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Improvement of Load-Carrying Capacity by Steel Pipe Confining of Concrete Columns and Tie Bar Confining of Reinforced Concrete Columns

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Abstract

Since the Hyogoken-Nanbu Earthquake, there has been vigorous activity in the area of earthquake-resistant reinforcement and repair of existing bridge piers. This study aims at developing design concepts for the earthquake-resistant reinforcement of reinforced concrete columns. Axial compression tests are carried out on two types of concrete columns, some subjected to stress hysteresis and others unstressed (sound), that have been reinforced by encasing in a steel pipe. The deformation properties of the steel pipe are analyzed to examine the effectiveness of steel pipe confinement. The impact on deformation properties and reinforcement performance of the physical characteristics of the filling materials injected between the concrete column and the steel pipe is also studied, with useful results. Considering the case of new construction, it is found that the pitch of lateral confining steel reinforcement controls the load-carrying capacity of the core concrete, based on the relationship between the volume of lateral confining steel reinforcement for the main re-bars and core concrete strength.

1. INTRODUCTION

There is often a main rebar cutoff part way up a reinforced concrete bridge pier for reasons of economy. In the Great Hanshin Earthquake, several bridge piers were damaged because this cutoff resulted in a short anchorage length or the tie bar volume was small. As a result, the standards for new bridge piers have been revised to require closer spacing of the tie bars (that is, the lateral confining steel reinforcement) so that the tie bars play a role in strengthening the structure. Existing bridge piers are also being retrofitted with earthquake-resistant reinforcement with the aim of ensuring adequate deformation performance in the event of an earthquake. Several methods are in use for reinforcement of existing reinforced concrete bridge piers, including the steel plate lining method, reinforced concrete lining method, and FRP fiber lining method[1,2). Each method has its particular advantages and disadvantages (**Table 1**[3)]). Of these, the steel plate lining method is frequently selected because of its many advantages. It has been found through testing and analysis at many research institutions that the steel plate lining method offers a high level of reliability as well

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Table 1 Comparison between various bridge pier reinforcement methods

Method		RC lining method	Steel pipe lining method	Carbon fiber sheet lining method
Characteristic	Merit	•Economically excellent •Easy maintenance	•Thin covering depth •High rehabilitation effect	•Excellent for construction due to light weight •Thin covering depth
	Demerit	•Increase of dead load in foundation •Thick covering depth	•Required to use heavy equipment work •Required to repaint	•Required to make quality control •Unfittable for developing flexural load-carrying capacity

as easier construction, while stress calculations are also relatively simple using this method.

The steel plate lining method of pier reinforcement entails arranging steel plates around the outside of a bridge pier and these are then welded together. A filling material is injected into the gap between the steel plate and the concrete to give a monolithic structure. By ensuring adequate deformation performance in the event of an earthquake, this method prevents a brittle fracture by bending failure or shearing force when a tall column is subjected to a horizontal force[4]. However, there has been little analysis of the effectiveness of this reinforcement method with consideration for differences among existing bridge piers in load-carrying capacity and earthquake-resistant performance; that is, their soundness with regard to stress. Also, although there has been a great deal of study with regard to flexure, there has been little study on the effectiveness of this reinforcing method with regard to the axial compressive forces that can result from inland earthquakes, in which axial forces are sometimes predominant.

In