We adopt the Menegotto-Pinto model as the constitutive model for each fiber of a reinforcing bar. Although it would be possible to use other models, rather than the Menegotto-Pinto model which was originally developed for the analysis of RC members subject to cyclic load, for consistency of analysis, we employ the model used for the analysis of the behavior of cast-in-place RC piles, as mentioned later. The spring constant of the spring representing the hoop resistance is obtained through a series of separate finite element analyses of an arch model subjected to a normal force at the top, as shown in **Figure 3.3**. In this model, the arch consists of a hoop reinforcement with both ends fixed and an open angle of 90 degrees. The distributed spring used to model the resistance of the core concrete does not resist tensile force, and the compressive spring constant is simply set large enough to prevent the longitudinal reinforcement from penetrating into the core concrete.

The reinforcing bar is divided into ten elements in the longitudinal direction, and the cross section is modeled by 50 layers. A distributed concrete spring is located at each node. Finally, a small imperfection is introduced into the coordinate of longitudinal reinforcing bar nodes without the end nodes in order to model residual curvature caused by bending-tension deformation along with bending loading of the pile. As a result, buckling can be triggered easily.

b) Determination of plastic buckling length of reinforcement bar

Two model cast-in-place RC piles are examined. These have been the subject of experiments where the behavior of piles subject to lateral cyclic loading was examined. One pile is a type-1 pile specimen as presented in Chapter 2 (1) and the other is summarized in **Figure 3.17**. We refer to the latter pile as the type-2 pile specimen. Material constants and parameters for the Menegotto-Pinto model are chosen to be identical to those employed in the numerical analysis described in **Figure 2.1** and **Table 2.1** in Chapter 2 (1), and **Figure 3.17** and **Table 3.2** in Section (3) b) of this chapter.

First, assuming that the hoops never yield, we obtain spring constants k_r which represent the hoop resistance by finite element analysis of the arch model shown in **Figure 3.3**. The value of k_r for the type-1 pile specimen is 1.7 MN/m, and 19.0 MN/m for the type-2 specimen. The spring constant of the distributed spring representing the core concrete must be sufficiently rigid and is set at 9.8 GN/m². As explained in Section (1) c), plastic buckling analysis of reinforcing bars of length $L = ms$ with various values of m is performed in order to determine the critical value $m = m_0$ which leads to the most unstable load-displacement history.

As for the type-1 pile, six cases, denoted as $m = 1, 2, \dots, 6$ are analyzed. **Figure 3.6** illustrates the initial states as dashed lines, and the corresponding deformed configurations at an average strain of 5% is shown by solid lines. The average strain is defined by the longitudinal displacement at the loading point per length L . The scale of each figure is also normalized by the length L . The springs are shown schematically in their figures, with both the initial and deformed states shown. The springs do not resist displacement of reinforcing bars in the longitudinal direction. The deformed configurations suggest that the buckling mode when $L = 5s$ and $6s$ is the same as when $L = 3s$.

Figure 3.7 shows the relationship between vertical load and displacement normalized by buckling mode in order to estimate the load-displacement relationship for each buckling mode. In all cases, buckling occurs immediately after initial yielding of the material.

Then, as can be expected from the buckled shape, the results for cases $L = 3s, 5s$, and

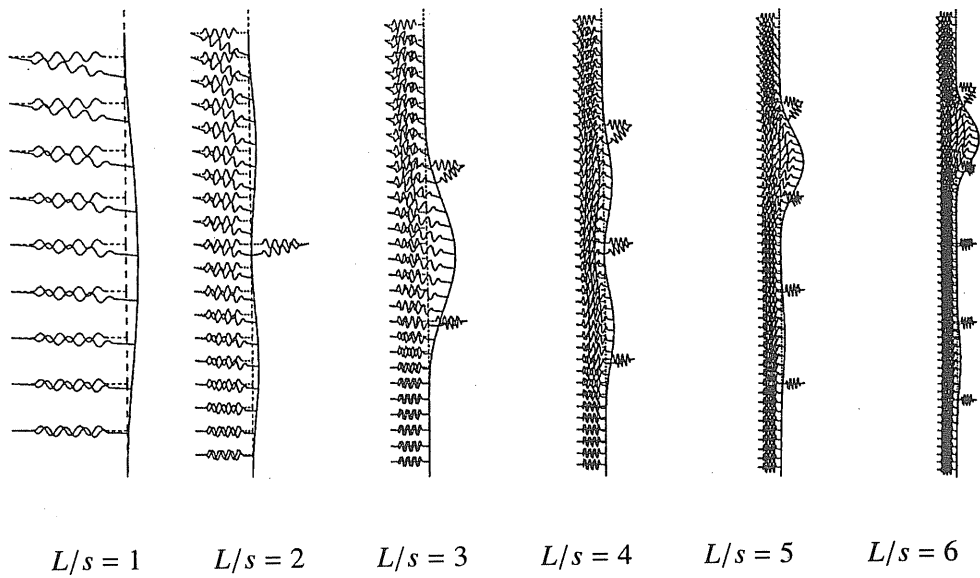


Figure 3.6 Deformed shape after plastic buckling of longitudinal bar in the type-1 pile specimen

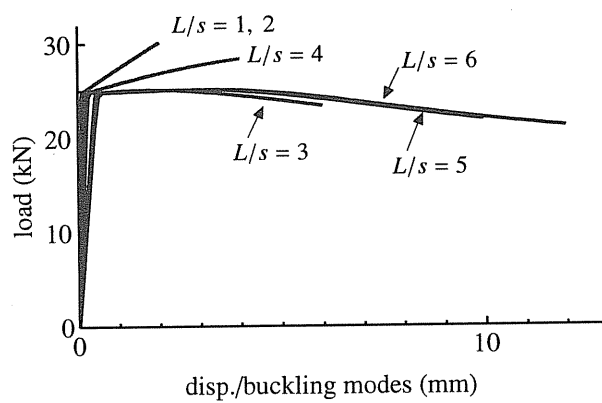


Figure 3.7 Normalized load-displacement relationship (type-1 pile specimen)

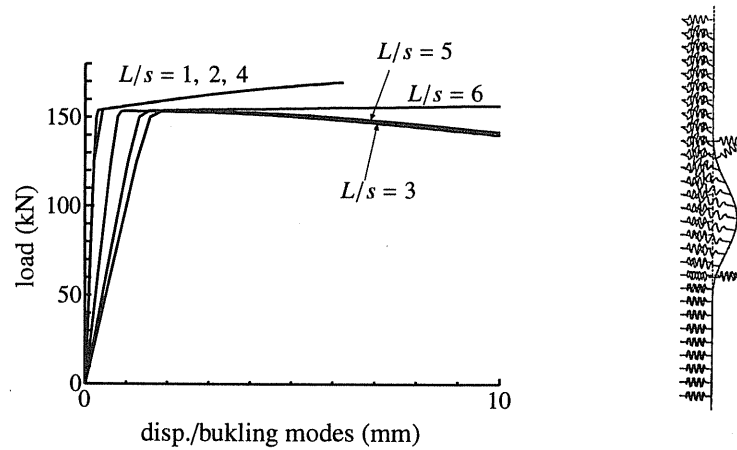


Figure 3.8 Normalized load-displacement relationship and plastic buckling deformation for case $L = 3s$ (type-2 pile specimen)

6s are similar and show a most significant decrease in load-carrying capacity. The length of the swelled portion of the longitudinal reinforcement bars observed in the experiments shown in **Photo 2.1** is approximately $3s$ (120 mm), which agrees well with the most unstable response obtained in this series of buckling analyses.

Similar results are obtained for the type-2 pile, as summarized in **Figure 3.8**. The deformation configuration when $L = 3s$ is also shown in this figure. In comparison with the experimental results in **Photo 3.2**, the shape of the swelling when $L = 3s$ (375 mm) coincides well with the experimental observations. The buckling mode at $L = 5s$ is also the same as that at $L = 3s$. Although there are differences in reinforcement arrangements between the type-1 and type-2 pile specimens, the buckled shapes corresponding to the most drastic decrease in the load-displacement curve are more or less the same as that observed experimentally.

These results indicate that plastic buckling is initiated by the loss of bending stiffness due to plasticization of the cross section, and that the buckling load is independent of length L . On the other hand, the buckling mode and post-buckling behavior depend on length L . However, comparison with experimental results for real piles shows that the actual swelling phenomena are strongly governed by the buckling mode, where the most drastic decrease in

the load-displacement curve occurs.

c) Differences in reinforcement bar behavior due to variations in hoop resistance spring rigidity

Although it is assumed in the plastic buckling analysis above that the hoop reinforcement remains elastic, it can also be imagined that the hoop reinforcement may yield if the horizontal swelling deformation of the reinforcement may become excessive. In order to examine such a possibility and the sensitivity to the plastic buckling behavior of longitudinal reinforcement bars, we attempt to analyze the response for three different definitions of the springs representing the hoop resistance, with the length to be analyzed fixed at $L = 3s$:

- an elastic spring (as in Section b)),
- an elastic and perfectly-rigid plastic (e-p) spring having a yield point defined by the state at which the outermost fiber of the arch model of the hoop yields first, and
- an equivalent linear spring with spring constant k_{fe} , which is the secant modulus of a spring at the point at which the longitudinal reinforcement undergoes vertical compressive deformation at $0.1L$ in the plastic buckling analysis with the elastic-perfect plastic springs. In this case, eventually, $k_{fe} = k_f/3$ in the case of the type-1 pile specimen, whereas $k_{fe} = k_f/1.4$ in the type-2 pile,

where it is supposed that the spring constants of the equivalent linear spring become so small that the results of the assessment of the performance of piles will err on the side of safety. In this analysis for length $L = 3s$, only two springs for the hoops are included and arranged symmetrically with respect to the central node in the longitudinal direction of the reinforcing bar. Therefore, both the springs deform quite similarly throughout the numerical analysis of plastic buckling.

Figure 3.9 shows the results for the reinforcement in the type-1 and type-2 pile specimens. In the type-1 case, the three models of the hoops yield completely different post-buckling behavior. The influence of the load-displacement curve model of the buckled lon-

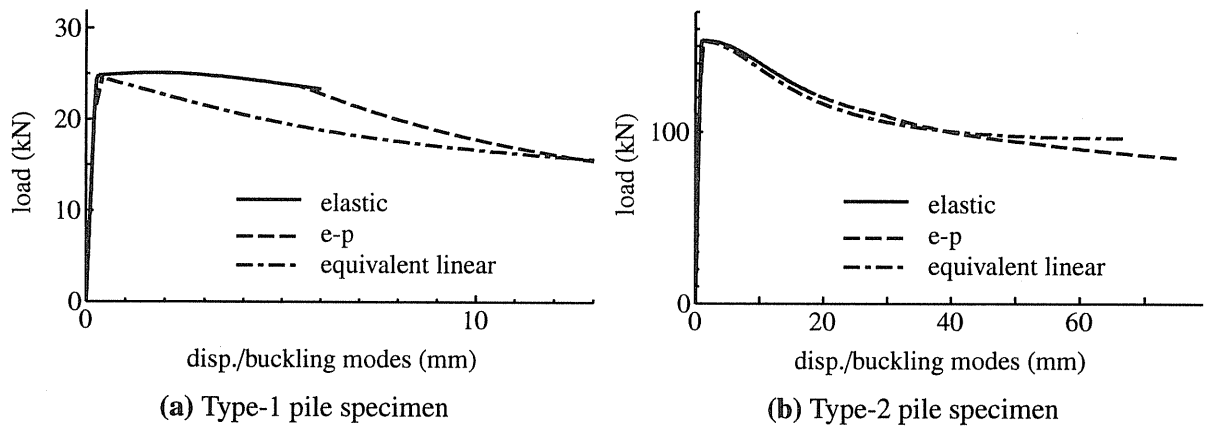


Figure 3.9 Difference in normalized load-displacement relationship caused by different spring constants for hoop resistance (type-1 and type-2 pile specimens, $L = 3s$)

gitudinal reinforcement on the behavior of the pile will be examined in the next section. On the other hand, the type-2 case exhibits no apparent difference due to the choice of hoop model. Other characteristics of post-buckling behavior of longitudinal reinforcement bar that depend on the hardening ratio, for example, have also been elucidated. A detailed discussion of such examinations is presented in Appendix B.

(3) Numerical Analysis of Cast-In-Place RC Pile Specimens Subjected to Reversed Cyclic Load

In order to verify the validity of the proposed model, experimental results obtained for cast-in-place RC piles subjected to cyclic loading will be numerically simulated using the proposed model. The results of experiments in which swelling of the reinforcing bars occurred due to bending are examined. The scale of the model specimens tested is sufficiently large compared to actual piles, and the specimens contain almost the same ratio of longitudinal reinforcement and volumetric ratio of lateral reinforcement as actual piles.

We use a two-dimensional finite element method within the framework of infinitesimal displacement theory. Each longitudinal reinforcing bar in a pile is modeled by a single fiber

element. Aside from the reinforcement, the cross section of a pile is first divided into 50 layers and then each layer is subdivided again into fibers of core concrete and cover concrete.

a) Experiment on the type-1 pile specimen subject to horizontal reversed cyclic load with a constant axial load

Numerical analysis model

We model the specimen as a cantilever beam in which the fixed end represents the pile cap. An element length of $3s$ (120 mm) is chosen because this is the smallest size which results in an unstable load-displacement relation among those tested in the plastic buckling analysis in section (2) b) of this chapter.

In order to check the effects of various types of post-buckling behavior of the reinforcement arising from differences in the spring representing the hoop resistance on predicted pile behavior, two averaged stress-strain relations of the plastic buckled reinforcement in **Figure 3.9 (a)** are tested. One relation is obtained using the equivalent linear spring, and the other is obtained using an elastic and perfectly-plastic spring. The cross-sectional area of a longitudinal reinforcement bar is as given in the JIS code, and the necessary material parameters are determined by element tests. However, since the hardening ratio b of the Menegotto-Pinto model is unavailable, it is assumed to be one-hundredth of the initial (elastic) rigidity, which is obtained from a standard tensile test of the longitudinal reinforcement in a type-2 pile specimen (SD345).

Since it is difficult to choose appropriate material parameters for the decreasing gradient of the concrete stress-strain relation, we use the parameters derived in Chapter 2 (3) b). Some problems may occur due to the chosen constitutive concrete relation, as discussed before, because the difference in element lengths between that used in the numerical examination here and in the simulation in Chapter 2 (3) b) is approximately 20%.

Loading in the numerical simulation also begins with application of a vertical load of 147 kN at the top of the pile. Then the horizontal cyclic displacement history is applied.

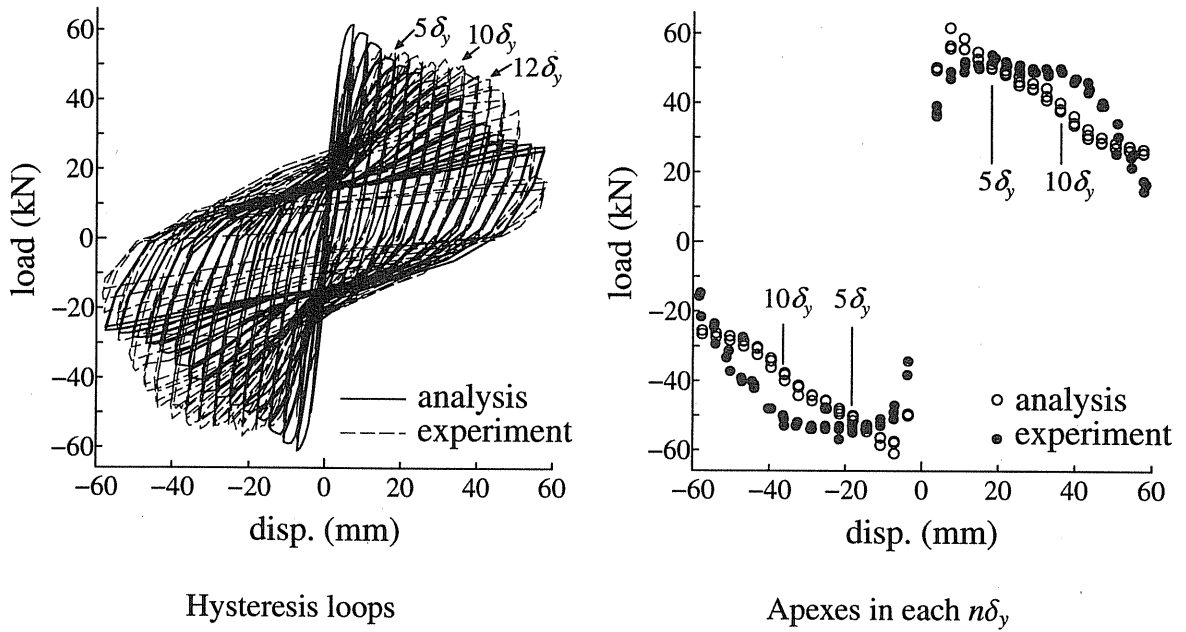


Figure 3.10 Horizontal load-displacement hysteresis at the loading point (type-1 pile specimen)

Numerical results and discussion

We first discuss the results of a case employing the stress-strain relation of a buckled longitudinal reinforcing bar, obtained by using equivalent linear springs for the hoops. This case undergoes a sudden decrease in the compressive resistance of the longitudinal reinforcement after yield, as shown in **Figure 3.9 (a)**. This case is selected to check the feasibility of the proposed numerical model in predicting softening behavior, because the strength of the reinforcing bar is reduced immediately after buckling begins and the estimated strength is lower than that obtained under the assumption of elastic perfectly-plastic springs. The calculated horizontal load-horizontal displacement history at the loading point is shown in **Figure 3.10** together with the experimental results.

The numerical analysis overestimates the loads up to the $4\delta_y$ cycle as compared with the experimental data. After that, swelling of the outer reinforcement begins in the $6\delta_y$ cycle, resulting in rapid reduction of the load. In the experimental results, this was observed during the 10 to $13\delta_y$ cycles. However, the outlines of the inner loops of the load-displacement relationships closely fit the experimental outlines at each cycle $n\delta_y$.

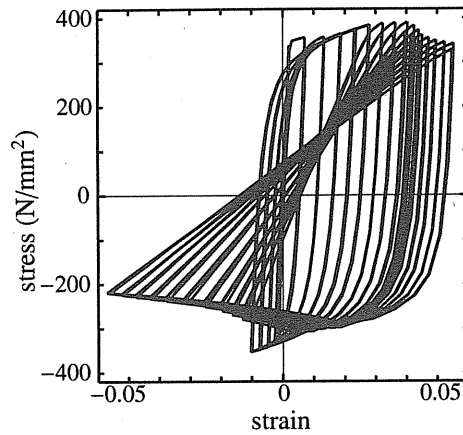


Figure 3.11 Stress-strain hysteresis of typical outermost longitudinal reinforcement bar in the element at the bottom

Figure 3.11 shows the hysteresis loops of the longitudinal stress-strain relation at the center of a typical outmost reinforcement bar in the element at the bottom. Swelling begins at the $6\delta_y$ cycle. The peak tensile stress at each step decreases as the pile deformation becomes greater. This indicates a progressive accumulation of residual swelling deformation and is in good agreement with experimental observations, as shown in **Photo 2.1**.

For the sake of comparison, another numerical result of the Menegotto-Pinto model reinforcement without swelling is illustrated in **Figure 3.12**. This model is referred to as the conventional model: Note that the element length employed in this reference calculation is different from the element length used in the present study. The element length employed in the reference calculation is one half of the diameter of the pile specimen (150 mm), same as the calculations in Chapter 2. Since the models are identical except for this difference, the envelope curves obtained in **Figures 3.10** and **3.12** are very similar. Furthermore, both calculated envelopes are larger than the experimental data from cycle 1 through $4\delta_y$. With respect to the shape of the envelopes, after the $4\delta_y$ cycle and up to the $10\delta_y$ cycle, the simulations agree relatively well with the experiments.

However, the conventional model is unable to predict phenomena such as the excessive decrease in load observed in cycles over $10\delta_y$ in the experiment, and ultimately it overestimates the ultimate strength. Judging from these comparisons, we must conclude that rein-

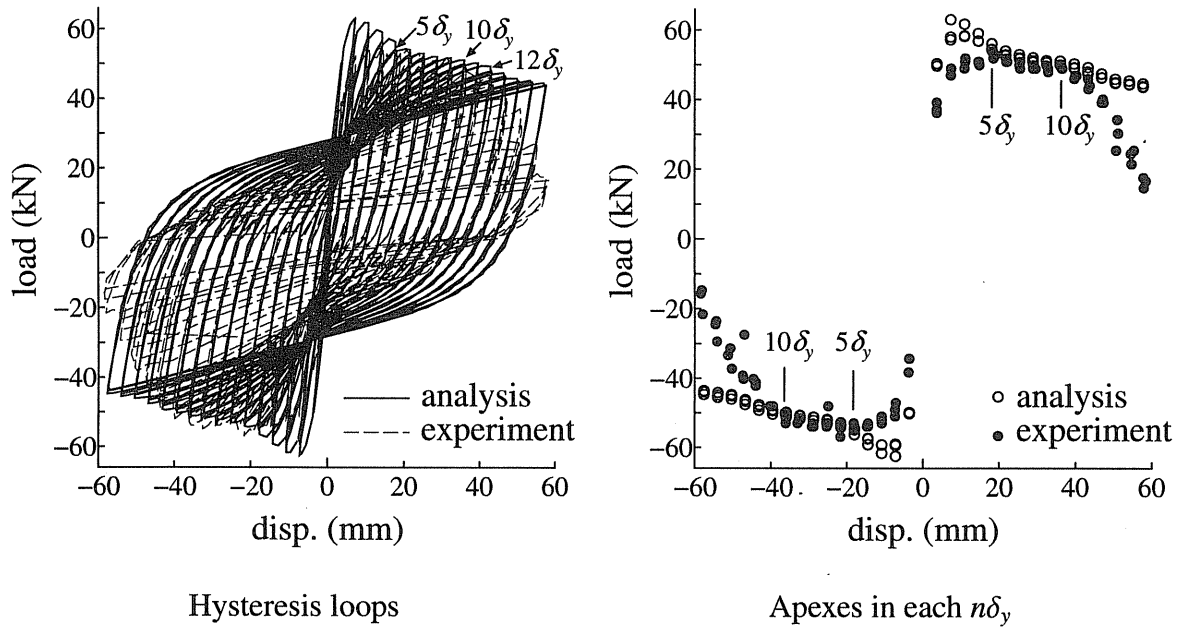


Figure 3.12 Horizontal load-displacement hysteresis at the loading point for the case without considering the swelling of reinforcing bars (type-1 pile specimen)

forcement swelling plays a very important role in the deformation history of piles at large displacements, and particularly the transition to rapid loss of load-carrying capacity and the shape of the inner loops of the hysteresis response.

Even the proposed model, however, has several problems. For example, the load is overestimated up to the $4\delta_y$ cycle, and the capacity of the pile rapidly decreases at a much earlier loading stage (at the $6\delta_y$ cycle) than indicated in the experimental results. These numerical characteristics result from the constitutive law used for the cover concrete, especially its skeleton curve. Of course, Young's modulus and the maximum compression strength of the constitutive model strongly influence both the predicted load at early stages of the loading history and the maximum strength of the pile. Moreover, the descending gradients of the resistance curves also have an extraordinary effect on the loss of load-carrying capacity, since the collapse of the cover concrete is a necessary condition for the development of reinforcement swelling.

In the numerical analysis, while the cover concrete fractures at the position of a typical outermost longitudinal reinforcing bar at the $5\delta_y$ cycle near the base of the pile, the core con-

crete experiences compressive failure during the $10\delta_y$ cycle. In contrast, in the experiments, only slight damage is observed on the surface at the $5\delta_y$ cycle, and excessive damage to the cover concrete starts after approximately the $12\delta_y$ cycle. Namely, in the numerical analysis, the initiation of swelling of the outmost reinforcement occurs at the $6\delta_y$ cycle, as shown in **Figure 3.11**, at which the cover concrete provides adequate confinement in the experiments.

Figure 3.13 is a side view of the specimen in schematic form, showing damage to the fibers obtained numerically after the final $13\delta_y$ cycle. In the two diagrams on the left-hand side, the areas with hatching indicate portions that experience tension cracks, while black areas represent concrete in which compressive collapse occurs. In the diagram on the right, the dotted lines represent the positions of longitudinal reinforcement, the areas with dashed lines are the yielded zones, and those with solid lines are the swelled sections. Compressive failure of the concrete and swelling of reinforcement occur only in the bottom elements; the element length is set at three times the spacing between two adjacent hoops (120 mm). These characteristics in the longitudinal direction coincide well with those observed in the experiment, as seen in **Photo 2.1**. On the other hand, in the cross-sectional direction, concrete damage progress is quite different in the numerical simulation and the experiment. **Photo 2.1** of the experiment shows that the damage does not penetrate across the position of the outmost reinforcement bars into the core section, while the numerically obtained damage, shown in **Figure 3.13**, expands into the inner region of the core concrete, and, accordingly, all except the central reinforcing bars buckle. The numerical analysis over-predicts the extent of damage.

This suggests the need to improve the peak branch with respect to the present concrete constitutive relation such that it has a sustaining branch like CC-R, as shown in **Figure 2.3**, and to modify the softening behavior branch below $0.5\sigma_{cc}$. Cast-in-place RC piles have a larger ratio of cover concrete depth to effective depth of section compared to other RC members, for example, a pile of 1 m in diameter generally has a cover concrete depth of 15 cm, but a pier of the same diameter has a cover concrete depth of 7 to 9 cm in Japan. As a result, in numerical analyses of cast-in-place RC piles, the modeling of cover concrete is

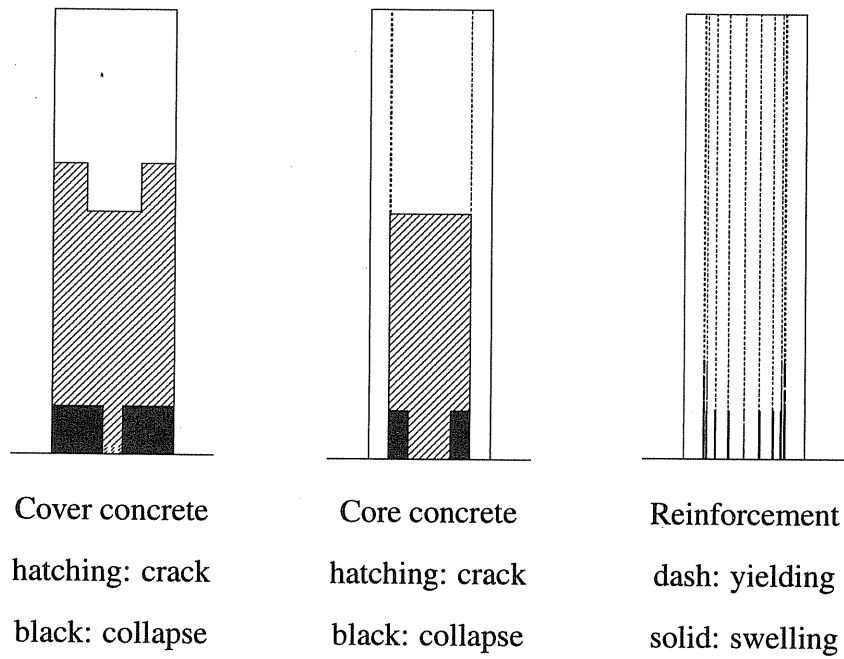


Figure 3.13 Damage state immediately after the final $13\delta_y$ cycle

much important than that in the numerical analyses of other kinds of RC members.

We carry out the numerical simulations of the pile using the different stress-strain curve of the buckled longitudinal reinforcement which is supported by the elastic and perfectly-plastic springs for the hoop resistance. However, there is no noticeable difference between these results, as shown in **Figure 3.14**, and the size of the discrepancy is no more than a few percent. This suggests that differences in the nonlinear properties of hoop resistance do not cause serious variations.

Maekawa et al.²⁷⁾ pointed out that the pull-out effect of longitudinal reinforcement occurring both at the interface between the pile and the pile cap and in the pile cap affects the numerical result for this pile experiment based on their recent numerical study. According to their comparative study, the damage to the pile around the base is overestimated where there is no consideration of the pull-out effect. Accordingly, another numerical check is performed for another experiment using the same specimen.

A cyclic pure bending test of a pile specimen of the same pile was also performed by Kimura et al.^{31),32)} Shown in **Figure 3.15** is an illustrative diagram of the test set up. Two sections near the ends were fixed by bands and controlled displacement was simultaneously

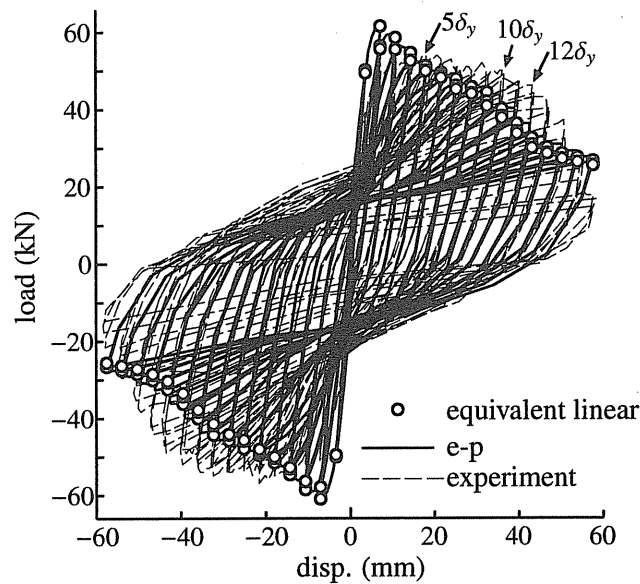


Figure 3.14 Difference in numerical results of horizontal load-displacement hysteresis at loading point depending on hoop spring modeling (type-1 pile specimen)

applied to two sections near the center through bands such that the uniform bending moment occurred in the sections between the loading sections. The applied load was reversed cyclically with three cycles in each displacement amplitude, where the displacement amplitude gradually increases from $1\delta_y$ to $n\delta_y$ and $1\delta_y$ was a presumed displacement at the time when a typical reinforcement bar reaches the yield point. **Figure 3.16** shows the load-displacement curve at a loading section. The load examined in **Figure 3.16** is half of the jack load and the displacement is the averaged value of the measured values at the loading sections. The test was not completed in the post peak region because of the capacity of the loading equipment. **Photo 3.1** presents the damage state after the experiment. Cracks were widely distributed, but no severe damage, such as spalling off of cover concrete, was observed.

In this calculation, the element length is 125 mm, because the element length should accommodate the distance between the loading sections of 1000 mm. This element length is set to between 120 mm and 150 mm in the simulations of which the results are shown in **Figures 3.12** and **3.13** and **Figure 3.14**, respectively. Since neither the collapse of cover concrete nor reinforcement swelling occurred for this pure bending experiment, the difference

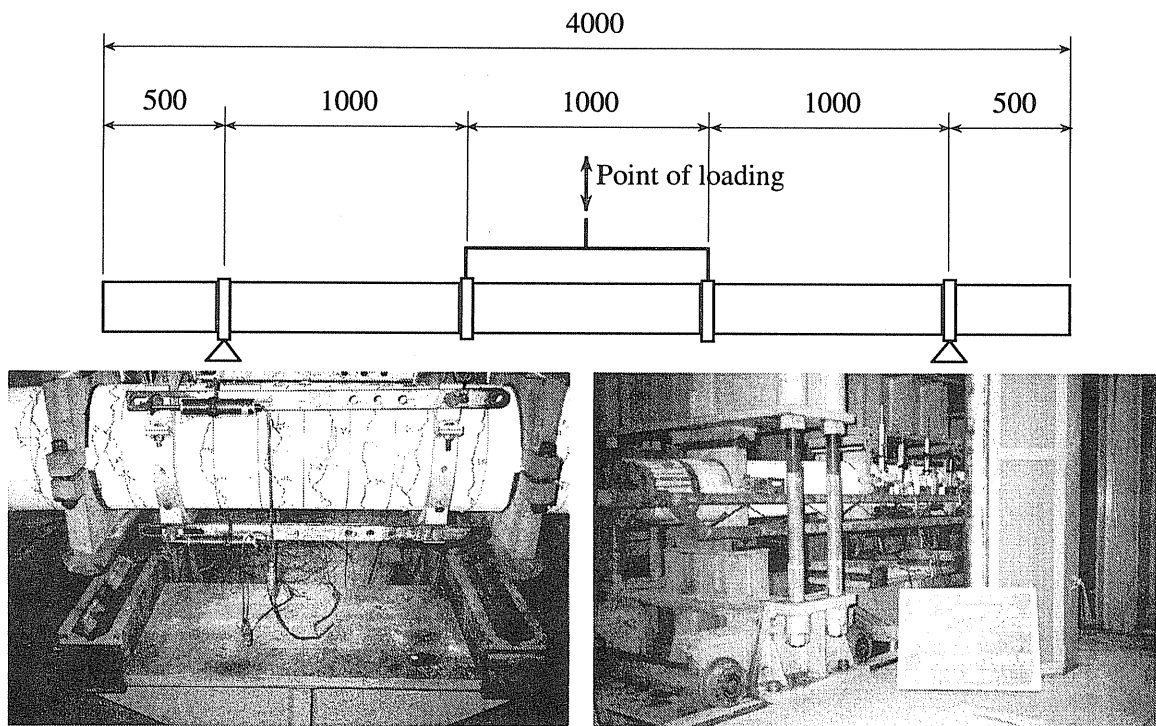


Figure 3.15 Set up for a pure bending experiment of type-1 pile specimen

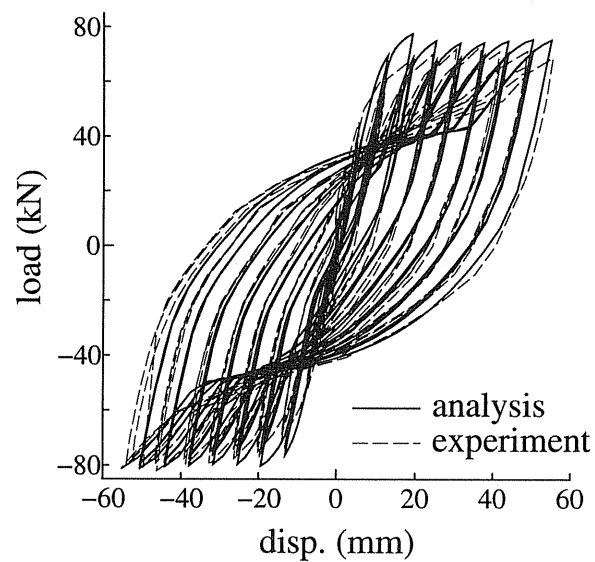


Figure 3.16 Load-displacement curve at a loading section, experimental and numerical results

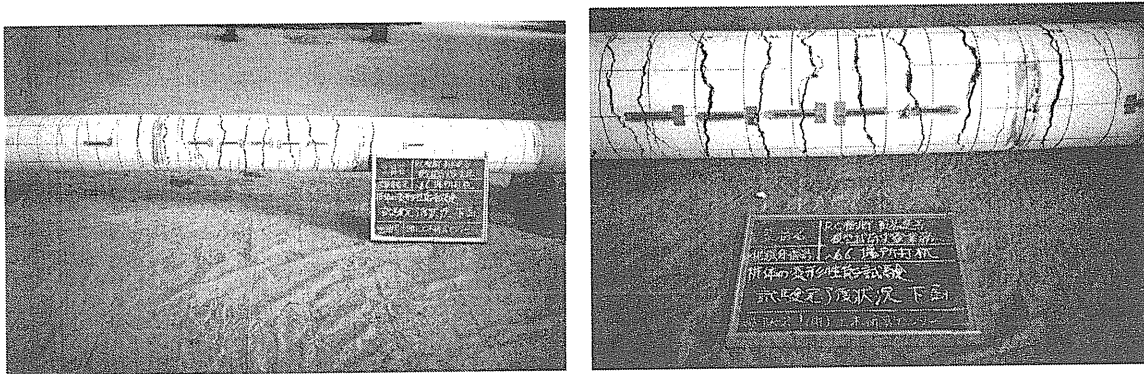


Photo 3.1 Cracks developed in type-1 pile specimen after the pure bending experiment

in the element length should not cause a noticeable difference in the numerical accuracy. The result of a standard material element test in this experiment is shown in **Table 3.1**, and the plastic buckling analysis of a reinforcement bar with a length of 125 mm is conducted in order to estimate the uniaxial compression behavior with reinforcement swelling, where elasto-plastic type springs are used as the hoop resistance. Both vertical and horizontal displacements are fixed at a particular side, and only the horizontal displacement is fixed at the opposite side.

The calculated load-displacement curve at a loading point is also shown in **Figure 3.16**, together with the experimental result. Good agreement is observed in the load displacement curves of numerical and experimental results, whereas the calculated load of 2 and 3 δ_y cycles is overestimated as in **Figures 3.10** and **3.14**. This overestimation is considered to be caused by the effect of distributed cracks on the evolution of plasticization of the longitudinal reinforcement bar. Longitudinal reinforcement bars did not develop uniform strength along the longitudinal direction due to a non-uniform elongation. The calculated damage also coincides well with the experimental damage. Reinforcement swelling does not occur in the numerical result either, and only the surface fibers of cover concrete reach the ultimate compressive strain. The numerical prediction is good compared to that for the experiment involving the pile cap.

As a consequence, as has been indicated by Maekawa et al., it is assumed that the pull-out effect of longitudinal reinforcement bars in the pile cap strongly affects the numer-

Table 3.1 Results of material element tests for spp-1 pile specimen corresponding to the pure bending test

Reinforcement		
Reinforcement	Yield stress (N/mm ²)	Young's modulus (kN/mm ²)
Longitudinal	351.3	185
Hoop	395.5	225

Concrete		
Compressive strength (N/mm ²)	Young's modulus (kN/mm ²)	Splitting strength (N/mm ²)
28.6	29.4	2.28

ical prediction of the experiment of this pile specimen with a footing. On the other hand, the possibility may exist that the pull-out effect would not influence so much in the lateral behavior of piles having large scale sections, as will be shown later as an example for the simulation result of type-2 pile specimen. More detailed future research on the modeling of cover concrete and the pull-out effect from the pile-cap employed in the fiber model is needed in order to resolve this problem.

b) Experiment on type-2 pile specimen subject to horizontal reversed cyclic load and varying axial load

Summary of experiment

The type-2 pile specimen chosen for another examination of the proposed model is the cyclic load test of a model cast-in-place RC pile performed by Tanamura et al.³³⁾ A summary of this experiment and the results of standard element tests are shown in **Figure 3.17** and **Table 3.2**, respectively. A specimen of this size can be considered to be a very large model, since more than 80% of cast-in-place RC piles used for highway bridges in Japan are

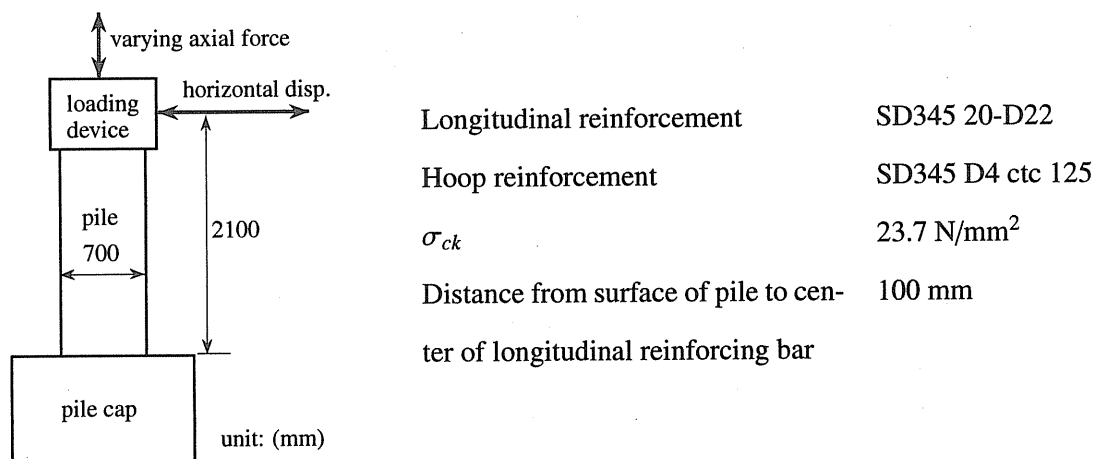


Figure 3.17 Experiment of type-2 pile specimen

either $\phi 1000$ or $\phi 1200$ in diameter.³⁴⁾ Horizontal loading was controlled by displacement, with the drift angle defined by the ratio of horizontal displacement applied at the loading point to the shear span, specified as follows: after consecutive cycles at angles of 1/1000 and 1/400, once at each magnitude, the amplitude was gradually increased ten times from 1/200 to 10/200 in steps of 1/200, and three cycles were applied at each magnitude.

One of the most noteworthy features of this experiment is that the pile was also subjected to an axial force which varied according to the horizontal reaction force, as depicted by the dashed lines in **Figure 3.18**. This loading condition was employed in order to simulate the actual loading states observed in grouped pile foundations subjected to horizontal displacement during earthquakes.

Photo 3.2 shows the damage states visible after the test. The longitudinal reinforcement bars swelled greatly between adjacent hoops, and the hoops swelled to some extent as well. **Figure 3.19** illustrates the horizontal load-horizontal displacement hysteresis at the loading point. The experimental results can be summarized as follows. The cover concrete collapsed on the compressive side of the pile at the bottom when the axial force increased in the compressive direction during the cycle with a drift angle of 2/200. When the displacement was reversed in the same cycle, the longitudinal reinforcing bars yielded as they moved from compression to tension, and the cover concrete on the opposite side, where the fiber stress changed from tension to compression, also collapsed simultaneously. Then, during the

Table 3.2 Results of material element tests of type-2 pile specimen

Reinforcement				
Reinforcement	Yield stress (N/mm ²)	Tensile strength (N/mm ²)	Young's modulus (kN/mm ²)	Extension (%)
Longitudinal	397	589	195	23.2
Hoop	397	569	189	22.8

Concrete			
Age (days)	Compressive strength (N/mm ²)	Young's modulus (kN/mm ²)	Splitting strength (N/mm ²)
10	23.7	21.9	2.4

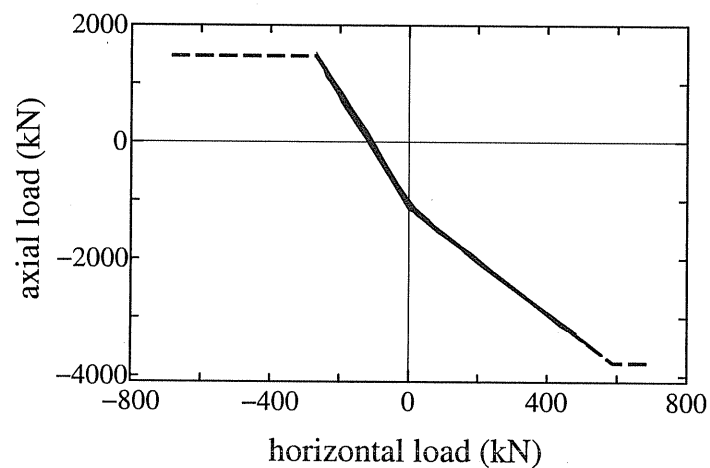


Figure 3.18 Pattern of varying axial load (dashed lines); tension is positive. Solid lines indicate the numerical results.

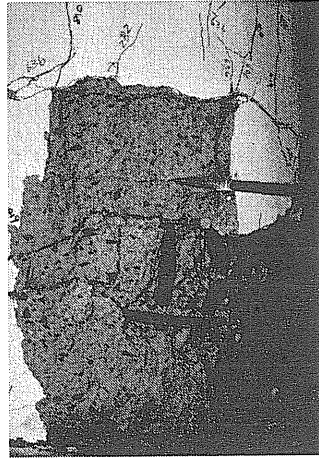


Photo 3.2 Damage at bottom of type-2 pile specimen after the experiment (compression side when the pile was subject to compressive axial force)

phase of increasing axial compression, the longitudinal reinforcing bars on the tensile side of the pile yielded for the first time at the cycle with a drift angle of 4/200. At the cycle with a drift angle of 6/200, the hoops started to yield. At the last cycle with a drift angle of 10/200, the longitudinal reinforcing bars in tension failed when the axial force changed from compression to tension.

Numerical analysis model

We model the specimen as a cantilever beam, as in the analysis of the type-1 pile specimen. Based on the results of a series of the numerical examinations for plastic buckling of the reinforcement, the element length is three times larger than the spacing between two adjacent hoops, as shown in **Figure 3.8**. The average stress-strain relation for the swelled reinforcing bars was based on the post-buckling behavior obtained using equivalent linear springs for the hoop resistance in **Figure 3.9 (b)**. As is shown in **Figure 3.9 (b)**, the post-buckling behavior of the longitudinal reinforcement in the arrangement in the type-2 pile specimen does not depend on the mechanical properties of the springs representing the hoop resistance.

We set the necessary sectional parameters and material parameters for the constitutive model of longitudinal reinforcement as in the simulation used for the type-1 pile specimen.

Table 3.3 Concrete material parameters in numerical analysis of type-2 pile specimen

Cover concrete			
σ_{cc} (MN/m ²)	ε_{cc}	ε_U	ε_L
23.2	0.00200	0.00790	0.0197

Core concrete			
σ_{cc} (MN/m ²)	ε_{cc}	ε_U	ε_L
40.7	0.00650	0.0220	0.0529

The hardening rigidity E_1 is set at one-hundredth of the initial rigidity (Young's modulus) E_0 , which is compatible with the order of the gradient between the yield point and the failure point measured in the material element test. The parameter values for the constitutive model of the cover concrete are the same as those in the numerical simulation for the type-1 specimen, as shown in **Table 3.3**.

Numerical results and discussion

In the numerical analysis, the horizontal displacement and axial load are input simultaneously. Of course, it is recommended that the axial load corresponding to the horizontal reaction force be input as in the experiment explained in **Figure 3.18**, but the software used does not provide a subroutine that makes this possible. We therefore determined the axial load history by trial and error through parametric numerical analyses with respect to several cases of axial load history such that the discrepancy is minimized in hystereses of the horizontal reaction force and axial load at each loading point between the numerical results and the experimental results. The calculated axial load-horizontal load hysteresis is indicated by the solid line in **Figure 3.18** together with the original setting in the experiment.

The calculated horizontal load-horizontal displacement hysteresis is shown in **Figure 3.19**. Both the envelope and the loop curves fit well the experimental data. The shapes of the envelope and loops resulting from the varying axial force are also well simulated, demonstrating the success of this scheme in taking such experimental variations into account

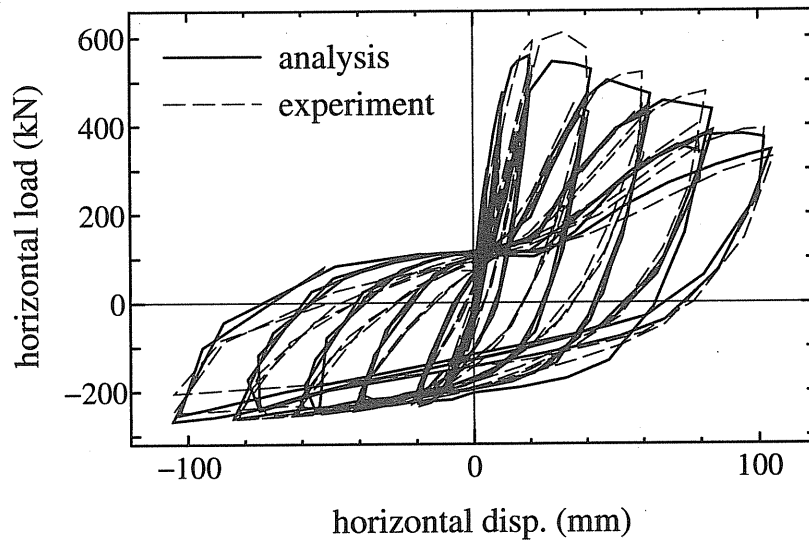


Figure 3.19 Horizontal load-horizontal displacement hysteresis at the loading point (type-2 pile specimen)

in the numerical analysis. However, the load at the final cycle of drift angle is overestimated in the negative load and displacement region. In the load state with negative horizontal displacements and axial tension force at this cycle, the longitudinal reinforcement fractured in the experiments. However, since the constitutive model of reinforcement in the numerical analysis does not include any failure criteria, such a fracture could never be predicted—so the load may be overestimated at such times.

Figure 3.20 shows the simulated damage results for the specimen at the cycle with a drift angle of 10/200. This figure is drawn such that the compressive axial force increases as the horizontal displacement is applied in the right-hand direction. The predicted pattern of damage distribution is quite similar to the experimental results and is non-symmetric. Although the area of damage in the experiment is slightly broader in the longitudinal direction compared with the numerical results, the patterns of collapse of the cover concrete are identical in the simulation and experiment, with the concrete being crushed and falling away. Judging from these results, we conclude that this numerical model can simulate well the development of progressive damage that is typically observed in experiments, and that the method of taking into account the effects of variations in axial load is appropriate.

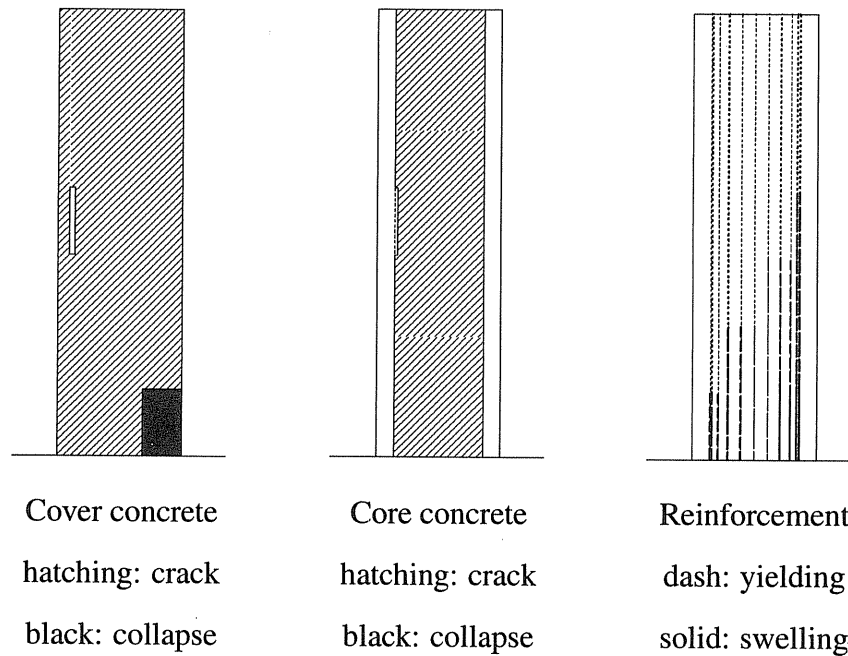


Figure 3.20 Estimated damage after final cycle

As for the progression of damage within the cross-section at the bottom of the pile, in the numerical simulation, the cover concrete collapses at the position of an outermost reinforcing bar during the cycle with a drift angle of $6/200$, and hence swelling of an extreme longitudinal reinforcement bar begins in the $8/200$ cycle of the drift angle. On the other hand, in the experiment, yielding of a hoop was detected in the cycle with a drift angle of $6/200$, and we can assume the occurrence of great damage to the cover concrete at this time. This comparison allows us to conclude that the evolution of damage within the section is also predicted well by the numerical analysis. In other words, accurate simulation of the behavior of this pile from the elastic region to postpeak conditions is possible in this particular analysis, mainly because damage progress in the cover concrete within the section as well as in the longitudinal direction is quite well predicted in accordance with the experimental observations. This indicates again that improvements in the modeling of the cover concrete are one of the most important problems to be solved, as mentioned in the discussion of numerical analysis results for the type-1 pile specimen.

In seismic design of foundations, the residual rotation angle after an earthquake is another important problem in reuse, and, for the pile foundation, the rotation angle is due to

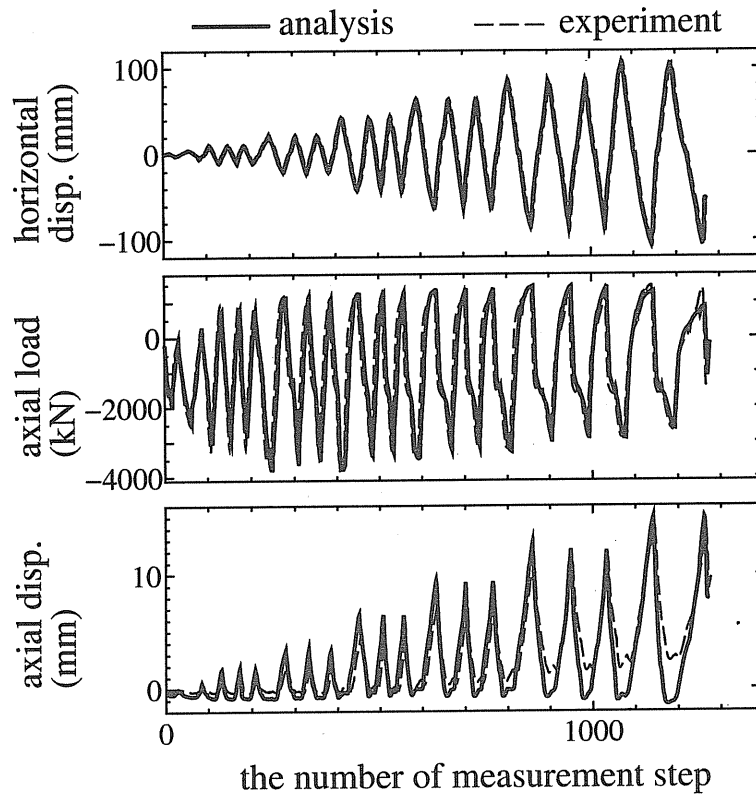


Figure 3.21 Histories of axial load, axial displacement, and horizontal displacement at the loading point: tension is positive for both axial load and displacement (type-2 pile specimen).

both bending deformation and axial displacement at the pile top of each pile. Furthermore, the axial displacement at the pile top is due to both the plastic deformation of each pile body, such as compressive failure of concrete and swelling of the reinforcement, and the bearing property of the ground. The proposed modeling can essentially estimate the elastoplastic axial extension-compression deformation of piles for the plastic deformation of pile bodies. However, this is not possible when using beam elements with the moment-curvature model. **Figure 3.21** compares histories of the axial load, axial displacement, and horizontal displacement at the point of load application between the experiment and the simulation, where the abscissa indicates the number of measurement steps during the experiment. **Figure 3.22** also shows their hysteresis loops. The simulation can estimate the increase in the extension along with the increase in the horizontal displacement with the tension axial load, similar to

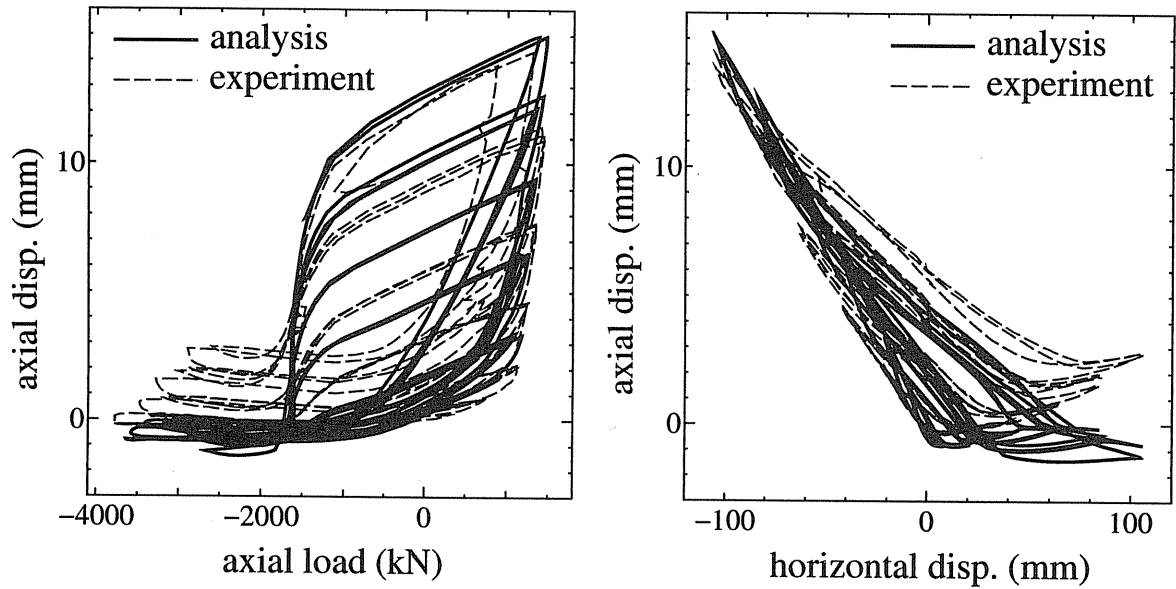


Figure 3.22 Hystereses of axial load, axial displacement, and horizontal displacement at the loading point: tension is positive for both axial load and displacement (type-2 pile specimen).

the experimental result. However, when we emphasize the values of the axial displacement at -1117 kN of the axial load in the unloading paths, which corresponds to zero horizontal load as stipulated in **Figure 3.18**, the residual extensions in the experiment increase as the loading progresses, while the numerical analysis cannot predict such accumulating behavior. Whereas the finite element model used here does not include a special element to model the connection between pile and pile cap, in the experiment, plastic strain may occur in the reinforcement in the anchored region in the pile cap due to large tensile stress. Consequently, we should attempt to obtain data on the strain distribution of reinforcement bars in the anchored region in the pile cap in order to estimate the axial tension displacement of piles.

(4) Concluding Remarks

Based on the numerical results obtained in these two verification experiments, we conclude that the proposed model is able to estimate the mechanical properties of cast-in-place

RC piles to a moderate degree of accuracy, regardless of the pile details. The proposed method is a powerful tool by which to trace the seismic collapse behavior of such piles from the elastic to the postpeak states consistently. Further studies are necessary in order to improve the accuracy of the numerical simulations, and certain points related to the proper criteria for reinforcing bar swelling must be elaborated. In particular, a more suitable stress-strain relation for the cover concrete must be modeled so that the sustaining branch path of maximum compressive stress and the diminishing gradients can be appropriately and quantitatively represented.

4 . Improvement in the Ductility Characteristic of the RC Pile in Soil Due to the Lateral Confinement Effect by Earth Pressures

It is of importance to investigate the lateral strength and ductility capacities of piles in order to establish a ductility design method of pile foundations. Many experimental results show that compressive collapse of cover concrete and reinforcement swelling play an important role in the collapse behavior of RC piles subjected to cyclic lateral loads, including cast-in-place RC piles and prestressed concrete piles. For example, the Japanese Specifications for Highway Bridges²⁾ recommends a limit value of system displacement ductility factor of 4 for pile foundations of highway bridges, based on cyclic load tests of single piles and some pile-groups,^{3),5)} as mentioned in the introduction. Such experimental results of RC piles, including cast-in-place ones and prestressed concrete piles, were obtained in air, because it is difficult to perform a direct observation of collapse process in piles under ground. However, under the ground surface, earth pressures from surrounding soils are expected to provide lateral confinement to the cover concrete of the pile and to improve the ductility of the pile.

In previous chapters, a modeling method of RC piles which can trace the behavior of a pile from elastic to postpeak using fiber elements. We simulate a collapse process observed in an experiment of model cast-in-place RC pile embedded in loose sand and subjected to a lateral reversed cyclic load at the top.

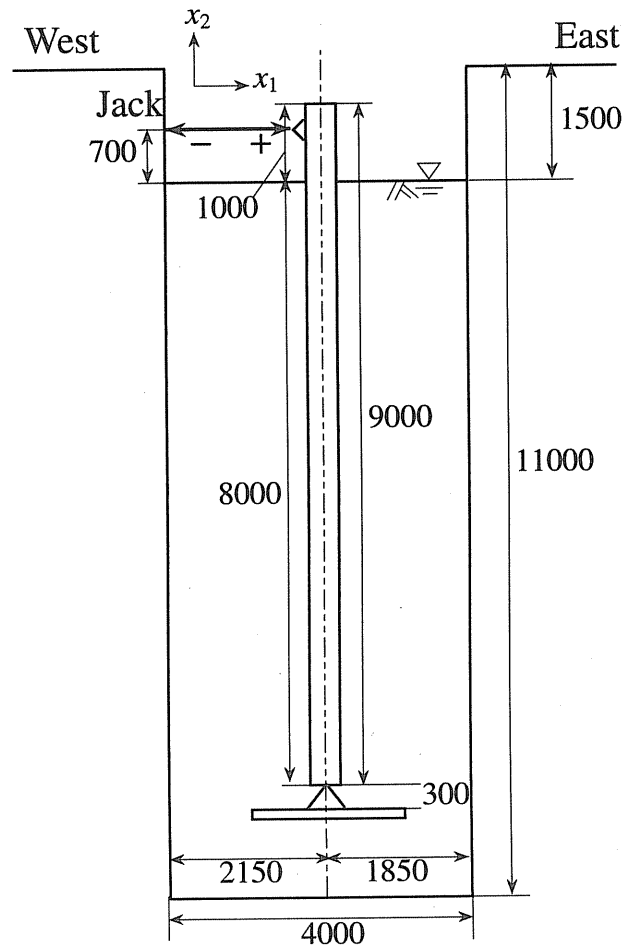


Figure 4.1 Test setup (unit: mm)

(1) Examined experiment

The experiment discussed herein was performed by Kimura et al.^{31),32)} Figure 4.1 illustrates the set up of experiment. The test soil deposit was made of Kashima sand in a deep test pit at the Foundation Engineering Laboratory of the Public Works Research Institute. The deep test pit has a length of 4 m and a width of 3 m in plan view, and is 11 m deep.

The sand was loose and saturated. The constructed soil deposit has the average relative density D_r of 17.3% and the average dry unit mass of the soil deposits estimated from core-cutter samples of 1.51 t/m^3 , respectively. During the experiment, the water level was maintained at the ground surface level.

The unit mass of the soil particles was 2.64 t/m^3 , and the maximum and minimum

Table 4.1 Strength data with respect to consolidation stress for the Kashima sand of $D_r = 30\%$

σ'_c (kN/m ²)	0.3×98	0.5×98	0.7×98
ϕ' (deg.)	39	36	35

$$(1 \text{ kgf/cm}^2 = 98 \text{ kN/m}^2)$$

dry unit masses are $\rho_{d\max} = 1.79 \text{ t/m}^3$ and $\rho_{d\min} = 1.50 \text{ t/m}^3$, respectively. The sand was composed of 1.8% gravel, 97.4% sand, 0.8% silt, 0% clay, and the coefficient of uniformity U_c is 2.66, and the median grain size D_{50} is 0.67 mm. Drained triaxial compression tests were conducted at several magnitudes of the consolidation stress σ'_c and the relative densities D_r in order to determine the internal friction angles. Estimated internal friction angles with respect to consolidation stress at $D_r = 30\%$ ($\rho_d = 1.53 \text{ t/m}^3$) are listed in **Table 4.1**. These internal friction angles were estimated under the assumption of $c = 0$, where c is cohesion. In other words, the one-point method was used. Cyclic triaxial compression tests provided a small strain shear modulus G_0 of this sand, which can be regarded as being a result of a shear strain level order of 10^{-4}

$$G_0 = 1935(\sigma'_c/98)^{0.65} \times 98 \quad (\text{kN/m}^2) \quad (4.1)$$

as obtained from a simple correlation of the test results of $D_r = 30\%$, where the unit of σ'_c is kN/m².

An RC pile specimen of the type-1 pile which was dealt with in previous chapters was also used. We refer to the experiment investigated here as the Case C1 experiment. The material properties obtained in typical element tests are shown in **Table 4.2**.

The point of application of the lateral cyclic displacement was 0.7 m above the initial ground surface. Positive displacement and jack load are indicated in **Figure 4.1**. A pinned device was attached at the bottom of the pile in order to clarify the boundary condition.

The lateral load versus lateral displacement response at the loading point is shown in **Figure 4.2**. The load intensities of positive and negative sides were quite different, whereas

Table 4.2 Results of material element tests for the RC pile specimen embedded in sand

Reinforcement			
Reinforcement	Yield stress (N/mm ²)	Tensile strength (N/mm ²)	Young's modulus (kN/mm ²)
Longitudinal	357.5	478	175
Hoop	389	554	213

Concrete		
Compressive strength (N/mm ²)	Young's modulus (kN/mm ²)	Splitting strength (N/mm ²)
30.3	27.1	2.70

the experiments in the same experiment series using steel piles^{31),32)} did not reveal any such noteworthy feature and the difference was small. The experiments using the same kind of specimens and being subjected to lateral reversed cyclic loads in air did not behave in such the aforementioned manner. The maximum strength was reached at 7 δ cycle (105 mm) and the positive peak load at every displacement cycle gradually decreased, whereas the corresponding negative peak load continued to increase gradually. Finally, a large and sudden decrease in strength was observed only in the final displacement cycle in both the positive and negative sides of load.

Photo 4.1 shows the state of the test ground after the experiment. The water level was reduced. The direction of cyclic loading was parallel to a line connecting the top-right with the bottom-left of this photo. Slumping of soil was observed around the pile, and passive deformation and cracks were developed in the surrounding slumping zone. In particular, a large subsidence of approximately 35 to 50 cm was developed around piles. In addition, soil densification occurred, since the values of cone penetration strength which were measured after each experiment increased compared to those measured before the experiment. These

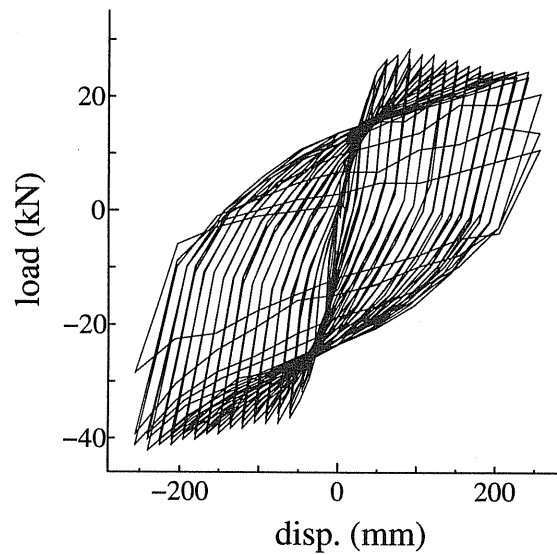


Figure 4.2 Load-displacement hysteresis at the load point

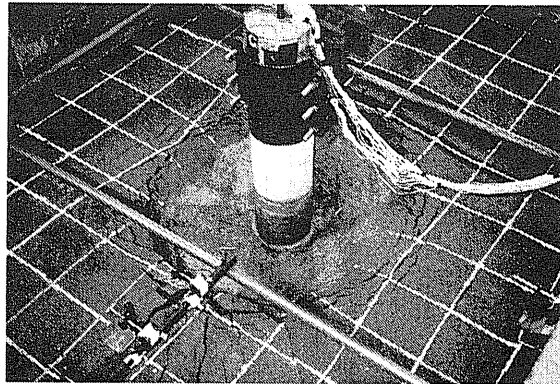


Photo 4.1 State of the test ground after the experiment and dewatering

observations indicate the development of a volume change, i.e. negative dilatant deformation, of the sands.

Photo 4.2 illustrates the final pile crack pattern observed after excavation, where a sketch was drawn for the east-side of the specimen. The scales of the photo and the sketch are different, and the hatching zones in the sketch indicate spalling of cover concrete. Damage was not localized very much, but rather was distributed over a large area, which differed from the experiment in air using the same type of specimen, as shown in **Photo 2.1**. **Photo 4.3** was taken after removing the cover concrete. Although swelling and tensile failure of reinforcing

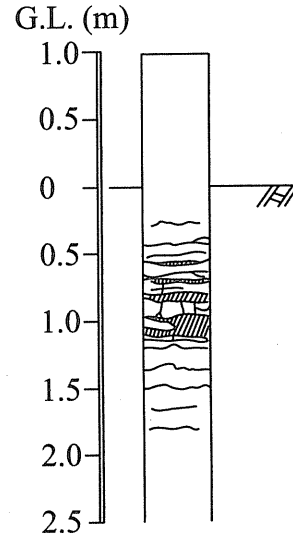
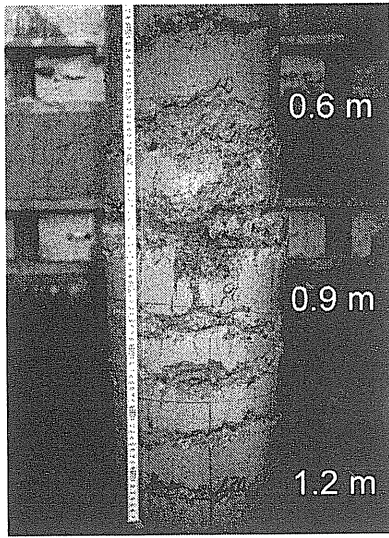


Photo 4.2 The final crack pattern of the tested pile in an experiment involving RC piles embedded in sand and subjected to a lateral cyclic load

bars appeared inside the cover concrete during the experiment, a large degree of spalling of cover concrete did not occur. The surrounding soil may have prevented cover concrete from spalling off, redistributing the damage.

(2) Enhancement of the toughness of cover concrete by a lateral confined effect due to earth pressures

Mander et al.³⁵⁾ has proposed a compression stress-strain relation for confined concrete. **Figure 4.3** is a schematic diagram of their model. The equations below represent concrete response.

$$\sigma_c = \frac{f_{cc}xr}{r - 1 + x}, \quad (4.2)$$

$$x = \frac{\varepsilon_c}{\varepsilon_{cc}}, \quad (4.3)$$

$$r = \frac{E_c}{E_c - E_{sec}}, \quad (4.4)$$

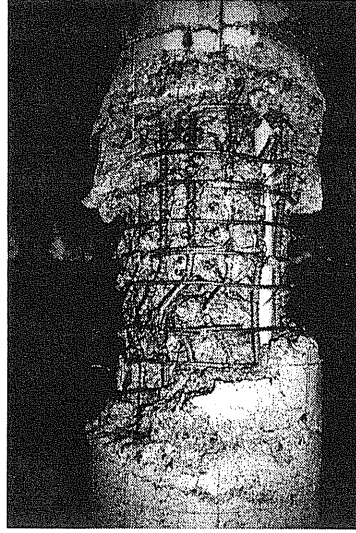


Photo 4.3 Final damage to reinforcing bars in the test pile specimen

where f_{cc} and ε_{cc} are the peak compressive stress and corresponding strain of confined concrete. f_{cc} and ε_{cc} can be obtained by

$$f_{cc} = f_{c0} \left(-1.254 + 2.254 \sqrt{1 + 7.94 \frac{f_l}{f_{c0}}} - 2 \frac{f_l}{f_{c0}} \right), \quad (4.5)$$

$$\varepsilon_{cc} = \varepsilon_{c0} \left[1 + 5 \left(\frac{f_{cc}}{f_{c0}} - 1 \right) \right], \quad (4.6)$$

where σ_{c0} and ε_{c0} are the stress and strain of unconfined concrete at the peak compressive strength, respectively, and f_l is the effective lateral confining stress. In Eq. (4.4), E_c is the initial Young's modulus of concrete and E_{sec} is the secant Young's modulus of concrete at the peak stress. These values can be estimated by

$$E_c = 5000 \sqrt{f'_{c0}} \quad (\text{MN/m}^2), \quad (4.7)$$

$$E_{sec} = f'_{cc} / \varepsilon_{cc}, \quad (4.8)$$

respectively. Thereby, we can estimate cover concrete strength while considering the lateral confining effect given by earth pressures by including the earth pressure in f'_l of Eq. (4.5).

However, the earth pressure changes cyclically along with the relative displacement between the foundation and the far-field soil. Nevertheless, we assume that an earth pressure

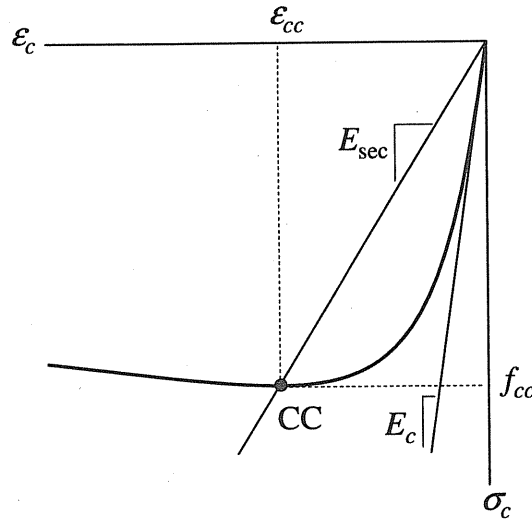


Figure 4.3 Schematic diagram of stress-strain model for concrete in compression proposed by Mander et al.

at rest is averagely effective as the lateral pile confinement during pile motion, because the changing earth pressure is thought of as consisting of the earth pressure at rest plus a variable component. Since the largest damage occurred as a depth of approximately 1 m below ground level, the earth pressure at rest corresponding to that depth is taken as the confining pressure f'_l of cover concrete, regardless of depth, for the sake of simplicity. The coefficient of earth pressure at rest, K_0 , is estimated by a typical empirical equation

$$K_0 = 1 - \sin \phi', \quad (4.9)$$

with $\phi' = 39^\circ$. A dotted curve in **Figure 4.4** shows the stress-strain behavior of cover concrete calculated by Eqs. (4.2) to (4.8) considering the lateral confined effect due to the earth pressure. For comparison, the stress-strain relations for the cover concrete and the core concrete obtained as in the simulation of Chapter 3 are shown by dashed lines. The comparison suggests that an improvement in the toughness of cover concrete can be expected if the lateral confined effect for cover concrete is effective.

The compressive collapse strain of cover concrete must be known in order to define the initial state of reinforcement swelling. Although Mander et al. associate the compressive collapse strain with the initiation of the fracture of hoop, this is not applicable to the case

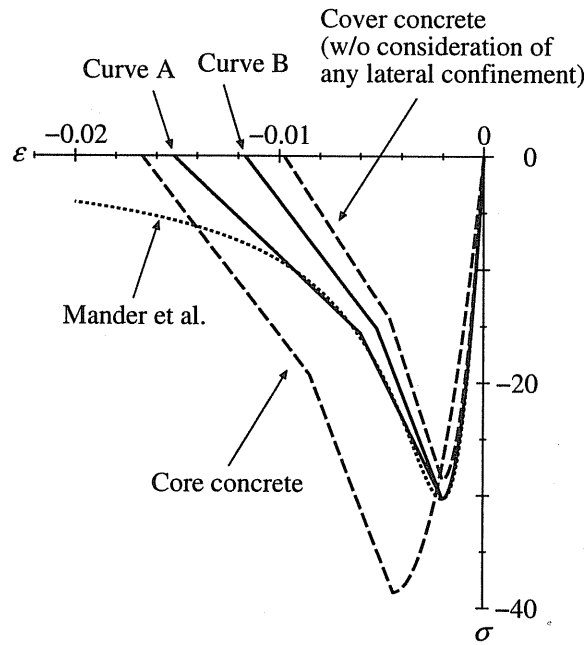


Figure 4.4 Comparison of stress-strain relationship for cover concrete among the numerical cases

in which the concrete confinement is induced by earth pressures. In order to resolve this problem, we utilize points that correspond to strains that are three and five times larger than that at the peak stress on the original stress-strain curve as the first and second control points, respectively, such that an alternative stress-strain curve is fit well up to approximately 1% in strain level in the compressive softening region. Curve A in **Figure 4.4** is the determined stress-strain curve. The first softening curve runs from the point of the peak stress to the first control point, and the second softening curve runs from the first control point to $p = 0$ via the second control point.

Another case is assumed in order to examine simply the possibility of the confining effect due to earth pressures. The first and second descending gradients are given as the gradients of core concrete. A stress-strain curve is derived as the curve indicated as Curve B in **Figure 4.4**.

On the other hand, the confining effect by earth pressures is not considered in the mechanical properties of core concrete. Further, for simplicity, we use the model used in

previous sections in the stress-strain relation of core concrete. These assumptions are based on the fact that the confining pressure due to earth pressures is much smaller than that due to hoop reinforcement in a shallow zone of, for example, approximately 1 m in depth and the confining effect to core concrete by hoops is expected even after the cover concrete is damaged. However, an inconsistency exists with respect to the method used to determine the softening behavior of compressive concrete, and this assumption can make the collapse strain of cover concrete larger than that of core concrete in a zone very deep below ground level. Hence, a more rigorous modeling is required, but this assumption can be regarded reasonable in the first trial in order to consider the confining effect to cover concrete by earth pressures.

The following may be another method for taking the lateral confining effect by earth pressures into account when using the model proposed by Hoshikuma et al. As has also been mentioned by Mander et al., if hoops uniformly develop their yield strength, the lateral confining stress to core concrete can be expressed as

$$f_l' = \frac{1}{2}k_e\rho_s f_{yh}, \quad (4.10)$$

where k_e is the confinement effectiveness coefficient, which can be estimated by

$$k_e = \left(1 - \frac{s}{2d_s}\right)^2, \quad (4.11)$$

where s is the hoop spacing, d_s is the diameter of the hoops, and ρ_s is the volume ratio of the lateral confinement bars and is defined by Eq. (2.3) and f_{yh} is the hoop yield strength. The area of longitudinal reinforcement section is assumed herein to be sufficiently small compared to that of the core concrete section. Thus, earth pressures may be converted to an equivalent value of ρ_s using Eqs. (4.10) and (4.11) and a stress-strain relation of cover concrete considering lateral confinement due to the earth pressures can be obtained by Eqs. (2.1), (2.2), and (2.4), although this is not examined in this study.

(3) Numerical calculations

a) Computational cases and modeling

A two-dimensional finite element analysis is employed within the framework of infinitesimal displacement theory. The pile and surrounding soil are modeled as a beam on nonlinear Winkler type distributed springs.

The hysteretic mechanism of p - y response for a nonlinear Winkler type interaction spring is the model proposed by Shirato et al.³⁶⁾ Shirato et al. have shown that the stress-dilatancy behavior of soil causes the loading pattern dependency of the load transfer hysteretic behavior between soil and pile; that is, the tendency for the soil resistances to mobilize at the same displacement level differs, depending on the loading patterns, based on the comparison of the experimental results of single piles embedded in sand and subjected to fully reversed or one-sided cyclic loading and the examination of element test results of sand subjected to cyclic passive and active deformation. Then, they proposed a load transfer hysteretic mechanism for Winkler type interaction springs for random loading histories. The hysteretic mechanism can be classified as a peak-oriented model, and is further extended in order to consider the loading pattern dependency of the hysteresis. However, the special mechanism for the effect of cyclic loading on one side (loading pattern dependency) is disregarded in calculations herein, since the load condition to the test pile was completely reversed cyclic loading. In short, only the basic hysteretic mechanism is used and the deterioration rule is ignored.

The hysteretic mechanism in response to fully reversed loading is called the basic hysteretic mechanism. It is a kind of peak oriented mechanism as shown in **Figure 4.5**. A p - y curve runs toward a point corresponding to the ever-largest displacement $|y|_{\max}$.

We herein use a bi-linear curve of elastic and perfectly plastic type as the skeleton curve, as shown in **Figure 4.5**. The ultimate value p_U of p is provided by

$$p_U(z) = \alpha_p K_p \gamma z, \quad (4.12)$$

where K_p is the passive earth pressure coefficient, γ is a soil weight per unit weight volume, z is a depth, and α_p is a modification factor for representing three-dimensional effects in the

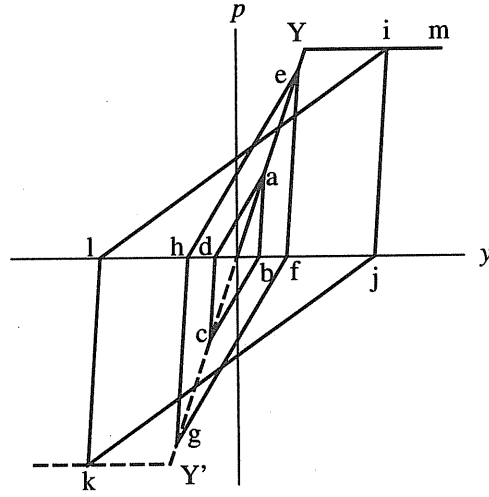


Figure 4.5 Basic Hysteretic Mechanism.

effective interaction zone in subsoil layers. We express a subgrade reaction coefficient as follows:

$$k_H(z) = \alpha_k k_0(z), \quad (4.13)$$

$$k_0(z) = (E_0(z)/B_0) \times (B/B_0)^n, \quad (4.14)$$

where E_0 is a small strain deformation coefficient of soil, k_0 is a reference subgrade reaction coefficient, B is a width of foundation and is a diameter in the case of piles, and B_0 is a reference width and 0.3 m is adopted in the Japanese Specifications. n is a constant value which represents the foundation width dependency of subgrade reaction coefficient. In this paper, $n = -3/4$ is assumed based on the Japanese Specifications.²⁾ α_k is a constant to fit the reference subgrade reaction coefficient with a bi-linear curve. It is assumed that the value of α_k is independent of depth. The values of α_k and α_p can be determined based on load tests on single piles subjected to monotonic loading or fully reversed cyclic loading.

In addition, an unloading path bounding for $p = 0$ from the last reversal point is regarded as a straight line with an unloading gradient of $k_{H\text{unl}}$. $k_{H\text{unl}}$ is provided by

$$k_{H\text{unl}} = \beta k_0. \quad (4.15)$$

We further assume $\beta = 1$ in Eq. (4.15) since the unloading gradient is closely related with an

unloading gradient of soil which has the same order of small strain levels as used to define k_0 .

In **Figure 4.5**, a point corresponding to the ever-largest displacement on the skeleton curve is referred to as a global control point C_i , where the suffixes i and j take 1 or 2, hereafter. A pair of global control points, C_1 and C_2 , exists on the positive and negative sides, respectively, and they are symmetric about the origin. For example, during a path $e \rightarrow f \rightarrow g$ in **Figure 4.5**, $|y|_{\max}$ is the absolute value of a displacement amplitude of point 'e', $|y_e|$, and then the other global control point is defined to be point 'g', i.e. $y_g = -|y|_{\max} = -y_e$. We further refer to the curves connecting the global control points, C_i - C_j , as the external curves. For simplicity of computer coding, internal curves which move inside the external curves always bound for the last reversal point on the external curve or the outmost internal curve.

The setting of material parameters for soils is the same in the calculation performed by Shirato et al.³⁶⁾ Distributed lateral Winkler springs are integrated into discrete lateral springs at nodes. The load-displacement relationship of each integrated lateral spring is based on a p - y relationship of the depth of a node, and is typically estimated by simply multiplying the width and length of the element by p of the p - y relationship. We derived the value of E_0 at each depth by $E_0 = 2(1 + \nu)G_0$, where G_0 and ν are a small strain shear modulus and a Poisson ratio of soil, respectively. G_0 at each depth was estimated by Eq. (4.1). σ'_c in Eq. (4.1) was estimated by $\sigma'_c = (1 + 2K_0)\sigma'_v(z)/3$, where $\sigma'_v(z)$ is an effective overburden stress at depth z , and the coefficient of earth pressure at rest K_0 is given by Eq. (4.9). ν was assumed to be 0.5. The internal friction angle ϕ' of the test sand deposit for estimating earth pressure coefficients is assumed to be 39° based on the element test shown in **Table 4.1**, since overburden stresses are small in the experiment. After all, $\alpha_p = 4.0$ and $\alpha_k = 0.3$ were chosen in reference to the numerical simulations of the same type pile load experiments using steel pile piles shown by Shirato et al.³⁶⁾

The pile is modeled using fiber elements. The element length is three times as long as the spacing between adjacent hoops, $3s$ ($= 120$ mm), where s is the hoop spacing, based on the result of plastic buckling analyses of longitudinal reinforcement described in Chapter

3(2). The constitutive relation for longitudinal reinforcement bars proposed in Chapter 3 is used to include the effect of reinforcement swelling. The hysteretic mechanism of the stress-strain relation for concrete is the same model used in Chapters 2 and 3. The modified Muguruma model proposed by Ristic et al.²⁴⁾ is also used.

In the constitutive relation of longitudinal reinforcement bars, Young's modulus and tensile strength are based on the element test result, and the values of other parameters for the Menegotto-Pinto model are the same as those used in Chapter 3. In addition, plastic buckling analysis is carried out in the same manner as in the previous chapter in order to determine a monotonic compressive stress-strain curve of the swelled longitudinal reinforcement bar, where the hoop resistance is modeled by elastic and perfectly-plastic springs. The parameters necessary for the constitutive relation of concrete are given in the same manner as determined in the simulations of experiments in Chapter 3, based on the material element test. The accuracy of these numerical settings for this pile specimen has been already demonstrated, for example, as shown in Figures 3.14 and 3.16.

Three different settings for the parameters of constitutive relations of cover concrete and reinforcement bar are tested as listed in Table 4.3. Case 0 is a conventional case and does not take reinforcement swelling or lateral confinement effect to cover concrete from the earth pressure into account. Case 1 considers reinforcement swelling but does not consider the lateral confinement to cover concrete. In Cases 2 and 3, both the reinforcement swelling and the lateral confinement to cover concrete are included. The stress-strain relations of cover concrete are given by Curve A in Figure 4.4 for Case 2 and Curve B for Case 3, respectively.

b) Results and discussions

The result for Case 0 is shown in Figures 4.6 and 4.7. In Figure 4.7, ε_L and ε_{L0} are the compressive collapse strains of the core and cover concrete, respectively, and ε_y is the yield strain of the longitudinal reinforcement bars. Case 0 is the case in which the most commonly used modeling for the pile is used. The strength in the positive side and the shape of internal

Table 4.3 Numerical cases

Cases	Reinforcement Swelling	Confinement to cover concrete
0	Not considered	Not considered
1	Considered	Not considered
2	Considered	Considered (Curve A)
3	Considered	Considered (Curve B)

loops in the relationship of load versus displacement at the load point are simulated very well, although the calculated strength is somewhat larger than the experimental strength. However, the calculation cannot predict the difference in strength between positive and negative loads at peak points as was observed in the experiment. This tendency is the same in the following numerical cases, but we have not yet been able to realize a clear reason for this difference. The numerical calculation cannot reproduce the tendency of the occurrence of the rapid degradation of strength at the final displacement cycle, either, because the numerical model does not take the effect of reinforcement swelling into account.

As is shown in **Figure 4.7**, the calculated depth at which the pile sustained the largest damage is in good accordance with the experimental result and occurs at approximately 1 m. Since we have already verified that the use of fiber elements for this RC pile specimen yields precise behavior in air, as elucidated in **Figure 3.16**, we can infer that the other numerical element, namely the model used herein for the hysteretic mechanism of p - y of the Winkler type interaction spring will be also advantageous with respect to soil-pile interaction.

However, a strain at the position of an extreme reinforcement bar exceeds the compressive collapse strain of cover concrete in an early stage of the loading, whereas large spalling-off of cover concrete was not observed in the present experiment. Nevertheless, the model used for the pile is not capable of simulating the initiation of clear strength degradation as seen in the experiment at the final displacement cycle. Case 0 eventually shows that more rigorous structural models of the pile are necessary for estimating the ductile capacity

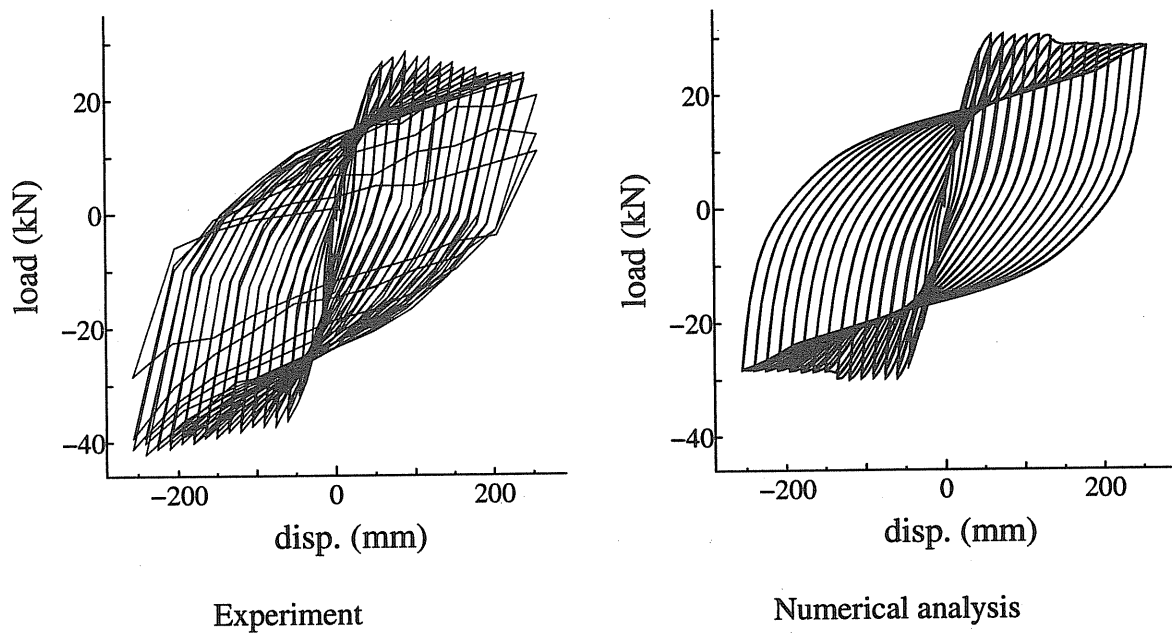


Figure 4.6 Load-displacement response at the loading point (Case 0: No consideration of reinforcement swelling and lateral confinement of the earth pressure to the cover concrete)

of the pile. The estimation of postpeak behavior given by typical models of piles does not associate the degree of damage to the piles with the ultimate ductility capacity of the piles.

Numerical results in Case 1 are shown in **Figures 4.8 and 4.9**. Case 1 further considers the reinforcement swelling in the pile model in addition to the model used in Case 0. The degradation of strength occurs in the calculation along with the reinforcement swelling after the 12 δ cycle. From this result, we realize again that the prediction of a large reduction in strength due to reinforcement swelling is quite effective for estimating the ultimate limit state in pile design. However, the predicted initiation of rapid degradation of pile strength is earlier than that observed in the experiment. This is due to the fact that the compressive damage to cover concrete is overestimated as in the result of Case 0.

Figures 4.10 and 4.11 shows numerical results for Case 2. Case 2 further considers the lateral confinement to the cover concrete by earth pressures using the constitutive relation proposed by Mander et al. The numerical results agree very closely with the experimental results in the load-displacement relationship. On average, the positive peak loads can be

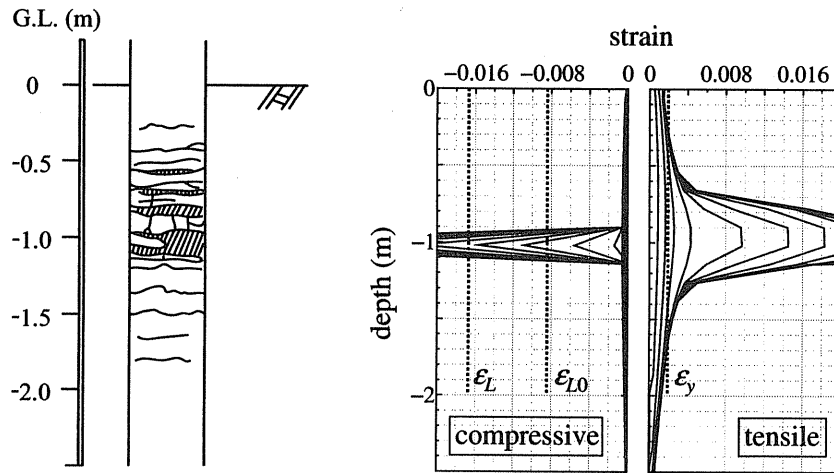


Figure 4.7 Distributions of longitudinal strain in extreme reinforcement bars at the positive peak displacements of the first cycles (Case 0)

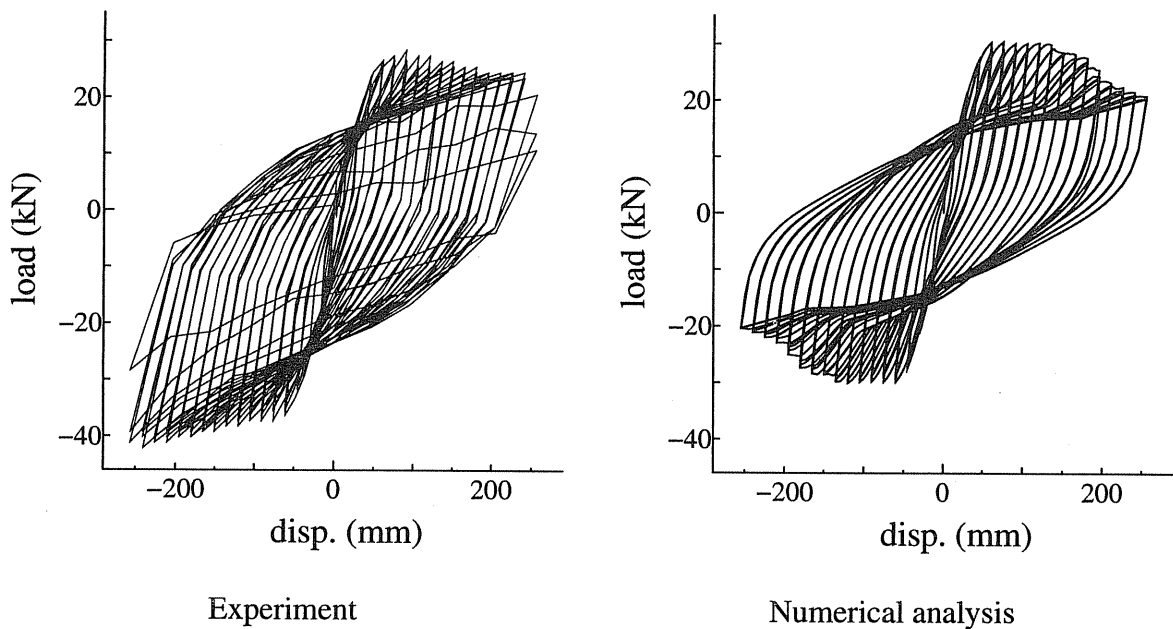


Figure 4.8 Load-displacement response at the loading point (Case 1: Including consideration of reinforcement swelling and without consideration of lateral confinement to the cover concrete by earth pressures)

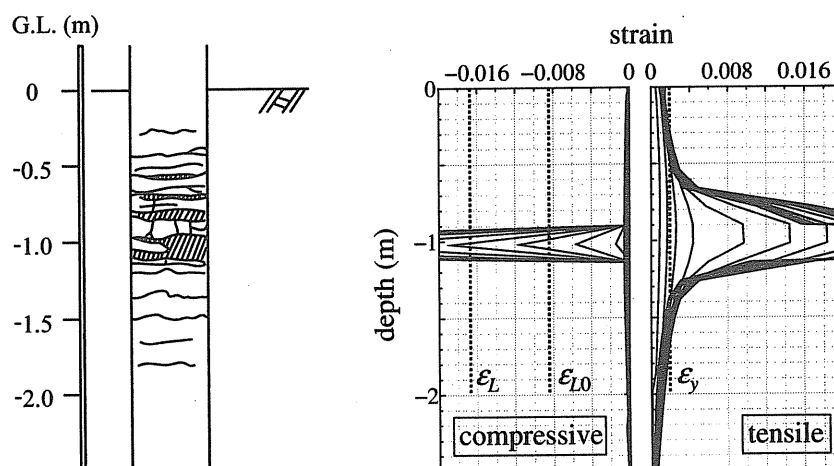


Figure 4.9 Distributions of longitudinal strain in extreme reinforcement bars at the positive peak displacements of the first cycles (Case 1)

predicted to be approximately 20% larger than the experimental results. The initiation of the large reduction in the lateral strength of the pile in the calculation is developed at the final displacement cycle, and this is identical to the experimental observation.

The calculated damage distribution for the state after the experiment is in good agreement with the experimental result. For example, the plasticized region of an extreme longitudinal reinforcement bar widens to a greater degree than that calculated in Case 1 and coincides with the region in which wide cracks were observed in the experiment. Therefore, the difference in the stress-strain relation of cover concrete may give a precise damage distribution.

While fracture of the reinforcement was observed in the experiment in the final displacement cycle, the calculated maximum tensile strain is 7.9% in the same displacement cycle and this is much less than a typical ultimate strain of reinforcement bars derived in element tests. As a result, it is inferred that the fracture of reinforcement bars in the experiment could be caused by repeated cyclic plastic deformation. However, from an engineering standpoint, the number of plastic deformations during an earthquake is expected to be much smaller than that in this experiment, and the difference in reinforcement fracture is not likely to lead to a serious problem in the seismic performance assessment. As a result, we believe

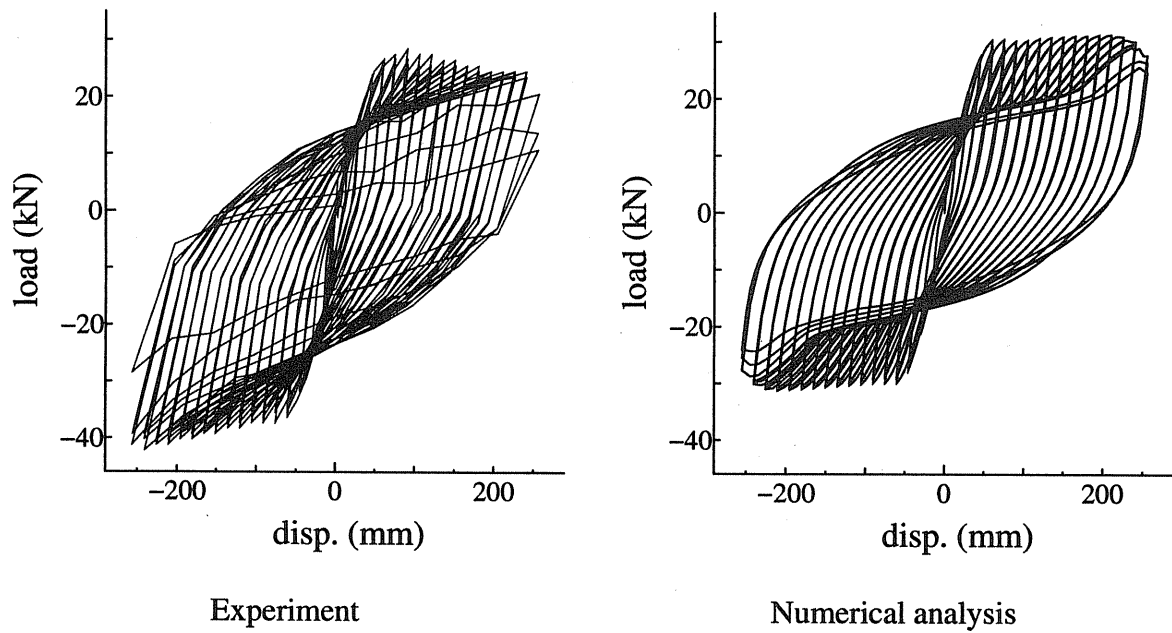


Figure 4.10 Load-displacement response at the loading point (Case 2: Including consideration of reinforcement swelling and lateral confinement to cover concrete by earth pressures)

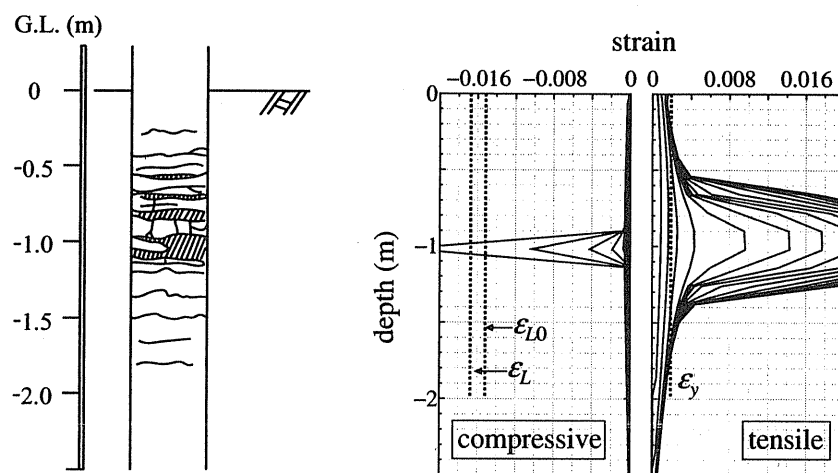


Figure 4.11 Distributions of longitudinal strain in extreme reinforcement bars at the positive peak displacements of the first cycles (Case 2)

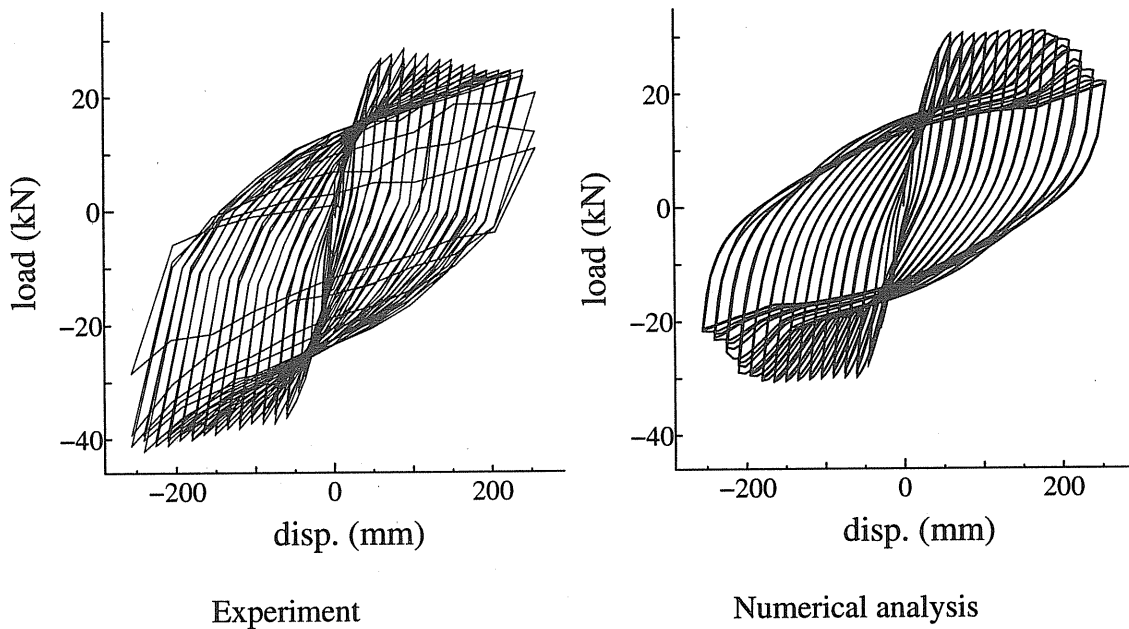


Figure 4.12 Load-displacement response at the loading point (Case 3: with considerations of both reinforcement swelling and lateral confinement to cover concrete by earth pressures)

that the modeling used in Case 2 can accurately predict the ductility behavior of RC piles.

Presented in **Figures 4.12** and **4.13** are the numerical results for Case 3. The predicted behavior and damage state are between the calculated results for Cases 1 and 2. This corresponds to the position of the stress-strain curve of cover concrete in **Figure 4.4**. From a comparison among Cases 1 through 3, it is indicated that the lateral confinement effect to cover concrete due to earth pressures can be increased toughness of RC piles.

Piles can sustain bending damage deep under the ground due to seismically induced ground deformation during a large earthquake as will be examined in the following chapter. However, based on the numerical results presented in this chapter, it is likely that piles have high ductility capacities owing to the confining effect on cover concrete by earth pressures. Pile foundations undergo higher earth pressures than those in the experiment examined in this chapter. Consequently, the confined effect can give an additional safety margin in the assessment of seismic performance of piles deep under the ground. Based on the numerical examination above, the strength of the cover concrete may be ascertained considering the

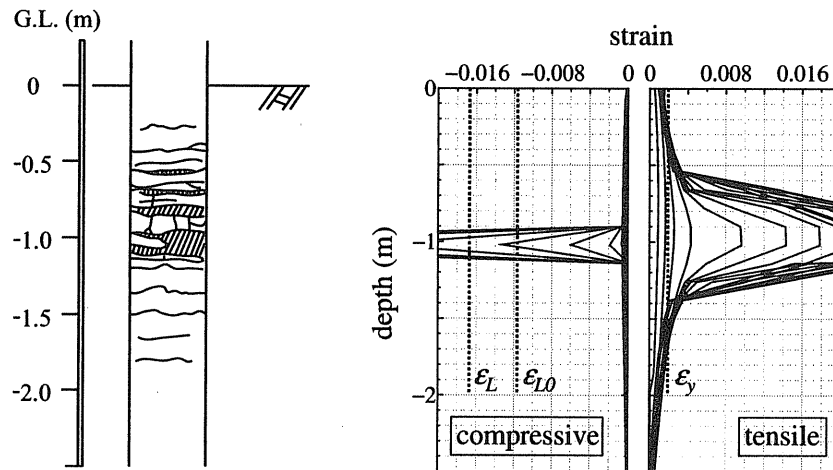


Figure 4.13 Distributions of longitudinal strain in extreme reinforcement bars at the positive peak displacements of the first cycles (Case 3)

confined effect due to earth pressures. We can at least expect the resistance carried by cover concrete in pile design, even though the lateral confinement effect is not taken into account in the estimation of the resistance of cover concrete.

(4) Concluding remarks

In this chapter, an experiment involving an RC pile embedded in loose sand and subjected to a reversed cyclic load was simulated. The comparison of pile behavior between the experiment and the simulation shows that the models used, namely, the hysteretic mechanism of soil-pile interaction springs and fiber elements installing a constitutive relation of reinforcement bar which can account for the swelling phenomenon, do a good job of predicting the nonlinear behavior of the pile in sand with an excellent accuracy up to postpeak region. The most appropriate calculated result was given when the lateral confined effect to cover concrete due to earth pressures is taken into account in the constitutive relation of fibers corresponding to cover concrete. Based on the numerical results, the toughness of RC piles embedded under ground can be improved compared to that in air, as the depth from the ground level increases. This advantage should be utilized to achieve more reasonable

designs. Especially, as described earlier, cover concrete of cast-in-place RC piles occupies a greater cross sectional area compared to other RC members, such as piers, and the constitutive model for cover concrete is important in numerical simulations for cast-in-place RC piles. We can at least expect the resistance carried by cover concrete in pile design, even though the lateral confinement effect is not taken into account in the estimation of the resistance of cover concrete. It is also quite effective to consider the lateral confined effect to cover concrete due to earth pressures especially for assessing the ductility and strength capacities of piles sustaining kinematic interaction deep under ground caused by the ground vibration during an earthquake.

In this chapter, since we examined pile behavior in the case subjected to a reversed cyclic loading, we disregard the special rule accounting for the loading pattern dependency of the hysteretic mechanism of the Winkler type interaction spring. However, in cases when treatment of the $P-\Delta$ effect is needed, for example, in the design of an extended RC pile shaft, we should consider the deterioration rule to involve the effect of cyclic loading on one side with the soil resistance. A hysteretic mechanism of $p-y$ considering the deterioration rule is important in estimating the $P-\Delta$ effect in the assessment of seismic performance of such structures.

5 . The Relationship in a Pile Foundation between Damage to Pile Members due to Kinematic Interaction and Stability of a Foundation as a Structural System

(1) Introduction

In the previous chapters, we showed numerical modeling methods that perform a numerical assessment of the seismic performance of a pile foundation in a practical seismic design. However, our ultimate goal is to apply the methods which we developed to the practical design of deep foundations. In the near future, limit states will be able to be defined based on the numerical results of a foundation during the collapse process, and then the foundation can be verified. Future study is still required to reach the ultimate goal, but, in this chapter, we attempt to show an example of the future verification process of a pile foundation.

In the example shown, special emphasis is placed on the effect of the bending damage in the pile caused by seismic-motion-induced ground displacement (i.e., the kinematic interaction), which is located in a position deeper than the area which is greatly affected by the inertial force from the superstructure (i.e., the inertial interaction). It has been clarified through existing earthquake damage case histories and recent studies^{14), 15), 21), 22)} that damage to piles can occur around the boundaries of the subsoil layers and the boundaries between

liquefied and non-liquefied subsoil layers, where the rigidities of the soils vary, due to the so-called kinematic interaction between piles and the ground during severe earthquakes, as well around the top of piles, when piles are embedded in deep soft soils. However, the effects of the damage to piles due to the kinematic interaction on the stability of grouped pile foundations have not been studied.

Simulating dynamic soil-pile interactions under any conditions through experimental investigations is difficult. On the other hand, numerical approaches are of great help because they can be applied to pile foundations of any configurations and any ground conditions. Hence it would be quite useful to numerically observe the postpeak behavior of cast-in-place RC piles in pile foundation systems in order to establish both the performance-based design and limit state design of pile foundations. Accordingly, we will investigate the relationship between seismically induced pile damage due to the kinematic interaction and the dynamic response of the foundation as a system via several dynamic analyses of a cast-in-place RC pile foundation.

The degree of the effect of pile damage on the response of the foundation as a system can be different, depending on the depth of the damage. It is inferred that damage to the piles at a deep position, where the piles do not contribute much to the support of the inertial force, is unlikely to influence the response of the foundation as a system, but this assumption has not been proved. The example shown in this chapter deals with this type of situation.

(2) Numerical settings

The object of the examination is a highway bridge pier with a grouped pile foundation. **Figure 5.1** shows the geometry of the pier and the foundation. This particular foundation was originally introduced in the literature³⁷⁾ as an example of ductility design for highway bridge foundations based on the 1996 Specifications for Highway Bridges in Japan.^{1),2)} The foundation comprises 4×3 cast-in-place RC piles, the dimensions of each are 1000 mm in diameter (D) and 23 meters in length. The minimum distance between the center of

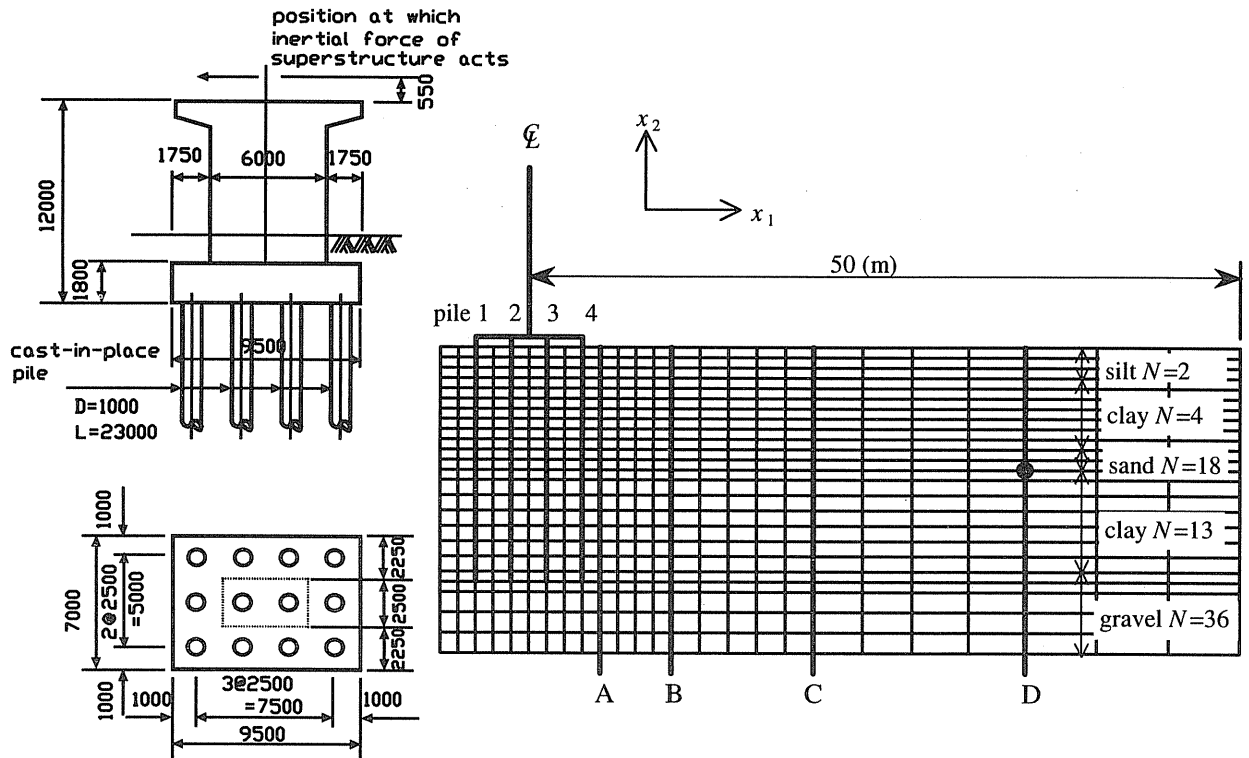


Figure 5.1 Simulated pier and finite element mesh

neighboring piles is $2.5D$. They are embedded in soft ground classified as Class III as defined in the Specifications. The assumed ground level for seismic design is GL -4 m in compliance with the Specifications, this corresponds to the bottom of the pile cap.

The finite element mesh to be analyzed is also shown in Figure 5.1. The mesh is actually symmetrical about the center line which is identical to the centerline of the pier. In carrying out the simulations, the finite element is formulated within the framework of infinitesimal displacement theory. The ground conditions used here are simplified for ease of setting the numerical parameters of the ground, as listed in Table 5.1, in comparison with the original settings in the literature.³⁷⁾ The characteristic period of this simplified ground is evaluated as 0.6 s by the Specifications. Namely, this ground is considered to be relatively hard for Class III ground, because this period classifies the ground as being near the boundary between Class II and III. Class II ground has a shorter characteristic period than Class III ground.

A seismic analysis is performed in order to determine the response in the direction

Table 5.1 Ground profiles

Layer	Depth (m)	Soil	SPT- N	V_{s0} (m/s)	Density ρ (t/m ³)
1	4 ~ 7	silt	2	130	1.6
2	7 ~ 14	clay	4	160	1.6
3	14 ~ 16	sand	18	210	1.7
4	16 ~ 26	clay	13	240	1.7
5	26 ~ 34	gravel	36	260	2.0

perpendicular to the bridge axis. We execute the three types of simulations summarized in **Table 5.2**. Case 1 models the details of the referenced pier-foundation system as strictly as possible. Case 2 is a modification of Case 1 as follows: the pier material is made elastic; the amount of reinforcement in the piles is greatly reduced, so the foundation is placed under greater stress than that in Case 1; and the acceleration of the input earthquake wave at the base is 2.2 times larger than that used in Case 1, with this value set by trial and error such that postpeak behavior of bending capacity would be observed deep in the piles. Case 3 is another modification of Case 1, in which further modifications are added to those in Case 2. The differences from Case 2 are as follows: the upper limit of bearing capacity at the bottom of the pile is released to make the compressive axial force on the piles large; and the acceleration of the input earthquake is made to be twice that used in Case 1, for the same reason as in Case 2. The arrangement of the reinforcement in the piles in these three cases is listed in **Table 5.3**.

A plane strain element is employed to model the ground. Nevertheless, two-dimensional finite element analyses are unlikely to be appropriate in design practice for pile foundation. For example, general considerations of the appropriate adjustments of constitutive law and perspective depth of the ground have not been established. However, the following supports this decision for the present example:

1. The present study places emphasis on determining the characteristics in the damage

Table 5.2 Summary of computational cases

	Case 1	Case 2	Case 3
Pier	nonlinear	linear	linear
End bearing resistance of pile	nonlinear	linear	linear
Amount of reinforcement in piles	–	reduced	reduced
Amplitude of base input acceleration	×1.0	×2.2	×2.0

behavior of piles deeper under the ground surface rather than in the area where there is inertial interaction between the soil and foundation;

2. In the area deep under the ground surface, the motion and deformation of piles would be dominated by the ground vibration displacement induced by the earthquake wave, as is well known,^{15), 16), 19), 20), 38)} because piles are flexible and slender and the mass of piles is overwhelmingly small in comparison with the ground region;
3. In addition, it is considered that plane strain modeling can perform well for one-dimensional shear vibration of the ground during earthquakes.

The depth of the element in the out-of-plane direction is twice that of the pile cap in that direction, which is identical to the arrangement employed by Tateishi et al.,²⁰⁾ who reported a seismic dynamic simulation for the same pier. They used this depth based on the investigation by Tazo et al.³⁹⁾ which studied the perspective depth of the plane strain element in order to evaluate the equivalent behavior derived by a three-dimensional finite element analysis on pile-groups in elastic media. As for the boundary conditions, both edges of the modeled ground are free to move in the horizontal direction but are restrained in the vertical direction, while the motion at the bottom is fixed in both the horizontal and vertical directions.

When considering seismic ground motion as one-dimensional shear vibration, we may adopt a one-dimensional constitutive model for the nonlinearity of soil. We employ Hooke's

law as the constitutive relation for the soil, and take the nonlinear characteristics of soil into account by changing the instantaneous shear modulus according to the stress history step-by-step.

The instantaneous shear modulus is given by the Ramberg-Osgood model²³⁾ in terms of the stress history of σ_{12} , where the Cartesian coordinate system x_1 - x_2 is set as indicated in **Figure 5.1**, and where we assume that soil material is isotropic. The skeleton curve of σ_{12} - ε_{12} of the Ramberg-Osgood model is given by

$$\varepsilon_{12} = \frac{\sigma_{12}}{2G_0} \left(1 + \alpha \left| \frac{\sigma_{12}}{\sigma_r} \right|^\beta \right), \quad (5.1)$$

where G_0 is the initial tangent shear modulus, σ_r is the shear stress σ_{12} when shear strain ε_{12} reaches a special strain called the reference strain in this model. It should be noted that the reference strain is required in order to set the nonlinear shape of the skeleton curve. α and β , which are also required in order to specify the skeleton curve nonlinearity, are usually given by the following equation:²³⁾

$$\alpha = \left| \frac{2}{\varepsilon_r G_0} \right|^\beta, \quad \text{and} \quad \beta = \frac{2\pi h_{\max}}{2 - \pi h_{\max}}, \quad (5.2)$$

where $h_{\max}$ is the maximum equivalent damping constant of a soil which is observed at very high strain regions. The values of the instantaneous tangent shear modulus G_t on the skeleton curve are set by obtaining the tangent of Eq. (5.1). The internal loop curves are described by the Masing rule.

When applying solid elements to soils, we should use an appropriate constitutive relationship of the soil which can reproduce the behavior of the soil subjected to cyclic passive-active (in other words, compression-extension) deformation especially for the near field soils greatly affected by the inertial interaction, in addition to cyclic shear deformation especially for the far field. However, the present analysis does not model the cyclic compression-extension deformation mode, and, for example, is not able to represent the stress-dilatancy behavior as considered in the Winkler type interaction springs proposed by Shirato et al.³⁶⁾ In addition, since the Ramberg-Osgood model does not include the strength parameters of soils, the stress level continues to increase in the elements which undergo large strains. Hence, the

resistance of the ground to the foundation is overestimated when the foundation response becomes relatively large. This situation is one of the problems in the simulation to be solved in the future. Discussions as to how to choose a nonlinear constitutive model of the soil are needed.

The initial shear modulus G_0 of the soil in each layer is determined by the phase velocity of shear waves in the corresponding layer, as listed in **Table 5.1**. Poisson ratios are set at 0.49 independently of the soil properties. Parameters for the nonlinear properties in the Ramberg-Osgood model are estimated by the usual procedure²³⁾ based on the maximum damping constant and the reference strain. We assume that the maximum damping constants of sand and clay are 0.3 and 0.2, respectively. It should be noted that these values have been considered to be general in the past. However, Murono⁴⁰⁾ and our research team⁴¹⁾ at the Public Works Research Institute have carried out parametric numerical studies with the Ramberg-Osgood model based on comparisons between the results of their large-scale shaking table tests using sand and numerical results, and both have pointed out that it is better to employ a somewhat larger value with respect to sand, such as 0.35 to 0.4. The reference strain ε_r is set to strain ε_{12} when G/G_0 reaches 0.5, where G is the secant shear modulus. The reference strain in each layer is estimated on the basis of existing experimental studies by Iwasaki et al.,^{42),43)} where the reference strain is determined from the soil profile and the confining pressure. We evaluate the confining pressure at the depth of the gravity center in each element, with the static earth pressure coefficient fixed at 0.5.

The heads of the piles are rigidly connected to the pile cap. Since plane strain analysis is to be carried out, the three piles in one row are gathered together into one beam.

The same plastic buckling analysis as in Chapter 3 is carried out for the longitudinal reinforcement for each sectioned reinforcement arrangement, and the hoops are modeled by elastic and perfectly-plastic springs. A series of numerical tests determined that the length of the reinforcing bar which shows the most drastic reduction in load after buckling is three times the spacing between two adjacent hoops for all the reinforcement arrangements in the examined foundations. However, because of the requirement for connecting the fiber el-

elements of piles to the plane strain elements of soils, as will be described later, it becomes much easier to implement the mesh if the length of a fiber element is equated to the thickness of a plane strain element for the soil. We therefore carry out the same series of numerical plastic buckling analyses again under the condition that the length of the longitudinal reinforcing bars is equal to the depth of the plane strain elements for the ground; i.e. 500 mm.

As a typical numerical result, a comparison of the load-displacement relations at the loading point is shown in **Figure 5.2** for Arrangements 'c' and 'd' in **Table 5.3** with analyzed lengths $L = 3s = 450$ mm and 500 mm. Discrepancies due to differences in analyzed length are barely discernible in the results for both arrangements. This same tendency is also seen in the numerical results for Arrangements 'a' and 'b'. In the case of Arrangement 'd', the two curves in **Figure 5.2** are too close to be distinguished. We therefore determine that the length of the fiber elements will be 500 mm and that the corresponding post-buckling behavior will be treated as swelling behavior.

Other parameters necessary for the piles are given in the same manner as for the simulations of experiments described in Chapter 3, where the design compressive strength of the concrete is 24 N/mm^2 . There is no consideration of the confining effect to cover concrete by the earth pressure.

We introduced joint spring elements between piles and ground at nodes having identical coordinates. These joint spring elements resist forces in the horizontal and vertical directions. The horizontal joint elements are linear springs with large rigidity so that the relative horizontal displacements and velocities between the piles and the ground are small enough to be ignored.

The vertical joint springs are classified into two types; one type is arranged around the circumference of the pile, while the other is fitted to the pile base. The former joint springs represent vertical shaft resistance between pile and soil, and these are a series of discrete elastic-perfect plastic springs. Each is an integrated component of a distributed spring with a constitutive law of elastic and perfectly-rigid-plastic type with strength equal to the maximum shaft resistance. The subgrade reaction coefficients of the distributed springs are

Table 5.3 Arrangement of reinforcement in pile

GL (m)	Case 1	Cases 2 and 3
-4.0 ~ -6.0	Arrangement 'a'	Arrangement 'c'
-6.0 ~ -12.0	Arrangement 'b'	Arrangement 'd'
-12.0 ~ -18.5	Arrangement 'c'	Arrangement 'd'
-18.5 ~ -27.0	Arrangement 'd'	Arrangement 'd'

Arrangement	Longitudinal	Hoop
a	SD295 22-D32	SD295 D19 ctc 150
b	SD295 22-D32	SD295 D16 ctc 150
c	SD295 11-D32	SD295 D16 ctc 150
d	SD295 11-D22	SD295 D16 ctc 150

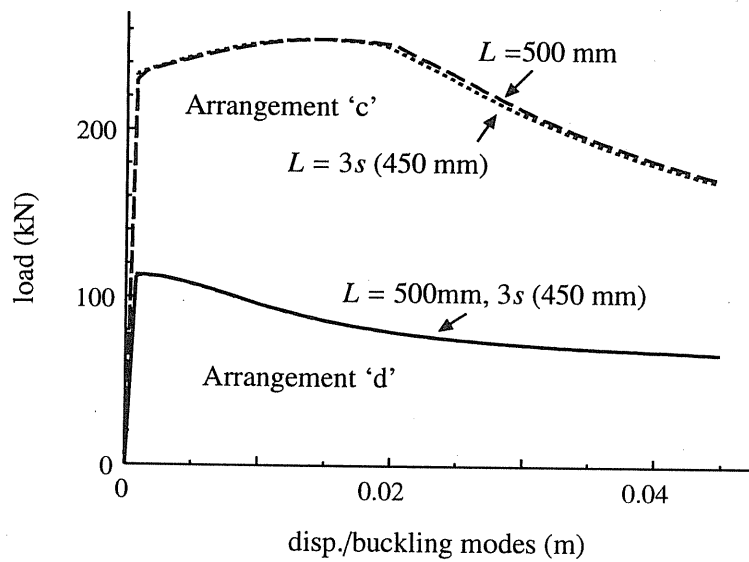


Figure 5.2 Numerical analysis of plastic buckling of longitudinal reinforcement bar (Arrangements 'c' and 'd')

evaluated using an empirical equation from the design Specifications²⁾ for caisson foundations, whereas the maximum shaft resistance is calculated from another empirical equation for cast-in-place RC piles given in the design specification. These empirical equations involve SPT- N values.

On the other hand, the vertical joint springs located at the bases of the piles are assumed to be non-linear elastic. They have a bi-linear response to compression with a maximum strength equivalent to the bearing capacity at the pile base, but they have no resistance to tension. The compressive spring constant is estimated from the vertical subgrade reaction coefficient, which is considered in the design of caisson foundations, while the maximum strength is derived from an empirical equation for the design of cast-in-place RC piles. Note that, in Case 3, the maximum value of bearing capacity at the base of pile is ignored. Furthermore, we also combine the three joint elements of three piles in the same row into a single joint element at each depth.

The upper structure system is modeled as a single lumped-mass of 8836 kN. The pier is modeled as a single lumped-mass of 4271 kN at its middle point and elastic beam elements. The entire section is considered effective, and Young's modulus of concrete is used in calculating the bending rigidities of the beam elements. However, a plastic-hinge region based on the Specifications¹⁾ is specified at the bottom of the pier only in Case 1. This plastic-hinge is modeled as a nonlinear rotational spring, the resistance of which is given by a somewhat modified nonlinear relation of the Takeda model.⁴⁴⁾ In this modification, the crack point is neglected; the yield bending capacity of the pier is given according to the Specifications;¹⁾ and the tangent bending rigidity after initial yielding is set to 1/10,000 of the initial bending rigidity. The pile cap is modeled as an elastic beam element with excessive rigidity, and its mass of 2492 kN is located as a lumped mass at its gravity center.

As global damping, Rayleigh damping is assumed, and the coefficients corresponding to the first and second modes are both set to 3%. An eigenvalue analysis of the whole system gives the first and second eigen periods, where we use the initial rigidity of all the structural members and soils in the calculation. The resulting characteristic periods are 0.52 s for the

first mode and 0.38 s for the second mode. In the first mode, the shear deformation of the ground is greater, and the displacements of ground and structure are in phase. But the second mode is mainly governed by the rocking motion of the structure, and the corresponding phases are opposed to each other. We can conclude that the characteristic period of the structure T_s is shorter than that of the ground T_G . This is also supported by the fact that the period of the ground derived according to the design Specifications is 0.6 s. Note that the initial damping matrix at $t = 0$ s is always used in seismic response analyses, because we can expect that most of the damping effect in seismic response analysis arises from hysteretic damping of the structural and soil materials because of the large earthquake motion.

(3) Results of dynamic analyses

The basic input earthquake motion is the NS component of one of the most commonly used earthquake records: that taken at the Japan Meteorology Agency in Kobe during the 1995 Hyogoken-Nanbu (Kobe) earthquake. In the dynamic analysis, the magnitude of this basic input wave is multiplied by a specific value, and then the adjusted motion is input directly into the nodes of elements at the base. Since the numerical results obtained in Cases 2 and 3 are quite similar, only the results for Cases 1 and 2 are explained in detail below.

a) Dynamic behavior of ground

Figure 5.3 shows the distributions of peak displacement of the ground at different positions, where the symbols A through D indicate the observation lines labeled A through D in **Figure 5.1**. The same symbols will be used throughout. The peak values in **Figure 5.3** are the maximum absolute values. The seismic design code for railway structures¹⁸⁾ introduces empirical equations to estimate distributions of horizontal ground displacement in the depth direction under design seismic motions for bedrock as specified in the code. The corresponding commentary explains that the equations are derived through a statistical compilation of many dynamic analysis results for many ground conditions. Examination of this railway

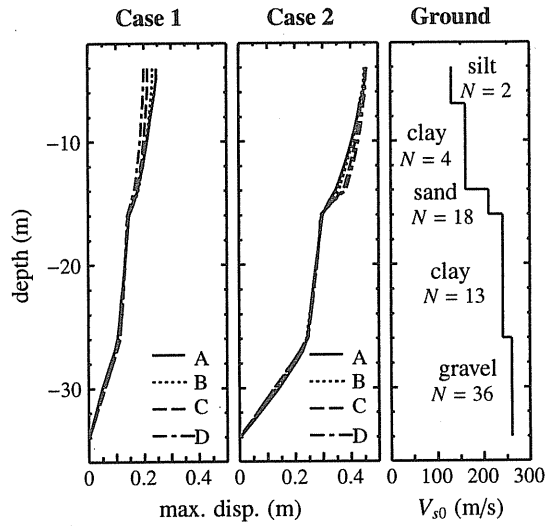


Figure 5.3 Distribution of peak ground displacement

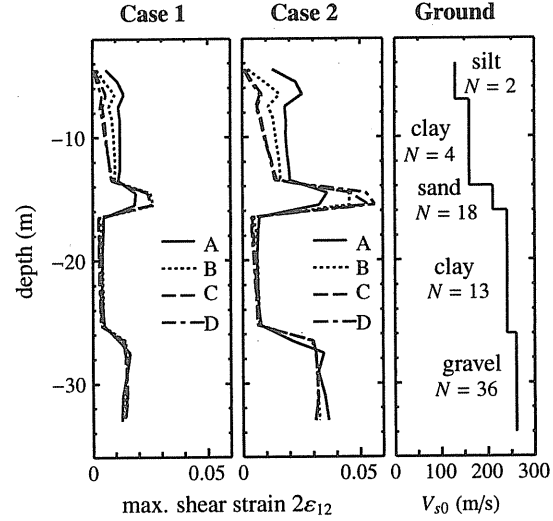


Figure 5.4 Distribution of peak shear strain in the ground

code suggests that the characteristics of L2 motion and Spectral II design seismic motion are very close to our inputs into the numerical simulations. Therefore, in order to evaluate the effect of earthquake motion on the foundation as obtained in our seismic simulations from a practical design point of view, it is very helpful to compare the horizontal displacement of the ground numerically obtained here to that calculated using the empirical equation given in the railway code. This calculation from the code turns out to be approximately 0.2 m, so the ground displacement obtained in Case 1 is of the same order as this value obtained by the usual design practice for severe earthquakes in Japan. However the displacement obtained in Case 2 is much more severe than that generally assumed in the seismic design of foundations.

Although there is a sudden increase in ground displacement across the boundary from the fourth clay layer to the third sand layer, the increment in displacement is small at the transition from the third sandy layer to the second clay layer. We can also see the same tendency in the distribution of the maximum shear strain, as is shown in **Figure 5.4**.

Several attempts have been made to predict the distribution of seismically induced horizontal displacement of the ground, as well as to evaluate the relations between ground profiles and damage locations in piles deep underground. Most treat the ratio of phase velocity V_{s0} in neighboring soil layers as a parameter, because transmission of the shear wave

causes vibration of the ground. We here follow the same procedure using phase velocities. Ratios of initial shear wave velocities in our settings are calculated to be 0.88 between the fourth and third layers and 0.76 between the third and second layers, where the ratios are obtained by dividing the velocity of the upper layer by that of the lower layer. It is usually considered that the smaller this ratio, the larger the increment in shear strain, because a small value indicates low shear rigidity at the upper layer. However, the results for the increment in shear strain obtained by our finite element analyses show the opposite characteristics. It is inferred that this disagreement with the usual result is caused by differences in the non-linear characteristics of clay and sand. Consequently, we need to take into account not only the ratio of phase velocities in two adjacent soil strata but also the soil properties when we investigate the distribution of seismic ground displacement under level 2 earthquakes or the relation between soil strata and the distribution of damage to piles deep underground.

We can compare distributions of horizontal displacement and shear strain of the ground at different horizontal positions in the region between lines A and D in **Figures 5.3 and 5.4**. The differences in response are large near the ground surface, but are very small deep underground below GL -15 m. This indicates that the response of the piles near the ground surface is influenced by the inertia of the superstructure as well as the free ground vibration, and this affects the ground motions. Deeper below the surface, on the other hand, the displacement of the piles is mainly caused by seismically induced ground displacement, and does not reflect on the vibration of the ground.

In addition, shown in **Figure 5.5** are distributions of absolute horizontal acceleration in the ground. In both Cases 1 and 2, the input motions are rapidly reduced in the gravel layer, are maintained at a constant level in the clay subsoil layer above the gravel layer and are finally amplified in the subsoil layers near the ground surface again. From the perspectives of the different horizontal positions at lines A through D, the distributions of the absolute horizontal acceleration correspond with each other deep underground, but we can see a clear difference above GL -10 m. In particular, the values on line A are much larger than those on lines C and D. This also indicates that this difference is due to the response of the foundation

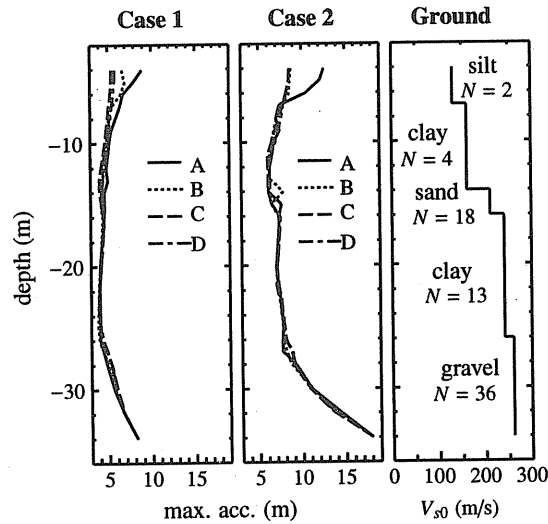


Figure 5.5 Distribution of peak ground acceleration

caused by the inertia of the superstructure.

b) Response and damage to foundation in Case 1

Figure 5.6 compares a number of time histories: horizontal displacement at the ground surface on line D indicated in Figure 5.1; horizontal displacement and acceleration of the superstructure at the position where the inertial force acts; the horizontal acceleration of the gravity center of pile cap; the bending moment at the bottom of the pier; and the reaction force at the base of pile No. 4. Here, the lateral force coefficient is defined by the horizontal acceleration divided by -9.8 m/s^2 , and the reaction force at the base of pile No. 4 is taken as positive in compression. Note that the reaction force at the base of pile No. 4 is chosen as a parameter because it changes according to the overturning moment at the top of the foundation and thus has a strong relationship with the inertial force of the superstructure. From this figure, clearly, the motions of superstructure, ground surface, and pile cap are in the same phase. The timing of the maximum of each parameter is 4.50 s for ground surface displacement, 5.04 s for the lateral force coefficient of the superstructure, 5.04 s for bending moment of the pier at the bottom, and 4.90 s for reaction force acting on the pile base, respectively. Namely, the maxima of the superstructure inertial force and the displacement of the ground surface are not achieved simultaneously, but, when the lateral force coefficient

of the superstructure reaches its maximum, the magnitude of ground surface displacement is almost equal to its own maximum value. Consequently, we can regard the maximum inertial force from the superstructure and the maximum ground displacement as occurring simultaneously and in phase.

Next, we discuss the distribution of damage to the pier and foundation during the earthquake in order to understand the general characteristics of damage distribution in foundations and factors affecting such damage. The bottom of the pier behaves elastically except for short periods when it becomes plastic at maximum bending moment. The response ductility factor of the pier is 1.34, as estimated from the horizontal displacement of the center of mass of the superstructure where the inertial force is acting. This indicates that we can assume elastic behavior of the pier on evaluation of the performance of the foundation.

In all piles, tension cracks are observed in the cover concrete over the entire surface. On the other hand, compressive failure never occurs, and hence no swelling of the reinforcement appears.

As for the longitudinal reinforcement of the piles, **Figure 5.7** depicts the distribution of tensile yielding. The gray lines indicate the reinforcement positions, and the black lines represent yielded portions. This tensile yielding occurs not only at the pile tops but also near the boundary of the third sand layer and the fourth clay layer at GL -16 m. The longitudinal range of this yield region is approximately one pile diameter above and below the boundary. Moreover, yielding is observed in a narrow band at a position directly above the ground base layer. Namely, deep underground, the reinforcement yields near the boundary between different layers, where the vertical distribution of horizontal displacement and shear strain of the ground abruptly changes.

Similar yielding develops near the boundary between the second and third layers of soil strata at GL -14 m. But the distribution of horizontal displacement and shear strain of the ground does not exhibit a sudden change, as is obvious in **Figures 5.3** and **5.4**. We can thus conclude that the primary reason for yielding is not only the effect of ground excitation but also the abrupt changes in the rigidity and capacity of the piles, since there is a reduction

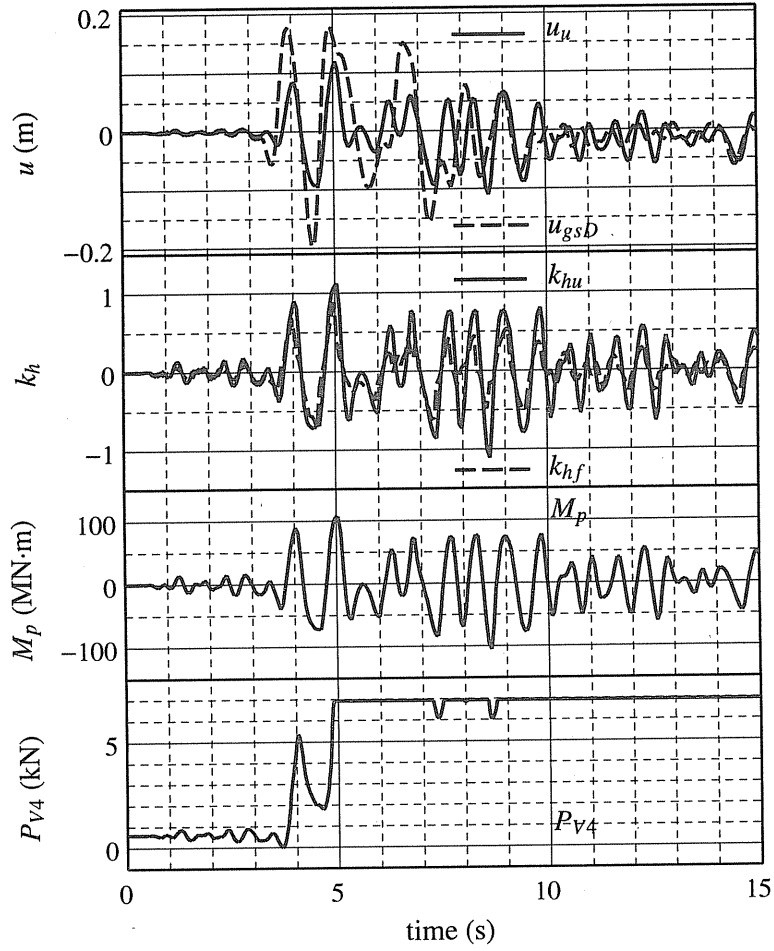


Figure 5.6 Time histories of: horizontal displacement at the ground surface on line D u_{gsD} ; displacement and lateral force coefficient of the upper structure u_u and k_{hu} ; lateral force coefficient of the pile cap k_{hf} ; bending moment of the pier at the bottom M_p ; and reaction force at the base of pile No. 4 P_{v4} (Case 1)

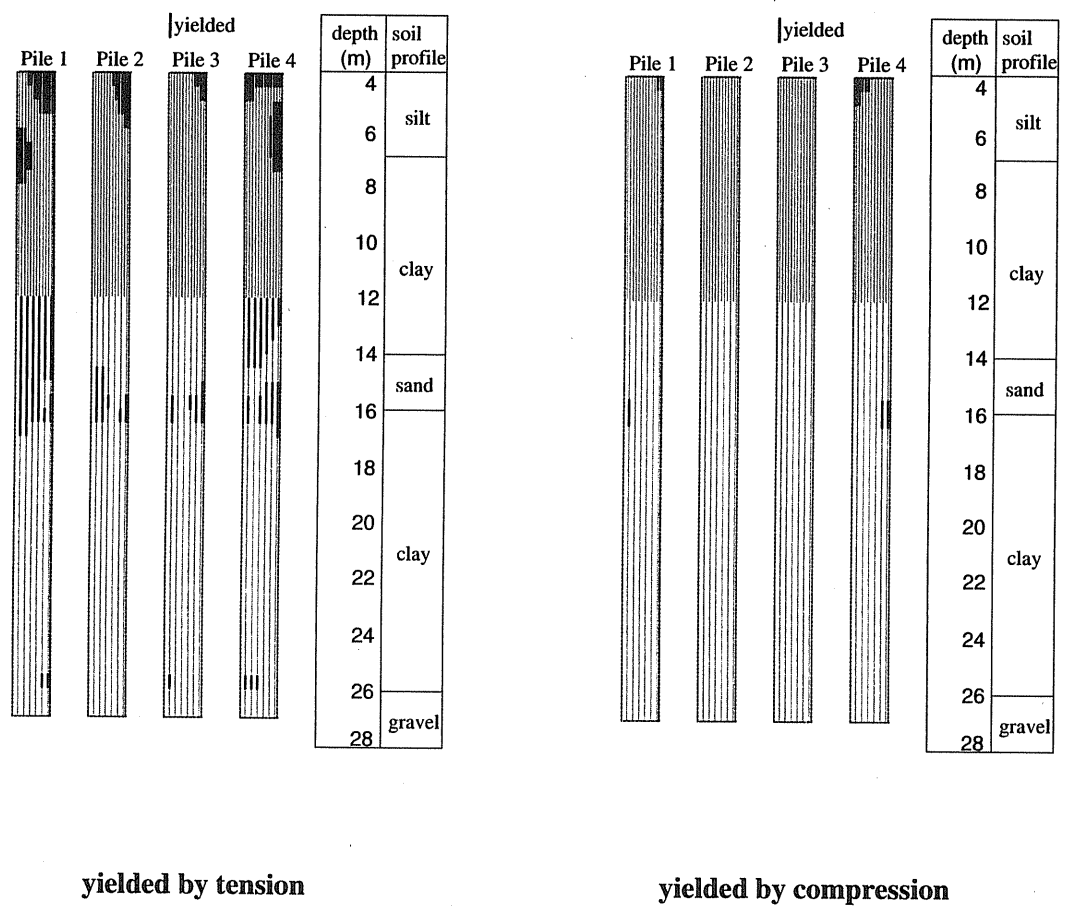


Figure 5.7 Yield zone of longitudinal reinforcing bars in piles (Case 1)

in reinforcement at GL -12 m.

The yield zone at the top of the piles differs depending on the position of the pile. Furthermore, even at the top of the piles, we notice a non-symmetrical distribution of the yield zone about the longitudinal centerline of the pile. In contrast, at approximately GL -16 m, there is almost no difference in yield zone among piles, whether near the center or at the end. These results indicate that the dominant factors governing yield differ depending on depth; yielding at the top of piles is triggered by the inertial force of the superstructure, whereas that near GL -16 m is affected by the seismic ground displacement.

Furthermore, compressive yielding occurs in the No. 1 and 4 piles around their circumference at GL -16 m as well as in the inner parts at the top, as illustrated in **Figure 5.7**. Remember that the bending moment in piles is dependent not only on the bending deformation, but also on the axial force, as the experimental results in Chapter 3 (3) b) show. Thus, the reason compressive yielding becoming possible only in the reinforcement of No. 1 and 4 piles is thought to be that the axial forces on surrounding piles become larger than that in the central piles.

Figure 5.8 compares the distributions of absolute values of maximum bending moment achieved at each position during the earthquake (solid curves) and of absolute bending moments (dashed curves) when the compressive reaction force at the base of pile No. 4 reaches the maximum; i.e. at $t = 4.9$ s. The former indicates that all piles experience large bending moments at the top as well as near the boundary between the third sand layer and the fourth clay layer.

The large bending moments deep below the surface are observed at the location at which sudden changes in horizontal displacement and ground shear strain appear, as shown in **Figures 5.3** and **5.4**, and these are the locations at which the pile reinforcement yields. These results imply the possibility of establishing a simple prediction method for pile damage locations without carrying out a sophisticated seismic response analysis of the system consisting of pile foundation and foundation ground. That is, by simply referring to a database of dynamic analysis of foundations under several ground conditions when subject to level 2

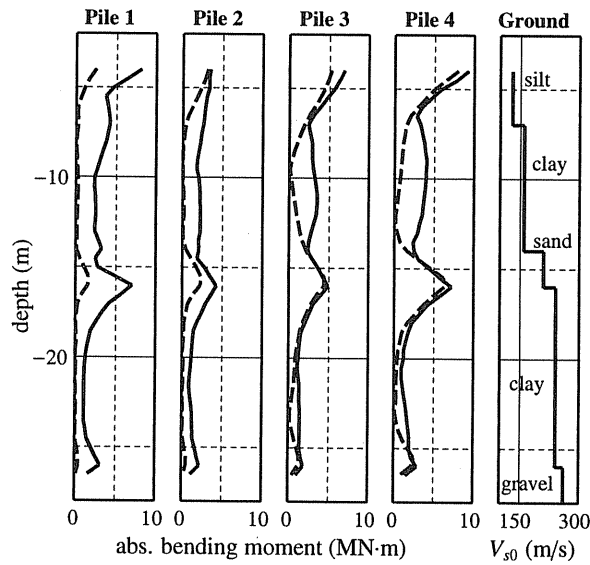


Figure 5.8 Distribution of bending moment in each pile (Case 1); solid lines are the absolute maximum values, and dashed lines are the values when the compressive reaction at the base of the No. 4 pile reaches the maximum ($t = 4.9$ s)

earthquakes, we may be able to determine the positions of possible damage from the phase velocity profile of the soil strata and the properties of the soil layers; e.g. sandy or clay.

On the other hand, the dashed curves in **Figure 5.8**, representing the distribution of pile bending moment when pile No. 4 reacts to the maximum compressive reaction force, clearly indicate that there arises almost no matching bending moment with the solid curves in the No. 1 and 2 piles. Incidentally, **Figure 5.9** shows the distributions of axial force in the piles at that same instant, revealing that almost no axial force appears in these piles, either. The time histories of horizontal ground displacement at GL -16 m on the line D, which corresponds to the solid circle in **Figure 5.1**, and the sectional forces and curvature of pile No. 4 at GL -16.25 m are shown in **Figure 5.10**. In these time histories, the bending moment clearly has different amplitudes in the positive and negative directions depending on the axial force, while the curvature of the same pile and the horizontal ground displacement oscillate with almost the same magnitude in both directions. This results from the fact that the bending moment of the piles is greatly affected by the axial force. As a result, we can take advantage of employing a fiber element; i.e. the element can automatically take into

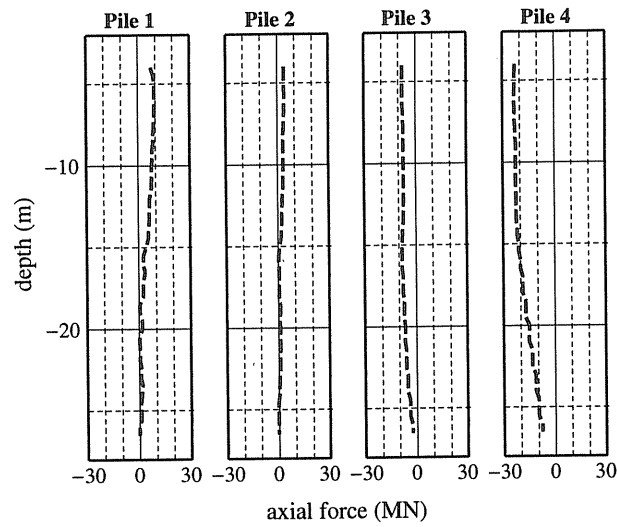


Figure 5.9 Axial force distributions on piles when the reaction force at the base of the No. 4 pile reaches a maximum, $t = 4.9$ s (Case 1), where tension is positive

account the influence of varying axial force on the mobilized sectional forces of piles.

c) Response and damage to foundation for Cases 2 and 3

Figure 5.11 shows the state of damage in the Case 2 simulation. Gray lines indicate the reinforcement. Here, larger numbers of reinforcement bars, compared to Case 1, are plasticized over the whole pile body, because in Case 2 the amount of reinforcement is reduced. Furthermore, at locations lower on the pile the damage is much more serious than at the top, especially near the boundary between the third and fourth soil layers, where some damage is also observed in Case 1. Near this boundary, it is also apparent that swelling of the reinforcing bars accompanies compressive collapse of the cover concrete. These kinds of damage can also be observed in Case 3.

The stress-strain hysteresis curves in **Figure 5.12** are for fibers in the outmost reinforcement bar, which undergoes swelling, as well as for the cover concrete and core concrete surrounding this particular reinforcement bar. The moment-curvature hysteresis of the same element is illustrated in **Figure 5.13**. This pile experiences instantaneous swelling of the reinforcement several times during the earthquake at this location. From the results for Case 1, we can conclude that the dominant factor causing this damage is seismic ground dis-

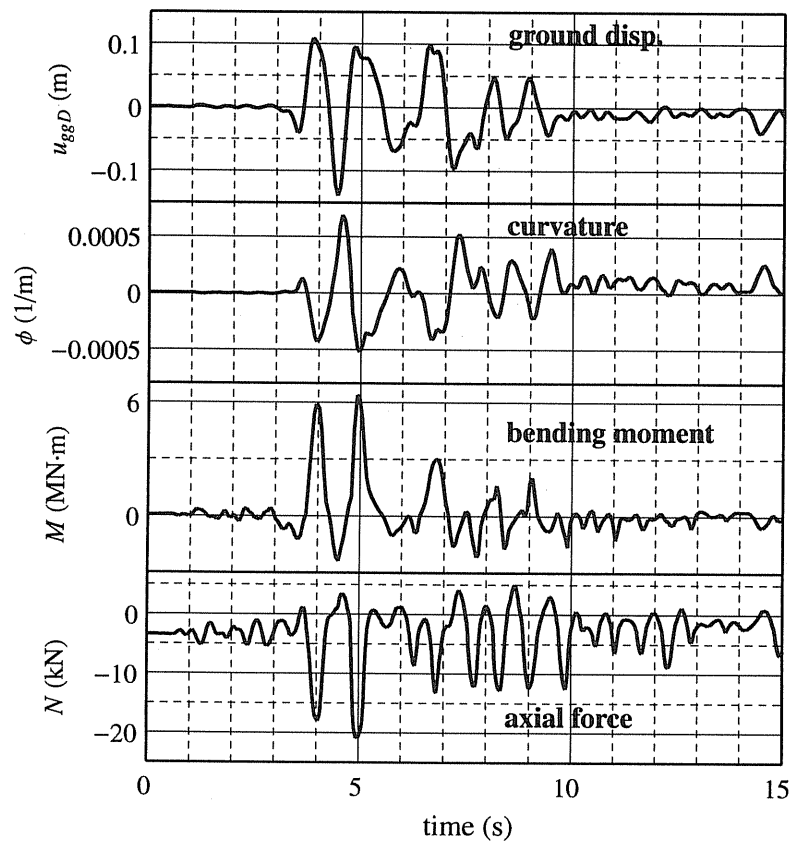


Figure 5.10 Time histories of: horizontal displacement of the ground at GL -16 m on line D u_{ggD} ; curvature ϕ ; bending moment M ; and axial force N of pile No. 4 at GL -16.25 m (Case 1)

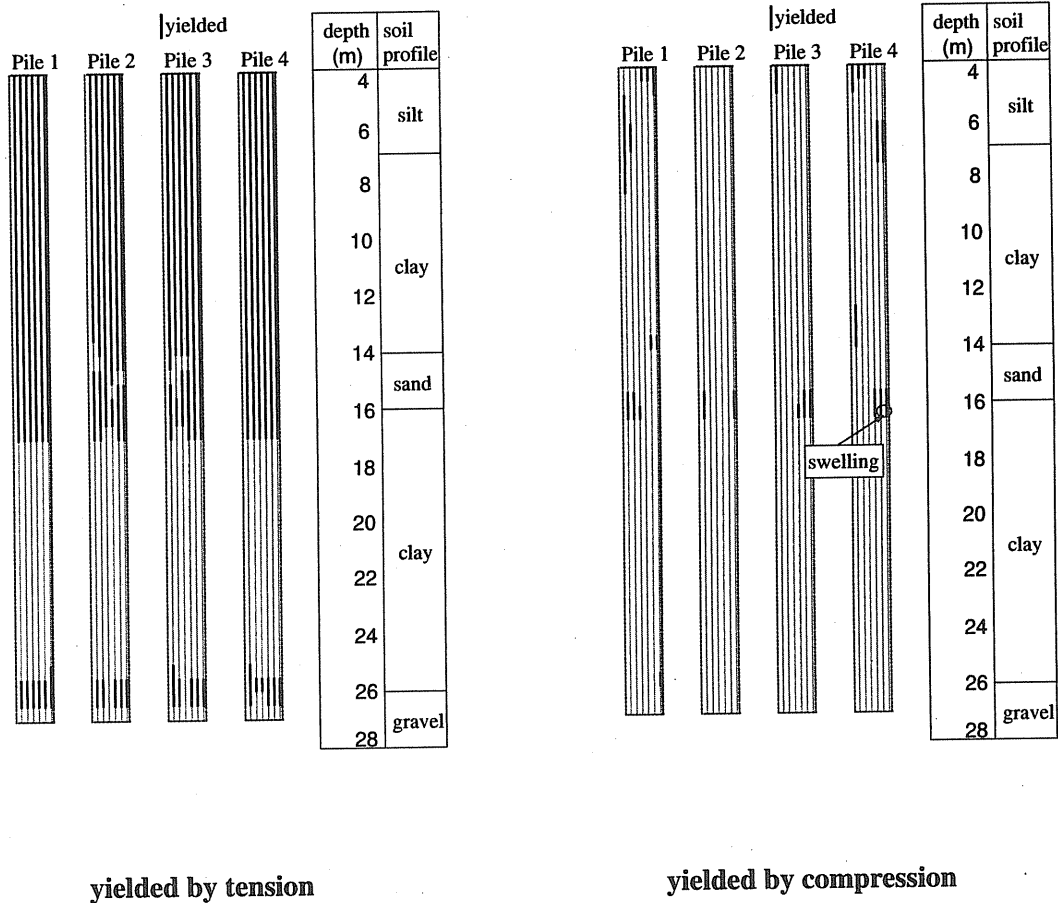


Figure 5.11 Yield zone of piles (Case 2)

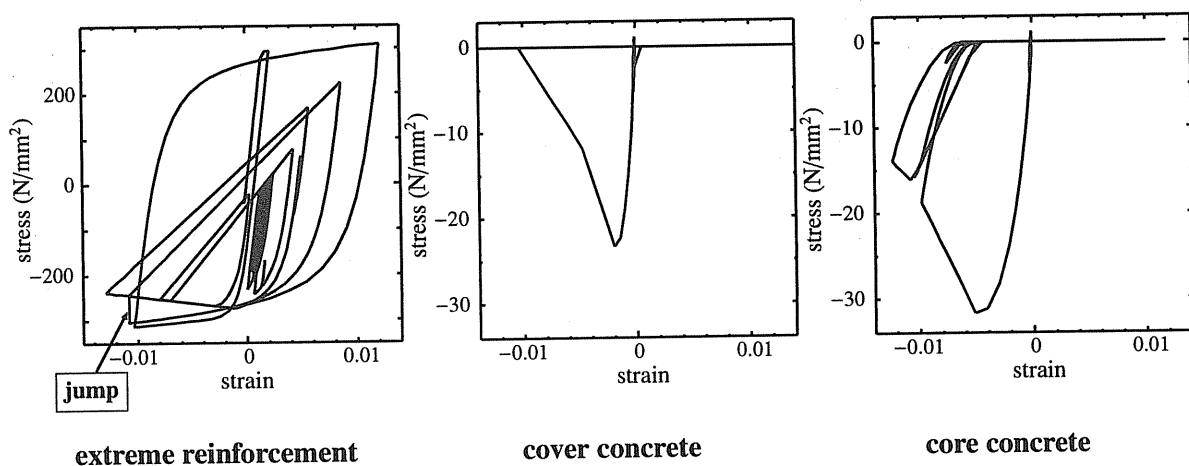


Figure 5.12 Stress-strain hysteresis curves of typical fiber element under swelling of the reinforcing bar

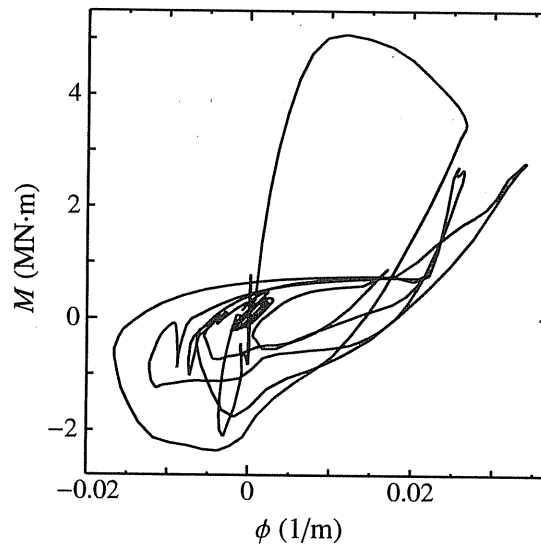


Figure 5.13 Bending moment (M -curvature ϕ hysteresis) of an element containing swelled reinforcements

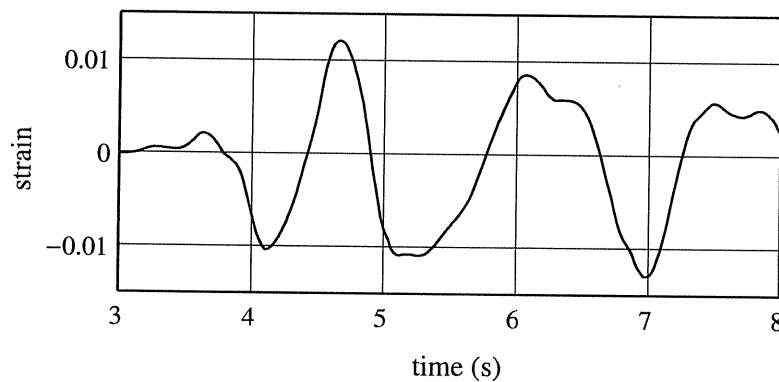


Figure 5.14 Time history of axial strain of swelled reinforcement (3 to 8 s.)

placement. However, it is not clear from **Figure 5.13** that swelling causes a serious loss of capacity in the pile itself. **Figure 5.12** also shows the existence of something like a snap-through jump due to swelling along a compressive path in the stress-strain hysteresis of the longitudinal reinforcement. In order to find the cause of this behavior, we examine the time history of strain at that reinforcement. **Figure 5.14** shows such a time history from three seconds after the initial state for a period of five seconds. As was assumed in Chapter 3, swelling becomes possible only after a reinforcing bar that has already experienced large strain accompanied with failure of the cover concrete is subjected to compressive re-loading

following unloading. In this case, the strain of the longitudinal reinforcement exceeds the compressive collapse strain of the cover concrete $\varepsilon_L = 0.01044$ when the strain suddenly increases at approximately five seconds. Thereafter, tiny oscillations of strain are perceived. We can conclude that the cause of the jump mentioned above is a short period of consecutive unloading and reloading once the strain of the reinforcement exceeds the failure strain of the cover concrete. Thus, the proposed model does not predict a smooth transfer of stress when a pile undergoes such a loading history.

Unfortunately, since experimental data for deformation histories of RC members under the loading paths described above are not available at present, it is not known whether such a jump can actually occur in the actual stress-strain hysteresis of the reinforcement along such loading paths in RC members. Looking at a study by Sakai and Kawashima,⁴⁵⁾ we consider that the appearance of minute unloading-loading oscillations in the fibers is one of the general numerical characteristics of dynamic analysis of RC members using fiber elements. However, as will be explained later, the possibility exists that such a jump in the behavior of the reinforcement may lead to difficulty or instability in dynamic analysis. Thus, we may need to further improve the hysteresis rule for swelled reinforcement bars. Moreover, a substantial discussion of the feasibility of fiber elements themselves in the dynamic analysis of RC members may be necessary.

In order to estimate the effect of serious pile damage deep below the surface on the dynamic behavior of the structural system as a whole, we begin by comparing the dynamic response of the superstructure in all numerical cases in **Figure 5.15**. In addition, for Case 2, the horizontal response of the superstructure, the ground surface on line D, and the pile cap are compared in **Figure 5.16**.

As **Figure 5.15** shows, the periods of the Case 2 and 3 responses are longer than that of Case 1 after the maximum response, which occurs at approximately five seconds. **Figure 5.16** shows that the phase difference between superstructure and ground surface in Case 2 is large, especially after $t = 8$ s. These results arise because of plasticization of the piles near the top connection, which is the most critical location as regards supporting the horizontal

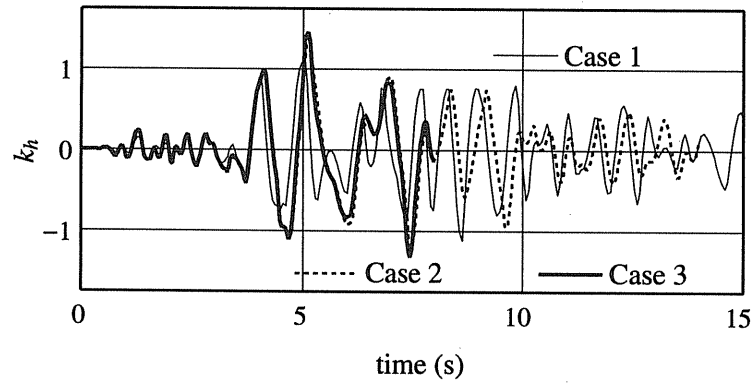


Figure 5.15 Comparison of lateral force coefficients obtained in different cases (Cases 1, 2, and 3)

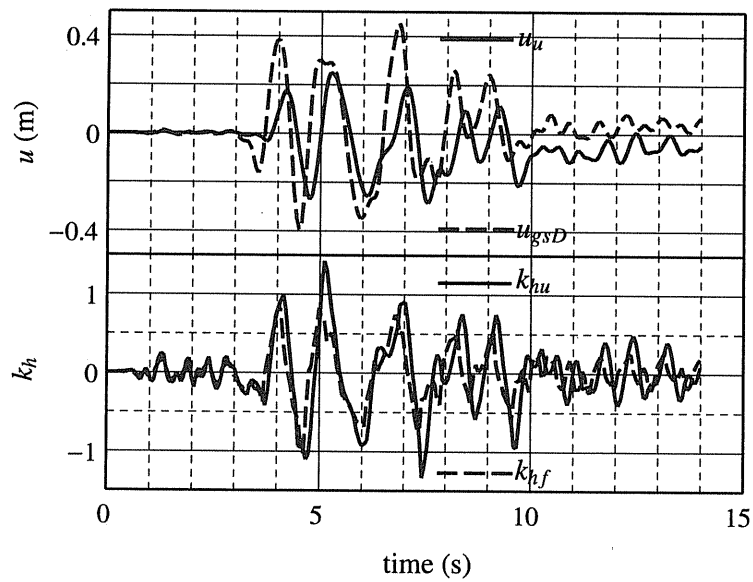


Figure 5.16 Time histories of: horizontal displacement of the ground surface on line D, u_{gsD} ; horizontal displacement of the superstructure, u_u ; lateral force coefficient of the superstructure and pile cap, k_{hu} and k_{hf} (Case 2)

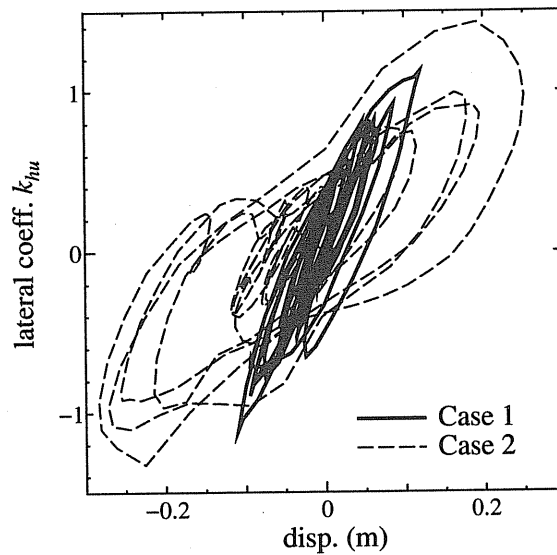


Figure 5.17 Hysteresis loops of responding lateral force coefficient versus responding horizontal displacement (Case 1 and 2)

forces imposed by the superstructure, and this makes the global period of the entire structure T_s long.

Figure 5.17 compares loops of lateral force coefficient versus horizontal displacement of the superstructure between Cases 1 and 2. Regardless of the development of local strength loss deep under the surface in Case 2, the foundation system retains enough restoring strength for stability. As a result, no phenomenon such as progressive tilting in a particular direction is observed. There are thought to be two reasons for this behavior. One is that the surrounding ground can support the horizontal inertial force of the superstructure even after the piles lose strength. The second is that, since after swelling the damaged portion is forced in the opposite direction by the seismic ground motion, it does not have a chance to accumulate in only one direction, so the set of pile and soil maintains an apparent restoring force. Therefore, displacement does not progress in a particular direction.

In the design of highway bridges, we should assess the serviceability and repairability of the structures after an earthquake, as well as the safety thereof. For example, in case histories of the 1995 Hyogo-ken Nanbu earthquake (Kobe earthquake), piers whose residual rotation angles exceed $1/60$ rad were replaced with newly reconstructed piers,¹⁾ since re-use

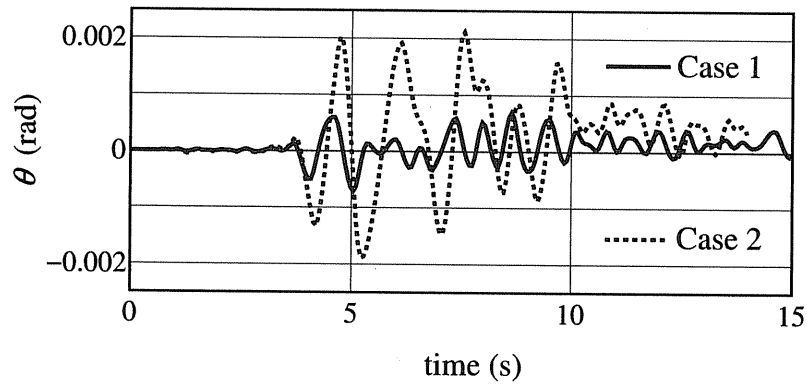


Figure 5.18 Comparison of rotation angles obtained in Cases 1 and 2

was difficult from the perspective of serviceability. As for the estimation of the residual rotation angle of pile foundations, we need to consider the residual deformation of pile members accompanying material damage, because grouped pile foundations are a kind of frame structure. As has been shown in the numerical examples of the simulation of an experiment of the type-2 pile specimen, the numerical model of RC members proposed in Chapter 3 can predict the hysteresis of the expansion and contraction of a cast-in-place RC pile due to both the action of varying axial force and the accumulation of damage to that pile. **Figure 5.18** shows the rotation hysteresees of the pile cap at its top in Cases 1 and 2. The numerical prediction of the rotation and displacement of a foundation is a comprehensive problem due to models of foundation members and soils, in other words, damage to piles and uplift and settlement behavior of piles (i.e. vertical interaction springs between soil and pile) and lateral soil-pile interaction (i.e. lateral interaction springs). The hysteresees shown in **Figure 5.18** do not include the influence of damage to the connection of the piles to the pile cap, but the proposed model predicts the contribution of pile damage to the residual rotation and displacement of foundation systems.

In **Figure 5.15**, the time histories are broken off at 14 s in Case 2 and at 8 s in Case 3, respectively, because the numerical calculations do not converge at these time steps. The numerical problem of convergence error arises, possibly as a result of the snap-through jump in the stress-strain hysteresis curve immediately after the initiation of swelling, as explained in **Figure 5.12**. The magnitudes of both acceleration and displacement are so small here

that we cannot consider the structural instability occurs at these timings. We also suspect that lack of consideration of the damping effect in the spring at the base of the pile may be another reason for termination of these calculations, judging from the fact that the response in Case 3 with a linear spring cuts off much earlier than in the Case 2.

(4) Discussion of seismic design of pile foundations including the effect of ground motion and Concluding Remarks

Next, on the basis of the numerical results in the previous section, we discuss a method of seismic design that will prevent severe damage to piles which may otherwise be induced deep underground by the ground excitation under level 2 earthquake motions.

Our numerical simulations demonstrate that pile foundations designed according to the current code have adequate resistance to the effects of ground displacement, and that bending damage to piles deep below the surface does not lead to excessive horizontal displacement to the degree that unseating of the girders will occur. In addition, we can expect higher toughness (i.e. higher ductility capacity) of piles deep under the ground than estimated herein, because the predetermined strength and ductility capacities in the present numerical examination are assumed for structural performance in air, although there is actually a confining effect to the cover concrete by the earth pressure based on the results in the previous chapter. As far as we know, past reports of earthquake damage show that, even if liquefaction occurs, the collapse or serious displacement of a pile foundation that is properly designed to withstand inertial forces is not possible unless the ground experiences a large horizontal residual displacement in one particular direction.

Therefore, we can conclude that the seismic design of foundations of highway bridges requires piles to be primarily designed to resist inertial forces imposed by the superstructures. If we cannot neglect the effect of seismic ground displacement resulting in damage to piles,

it is better to carry out the numerical estimation of deformation of the piles subjected to seismic ground vibration, and to give necessary ductility to the piles which may yield. If, in practice, design is to be carried out without any sophisticated numerical analysis which is able to take into account the effects of seismic excitation of the ground, we should specify certain structural details that will improve the toughness of the piles beyond that given by the primary design results.

For example, we recommend that reduction of the section or amount of reinforcement should be avoided for a length equal to twice the pile diameter centering on soil strata boundaries, and that some additional lateral reinforcement must be specified in these regions. These kinds of design modifications can be recommended because, based on the seismic response analysis in the previous section, we can predict to some extent the region of damage in advance based on standard investigations of the ground; even when reinforcement swelling occurs deep below the surface, the displacement of the pile itself and pile foundation system will not increase abruptly in one particular direction as long as the damaged areas remains able to transfer axial and shear resultant forces.

Since this formulation is within the framework of the infinitesimal displacement theory and material nonlinearity of soil is taken into account only by the Ramberg-Osgood model, the predicted horizontal bearing capacity of the ground may be overestimated. In addition, analysis of plane strain may cause the loss of some three-dimensional effects. Furthermore, we have not discussed what effect the input seismic motion characteristics might have on the numerical results. Hence, further studies of these problems and effects are required in future. In particular, a study of how to set the ultimate state of pile foundations that undergo damage deep below the ground as a result of seismic ground excitation should be carried out by numerical dynamic examinations of foundation responses when the damage is much greater than obtained in this study.

We have to consider carefully foundations located close to embankments or revetments of rivers, canals, and the sea, for example. The foundations and the ground in such situations will undergo biased loading arising from eccentric earth pressures. If a pile sustains large

damage, the drawing back of damaged piles by the ground vibration no longer be expected in these situations, and so the displacement of the pile could accumulate in one direction. In addition, the horizontal ground reaction intensity from the surrounding soil tends to decrease when the surrounding soil undergoes one-directional cyclic loading by the pile movement, rather than reversed type cyclic loading.

Another crucial situation is when lateral movement of the revetment neighboring a foundation is caused by the lateral spreading of the ground due to liquefaction. The horizontal ground reaction to the foundation will be lost at the revetment side. Even when ground lateral spreading displacement after an earthquake occurs, there is a possibility that the piles have already experienced damage deep under ground before starting the lateral spreading, due to the ground vibration displacement during the earthquake. Tazoe et al.⁴⁶⁾ have pointed out that a structure could have undergone such damage during a past earthquake.⁴⁶⁾

6 . Conclusions

This study has proposed a new numerical model for estimating the structural limit states of cast-in-place RC pile foundations and other RC pile foundations. The proposed modeling of concrete beams using fiber elements with a sophisticated constitutive relation can account for the postpeak behavior of pile members due to reinforcement swelling.

1. With the aim of performing consistent simulations for the mechanical behavior of concrete piles from the elastic to postpeak states, a numerical model for RC beams has been proposed, using fiber elements, which takes into account the effect of reinforcement swelling. The proposed model uses a simple uniaxial constitutive relation for a reinforcement bar in which a plastic buckling model is used to describe the swelling of a longitudinal reinforcing bar. In addition, the plastic buckling length of the longitudinal reinforcement bar is regarded as the length of the fiber element in order to avoid the mesh dependency of finite element results due to the softening in the constitutive relation. Numerical calculations to verify the accuracy of the proposed model were carried out, and the numerical results were compared with experiments of model cast-in-place RC pile specimens subjected to cyclic horizontal loadings. The proposed model gives good predictions of relationships of load versus displacement at the point of loading and the development of damage to piles, including the initiation and evolution of rapid bending strength loss.
2. An experiment of a model cast-in-place RC pile specimen embedded in sand and subjected to a fully reversed cyclic load was simulated with the proposed model. The comparison of pile behavior between the experiment and the simulation elucidates an

excellent agreement up to postpeak region. In addition, the numerical results suggest that the toughness of concrete piles embedded in the ground can be improved compared to that in air due to the lateral confinement of earth pressures to the cover concrete.

An example of dynamic analysis is given, which examines the seismic response of a bridge pier with a cast-in-place RC grouped pile foundation. The proposed model gives predictions of the evolution of the damage to the piles, including the swelling and the initiation of the rapid decrease in the bending strength capacity. Based on the results of the dynamic analysis, a seismic design method for pile foundations that can take into account the effects of ground vibration during earthquakes is discussed.

Recommendations for future research are summarized as follows:

- In the present study, there is no attempt to evaluate the performance of the modeling of concrete piles considering reinforcement swelling in simulations for a response of a pile group. We need to examine and improve the accuracy of the proposed model for the prediction of the behavior of pile groups.
- For the proposed model, there remain several problems to be solved and improved, particularly related to the conditions under which swelling is initiated. For example, we should develop a more suitable stress-strain relation for the cover concrete along the sustaining branch path of the maximum compressive stress, and we should establish a modeling method for the degrading gradients of such a constitutive law. The hysteretic mechanism for the stress-strain relation of reinforcing bars in the state immediately after swelling may need to be improved further.

Further discussions are needed as to how to set the design criteria for the ultimate limit state accompanying excessive tilting and/or horizontal displacement of pile foundations from the viewpoint of the foundation system. We need to carry out more numerical analyses using the proposed models in order to finally clarify the ultimate limit state of bridge foundations.

REFERENCES

- 1) Japan Road Association: *Specifications for Highway Bridges, Part V, Seismic Design*. Maruzen, Tokyo, 2002. In Japanese.
- 2) Japan Road Association: *Specifications for Highway Bridges, Part IV, Substructures*. Maruzen, Tokyo, 2002. In Japanese.
- 3) Shirato, M., Fukui, J., Unjoh, S., and Hoshikuma, J.: Japanese seismic design specifications for highway bridges. *Proc. of LSD2003: International Workshop on Limit State Design in Geotechnical Engineering Practice*, (Ed. by Phoon, Honjo, and Gilbert), Cambridge, Massachusetts, USA, 2003. ISSMGE TC23, World Scientific, Singapore. CD-ROM.
- 4) Fukui, J., Nakano, M., Kimura, Y., Ishida, M., Ookoshi, M., and Banno, A.: *Technical Memorandum of PWRI, Public Works Research Institute*, No. 3553, 1998. In Japanese.
- 5) Kimura, Y., Ookoshi, M., Nakano, M., Fukui, J., and Yokoyama, K.: An experimental study on the ductility of pile foundations. *J. of Struct. Eng., JSCE*, Vol. 44A, pp.1597–1606, 1998. In Japanese.
- 6) Maekawa, K., Tsuchiya, S., and Fukuura, N.: Nonlinear analysis of reinforced concrete and seismic performance evaluation. *Proc. of the Second Symposium on Nonlinear Numerical Analysis and its Application to Seismic Design of Steel Structures*, pp. 1–16. JSCE, 1998. In Japanese.
- 7) Suda, K. and Masukawa, J.: Models for concrete cover spalling and reinforcement buckling of reinforced concrete. *12th WCEE*, No. 1437, 2000.

- 8) Suda, K., Murayama, Y., Ichinomiya, T., and Shimbo, H.: Buckling behavior of longitudinal reinforcing bars in concrete columns subjected to reversed lateral loading. *11th WCEE*, No. 1753, 1996.
- 9) Nakamura, H.: An effect of the buckling of reinforcing bars on the postpeak behavior of RC structures. *Concrete Engineering Series 20*, pp. 98–100. JSCE, 1997. In Japanese.
- 10) Nakamura, H., Niwa, J., and Tanabe, T.: An effect of the buckling of reinforcement bar on the postpeak behavior of RC structures. *Proc. of the JCI*, Vol. 14, No. 2, pp.337–343, 1992. In Japanese.
- 11) Gomes, A. and Appleton, J.: Nonlinear cyclic stress-strain relationship of reinforcement bars including buckling. *Engineering Structures*, Vol. 19, No. 10, pp.822–826, 1997.
- 12) Nakamura, H.: Stress-strain relation of concrete as a characteristic of material. *Concrete Engineering Series 20*, pp. 92–95. JSCE, 1997. In Japanese.
- 13) Tomita, Y.: Overview of elasto-plastic finite deformation analyses by finite element method (Localized deformation and numerical simulation). *J. of the Japan Soc. for Tech. of Plasticity*, Vol. 36, No. 408, pp.2–9, 1995. In Japanese.
- 14) Public Works Research Institute: Report on the disaster caused by the 1995 Hyogoken Nanbu Earthquake. *J. of Research, Public Works Research Institute (PWRI)*, Vol. 33, 1996.
- 15) Research group on seismic design of pile foundations: Report on the effect of ground excitation to pile foundations during severe earthquakes. Technical report, 2000. In Japanese.
- 16) Kavvadas, M. and Gazetas, G.: Kinematic seismic response and bending of free-head piles in layered soil. *Geotechnique*, Vol. 43, No. 2, pp.207–222, 1993.

- 17) Subcommittee on dynamic interaction problem: Dynamic interaction of foundation, soil, and structure. An introduction to seismic design. Technical report, The earthquake engineering committee of JSCE, 1992. In Japanese.
- 18) Railway Technical Research Institute: *Seismic design code for railway structures*. Maruzen, Tokyo, 1999. In Japanese.
- 19) Murono, Y. and Nishimura, A.: Evaluation of seismic force of pile foundation induced by inertial and kinematic interaction. *12th WCEE*, 2000.
- 20) Tateishi, A. and Furuie, A.: A study on seismic performance characteristics of pile foundations estimated by nonlinear dynamic analysis with the effect of the varying axial force. *Proc. of the 25th JSCE Earthq. Eng. Symp.*, pp. 585–588. JSCE, 1999. In Japanese.
- 21) Fukui, J. and Shirato, M.: A relationship between the effects of kinematic interaction on pile foundation and ground conditions. *Technical memorandum of PWRI, Public Works Research Institute*, 2004. In Japanese.
- 22) Shirato, M., Tomonaga, N., and Fukui, J.: Ductility design of pile foundations considering both inertial and kinematic dynamic interaction. *Proc. of the fourth symposium on ductility design method for bridges*, pp. 379–386. JSCE, 2000. In Japanese.
- 23) Architectural Institute of Japan: *An introduction to dynamic soil-structure interaction*. Maruzen, Tokyo, 1996. In Japanese.
- 24) Ristic, D., Yamada, Y., Iemura, H., and Petrovski, J.: *Nonlinear behavior and stress-strain based modeling of reinforced concrete structures under earthquake-induced bending and varying axial loads*. No. 88-ST01 in Research Report. School of Civil Engineering, Kyoto University, Kyoto, 1988.
- 25) Hoshikuma, J., Kawashima, K., Nagaya, K., and Taylor, A. W.: Stress-strain model for

- confined reinforced concrete in bridge piers. *J. of Struct. Eng., ASCE*, Vol. 123, No. 5, pp.624–633, 1997.
- 26) Mander, J. B., Priestley, M. J. N., and Park, R.: Observed stress-strain behavior of confined concrete. *J. of Struct. Eng., ASCE*, Vol. 114, No. 8, pp.1827–1849, 1998.
 - 27) Maekawa, K., Pimanmas, A., and Okamura, H.: *Nonlinear mechanics of reinforced concrete*. Spon press, New York, 2003.
 - 28) Okamura, H. and Maekawa, K.: *Nonlinear analysis and constitutive models of reinforced concrete*. Gihodo, Tokyo, 1991.
 - 29) Menegotto, M. and Pinto, P. E.: Method of analysis for cyclically loaded R.C. plane frames including change in geometry and non-elastic behavior of elements under combined normal force and bending. *IABSE symposium on resistance and ultimate deformability of structures acted on by well-defined repeated Loads, Final Reports*, Vol. 13, pp. 15–22, Lisbon, 1973.
 - 30) Ohtsuka, H., Hoshikuma, J., Nagaya, K., and Murai, K.: Experiments of cast-in-place RC piles subjected to cyclic load. *Technical Memorandum of PWRI, Public Works Research Institute*, No. 3462, 1996. In Japanese.
 - 31) Kimura, Y., Ookoshi, M., Banno, A., and Fukui, J.: Experiments on restoring force characteristics of single pile in sand and subjected to lateral loads. *Proc. of the first symposium on ductility design method for bridges*, pp. 267–270. JSCE, 1998. In Japanese.
 - 32) Fukui, J., Kimura, Y., Ookoshi, M., and Banno, A.: Large scale experimental study on horizontal restoring force of single pile in sand. *Technical Memorandum of PWRI, Public Works Research Institute*, No. 3552, 1998. In Japanese.
 - 33) Tanamura, S., Kondoh, M., Kanamori, M., and Sugawara, A.: Cyclic loading tests of model piles with high-strength shear reinforcements. *RTRI Report, Railway Technical Research Institute*, Vol. 12, No. 12, pp.47–52, 1998. In Japanese.

- 34) Fukui, J., Nakano, M., Ishida, M., Nanazawa, T., Adachi, T., and Taguchi, H.: An investigation of actual situations on adoption of types of foundations. *Technical Memorandum of PWRI, Public Works Research Institute*, No. 3500, 1996. In Japanese.
- 35) Mander, J. B., Priestley, M. J. N., and Park, R.: Theoretical stress-strain model for confined concrete. *J. of Struct. Eng., ASCE*, Vol. 114, No. 8, pp.1804–1826, 1998.
- 36) Shirato, M., Koseki, J., and Fukui, J.: Nonlinear hysteretic load transfer model for winkler type soil-foundation interaction springs for deep foundations. *Int. Conf. on cyclic behavior of soils and liquefaction phenomenon*. Swets & Zeitlinger, Lisse, 2004.
- 37) Committee on publishing design examples of highway bridges: *Useful design examples of highway bridges, Civil Engineering Journal, Supplement*, pp. 121–141. Sankaido, Tokyo, April 1998. In Japanese.
- 38) Kimura, M. and Zhang, F.: Seismic evaluations of pile foundations with three different methods based on three-dimensional elasto-plastic finite element analysis. *Soils and Foundations*, Vol. 40, No. 5, pp.113–132, 2000.
- 39) Ōhira, A., Tazoh, T., Nakahi, S., and Shimizu, K.: Formulation of an analytical model of piled foundation for two-dimensional finite element method considering the effect of three-dimensional subgrade reaction. *J. of Struct. Mech. and Earthq. Eng., JSCE*, No. 356/I-3, pp.303–312, 1985. In Japanese.
- 40) Murono, Y.: *A study on the seismic design methodology of pile foundation considering the non-linear dynamic interaction caused by strong ground motion*. PhD thesis, Kyoto university, 1999. In Japanese.
- 41) Akita, N., Shirato, M., and Fukui, J.: Comparative study between large scale shaking table test and numerical analysis on dynamic behavior of sandy ground. *Proc. of the fifth symposium on ductility design method for bridges*, pp. 127–134. JSCE, 2002. In Japanese.

- 42) Iwasaki, T., Tatsuoka, F., and Takagi, Y.: An experimental study on dynamic deformation characteristics of soils, Part II – dynamic deformation characteristics of sand in wide range of strain level –. *J. of Research, Public Works Research Institute (PWRI)*, No. 153, 1980. In Japanese.
- 43) Iwasaki, T., Tokita, K., and Yoshida, S.: Dynamic deformation characteristics of alluvial clays – strain-dependency of shear rigidity –. *Technical memorandum of PWRI, Public Works Research Institute*, No. 1504, 1979. In Japanese.
- 44) Takeda, T., Sozen, M. A., and Nielsen, N. M.: Reinforced concrete response to simulated earthquakes. *J. of the Struct. Div., ASCE*, Vol. 96, No. ST12, pp.2557–2573, 1970.
- 45) Sakai, J. and Kawashima, K.: Nonlinear seismic response analysis of a reinforced concrete bridge column by the fiber element. *J. of Struct. Eng., JSCE*, Vol. 45A, pp.935–946, 1999. In Japanese.
- 46) Tazoh, T., Sato, M., and Mano, H.: The cause of ground fissures radiated from the footing of a bridge pier generated by the 1995 Great Hanshin earthquake. *Tech. Res. Report of Shimizu Co.*, No. 69, pp.47–56, 1999. In Japanese.

A . Finite Element Analysis of Plastic Buckling Behavior of Longitudinal Reinforcing Bars Constrained by Core Concrete and Hoop Bars

In the constitutive relation of longitudinal reinforcement bars including swelling proposed in Chapter 3, the stress-strain path from compression with swelling to expansion passes through the historical maximum tensile strain point at that time, as depicted in **Figure 3.2**. This assumes that longitudinal swelled reinforcement bars cannot carry tensile resistance compared with those in the straight state without swelling because of their kinematic shapes. In this Appendix, we briefly compare the behavior of swelled reinforcing bars obtained directly by finite element analysis and the proposed constitutive relation. Two patterns of reversed cyclic vertical loading are examined.

The type-1 and type-2 pile specimens are referred to again. The finite element model employed here is the same as that used in the plastic buckling analyses in Chapter 3 (2) with the length to be analyzed fixed at $L = 3s$. This length, $L = 3s$, is also employed in the numerical simulations for the experiments of the type-1 and type-2 specimens in Chapter 3 (3). We adopt equivalent linear springs for hoops in both types of specimen.

The averaged longitudinal stress-strain hystereses of the longitudinal reinforcing bar obtained by the finite element analysis are shown in **Figure A.1** for the type-1 specimen and in **Figure A.2** for the type-2 specimen, respectively, by solid lines. The averaged stress is estimated by dividing the vertical load at the loading point by the nominal cross sectional area of the reinforcing bar, and the averaged strain is estimated by dividing the vertical dis-

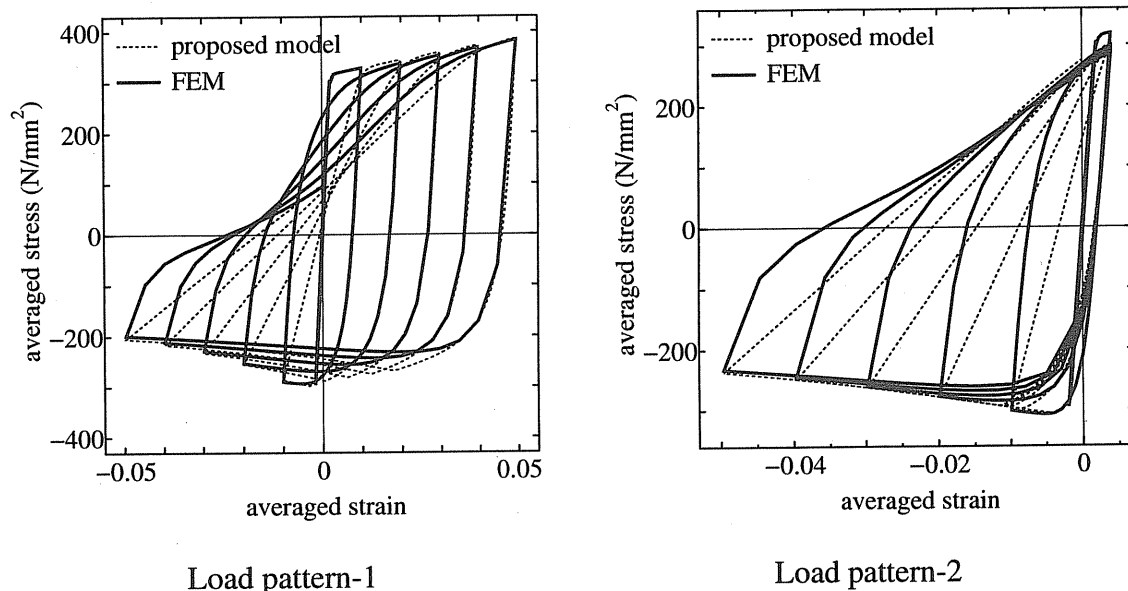
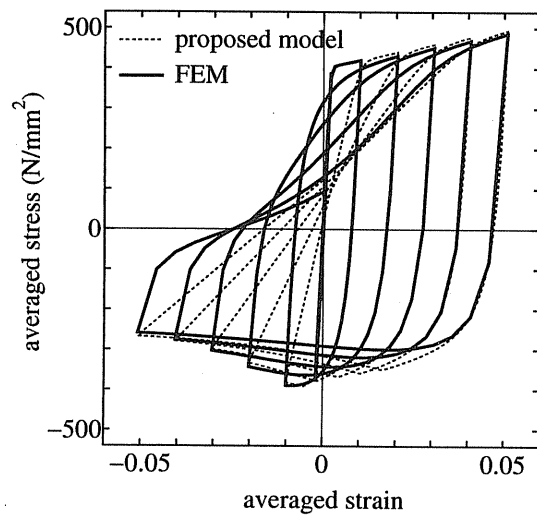
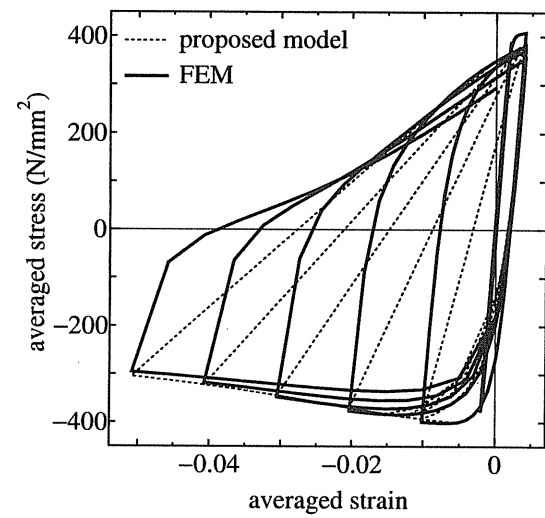


Figure A.1 Comparison between the finite element analysis result and the prediction by the proposed constitutive model for the behavior of the swelled reinforcing bar (type-1 pile specimen)

placement at the loading point by length $L = 3s$. The hystereses drawn by dotted lines are the stress-strain relationship given by the proposed model. Reinforcing bars carry tensile stress in RC members, and the coincidence in the tensile loading paths of FEM and the present model is of significance. The proposed model yields results that are in good agreement with the FEM results. The proposed model gives less rigidity than the FE results in an unloading path from a compressive loading path along with swelling to zero stress. However, the difference in the unloading paths from the compressive loading paths would be of small significance in the behavior of RC members because most of the compressive stress would be carried by core concrete.



Load pattern-1



Load pattern-2

Figure A.2 Comparison between the finite element analysis result and the prediction by the proposed constitutive model for the behavior of swelled reinforcing bar (type-2 pile specimen)

B . Effect of Stiffness of Hoop Bars in the Post Buckling Behavior of a Longitudinal Reinforcing bar

It is important to estimate precise plastic hinge length in seismic design of beam-columns, such as piers, piles, and column type foundations. The formation of a plastic hinge region and the decrease in lateral load resistance in such a region are highly affected by reinforcement swelling, because the softening of the resistance of reinforcement against bending-tension leads to that of bending strength and localization of deformation of an RC member. The swelling length and load resistance of a reinforcing bar depend on the arrangement, but their characteristics remain unclear.

In this appendix, we conduct numerical analyses for the plastic buckling behavior of longitudinal reinforcing bars in order to examine the characteristics thereof depending on the hardening ratio of longitudinal reinforcing bars, rigidity and spacing of hoops. We refer again to the arrangement of type-1 pile specimens as the basic pattern of arrangements to be examined in this appendix.

(1) Numerical model

The same plastic buckling simulation for a longitudinal reinforcement bar in Chapter 3 (2) is applied here. The differences are as follows:

- A hardening ratio E_1/E_0 is parametrically given from the values of 0, 0.01, and 0.02;

- An elastic spring is assumed to represent the resistance of a hoop. According to Section (2)a) of Chapter 3, a typical spring constant is $k_r = 1.7 \text{ MN/m}$ for hoops of the type-1 specimen;
- For the distributed spring representing the resistance of core concrete, the values of 1.57 kN/m, 15.7 kN/m, and 9.8 GN/m are given for the compressive spring constants. The compressive spring constant of 1.57 kN/m is estimated assuming the subgrade reaction coefficient of deteriorated concrete to be 1 kN/m^3 , and assuming that one spacing between two adjacent longitudinal reinforcement bars to be effective to resist the penetration of a longitudinal reinforcing bar. However, the numerical results are not sensitive to differences in these three values.

(2) Numerical results

a) Difference in the behavior of a reinforcing bar due to hardening ratio

First, we scrutinize sensitivities of swelling shapes and longitudinal load-displacement relationships to three hardening ratios $E_1/E = 0, 0.01, \text{ and } 0.02$. The lengths L of two or three times the length of the hoop spacing s are examined, and other factors are fixed, where the spring constant for hoop resistance is $k = k_r$, and the compressive spring constant of the distributed spring for core concrete is 1.57 kN/m. **Figure B.1** depicts the deformed shapes of $L/s = 2$ and 3 at an average strain of 5% by solid lines, together with the initial states by dashed lines. In the cases of $L/s = 2$, plastic buckling modes change with increase in the hardening ratio. In short, the effective plastic buckling length becomes longer with the increase in the hardening ratio. Plastic buckling occurs between layers of hoop when the values of hardening ratio E_1/E are the smaller values, 0 and 0.01, while plastic buckling involves both layers of hoop when E_1/E is the largest value of 0.02. However, plastic buckling modes do not change in the cases of $L/s = 3s$.

The load-displacement relationships are shown in **Figure B.2**. As the hardening ratio

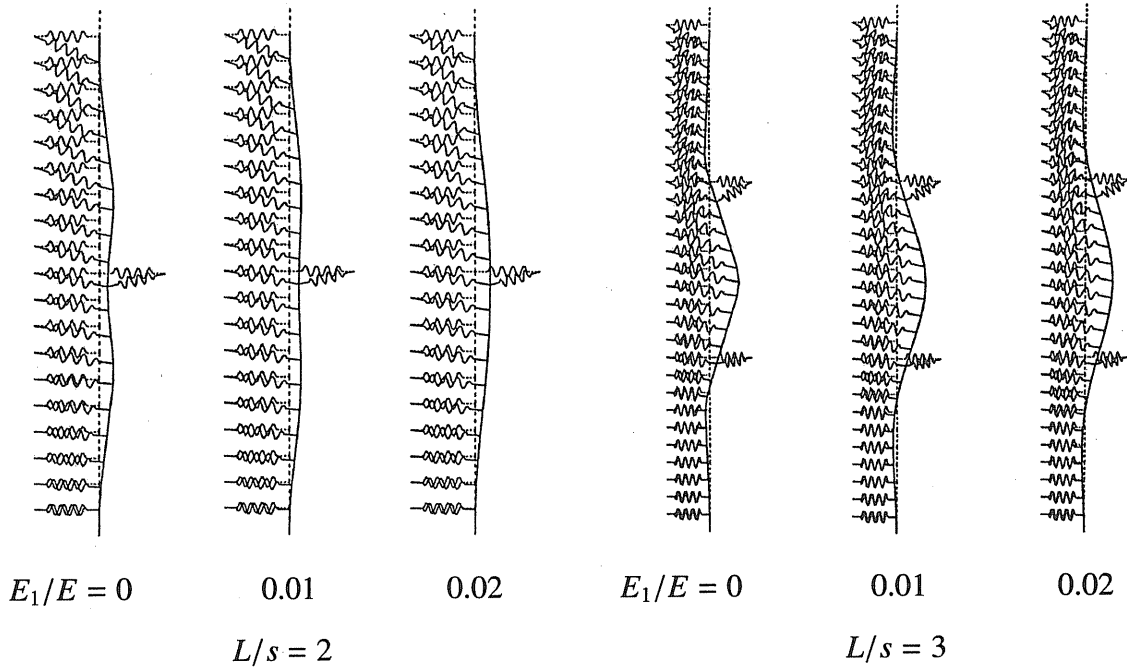


Figure B.1 Difference in deformed shape related to changes in the hardening ratio of the longitudinal reinforcing bar

becomes larger, the degradation of axial load resistance becomes smaller. This holds both in the cases of $L/s = 2$ and 3. It should be noted that although the curves in the case of $E_1/E_0 = 0.02$ continue to rise, the displacement level in both graphs is small compared with **Figure 3.9**, and the load decreases when further displacement evolves.

These findings indicate that post-buckling behavior is highly dependent on the hardening modulus of the longitudinal reinforcement bar. This also indicates that the bending stiffness after initial yielding influences the post-buckling behavior, and that employing longitudinal reinforcing bars of larger diameter demonstrate a longer plastic hinge length if the same material is considered. This inference is also supported by the experimental results for piers subjected to cyclic lateral loading, e.g. those reported by Hoshikuma et al.²⁵⁾

b) Difference in behavior depending on the spacing and rigidity of hoops

Next, plastic buckling behavior is computed under the conditions that the spring constant for hoops is reduced among the cases to be analyzed, but the length L , hardening ratio E_1/E_0 , and spring constant of the distributed spring for core concrete are maintained.

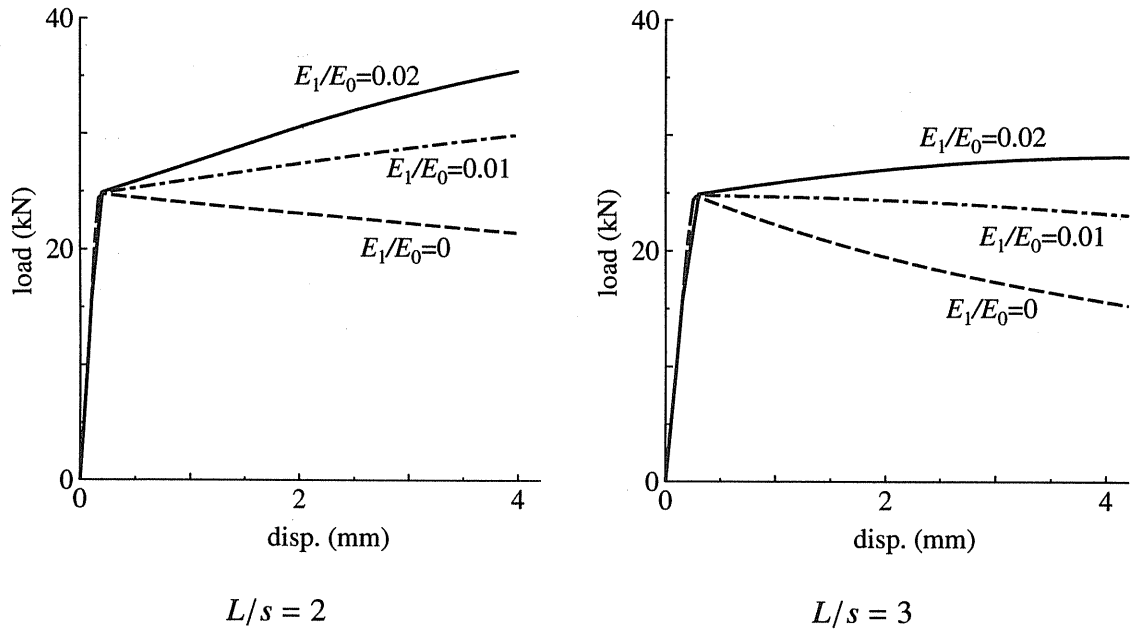


Figure B.2 Difference in normalized load-displacement relationship related to changes in the hardening ratio of the longitudinal reinforcing bar

Figure B.3 plots the relationships between longitudinal load and displacement normalized by each buckling mode, where $L/s = 3$, $E_1/E = 0.01$, and the value of the distributed spring constant for compressive resistance of core concrete is 9.8 GN/m. The smaller hoop resistance causes the larger decrease in fiber strength mobilized by reinforcement bars. As is evident from the deformed shapes shown in **Figure B.3**, together with the load-displacement relationships, which are at an average strain of 5%, lateral displacements at hoop positions decrease as the hoop rigidity increases.

Finally, plastic buckling simulations are executed under the conditions of 80, 40, or 20 mm of hoop spacing, where $L = 240$ mm, $E_1/E = 0$, and the compressive distributed spring constant for core concrete being 9.8 GN/m is assumed. The load-displacement curves at the loading point and the deformed shapes at an average strain of 5% are shown in **Figure B.4**. The length of each swelling portion is somewhat longer than the length of the hoop spacing, similar to the results obtained above. However, the important thing is that the smaller reduction in the vertical resistance of the reinforcing bar is revealed with decrease in hoop spacing due to shortening of effective plastic buckling length.

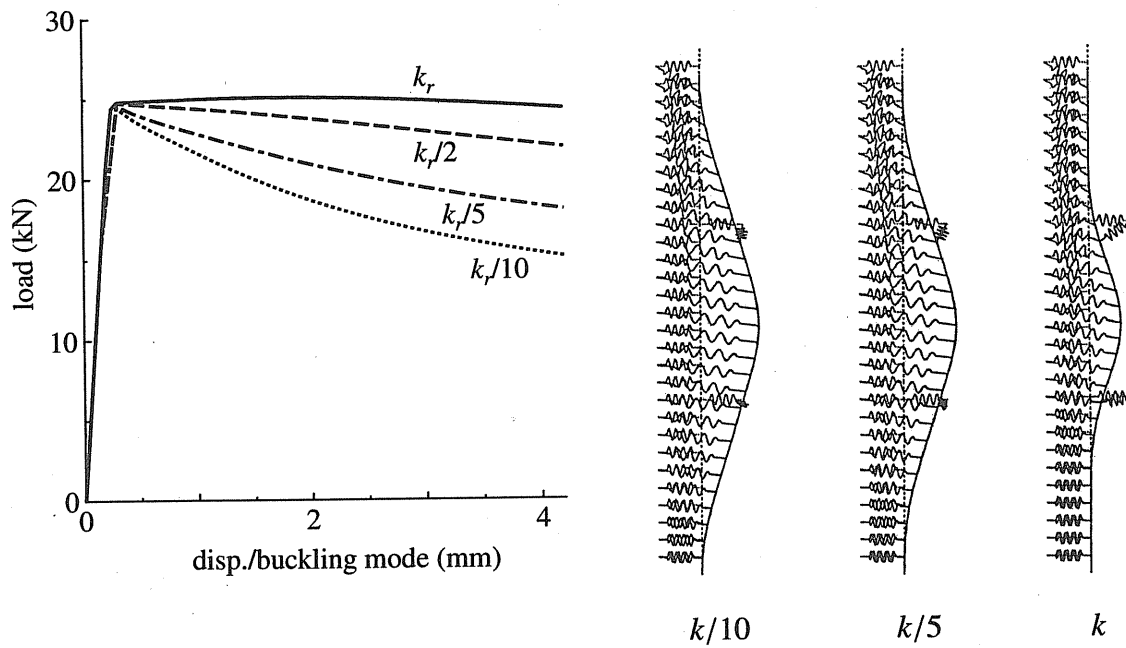


Figure B.3 Differences in normalized load-displacement relationships and plastic buckling shapes at an averaged strain of 5% depending on hoop resistance

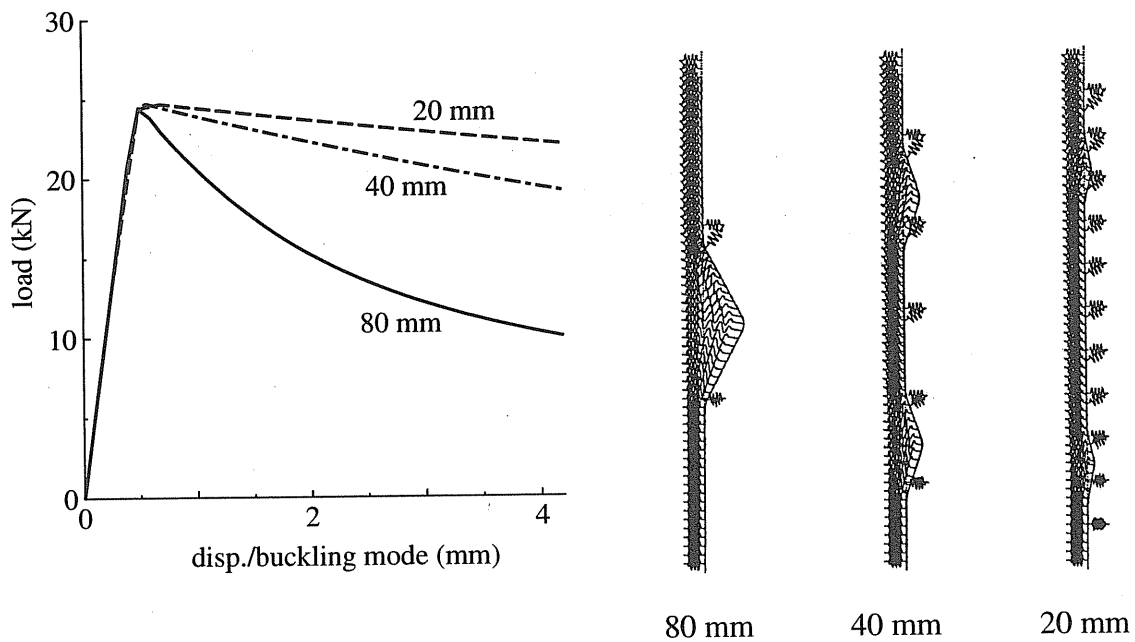


Figure B.4 Differences in normalized load-displacement relationships and plastic buckling shapes at an averaged strain of 5% depending on different hoop spacing

(3) Recommendations for future research

The plastic buckling length of longitudinal reinforcing bars depends on the hardening bending rigidity of reinforcing bar, hoop resistance, and hoop spacing. As a result, it is anticipated that reasonable estimations of swelling length of reinforcement and plastic hinge length are provided based on these factors. In addition, we demonstrate that plastic buckling analysis is a powerful method by which to estimate plastic hinge lengths as well as swelling length and the average stress-strain relation of a reinforcement. Furthermore, research on a reasonable formula of plastic buckling length to use in design practice and a sophisticated arrangement of reinforcements by which to obtain sufficient toughness of RC members are expected to be achieved by means of the plastic buckling simulation.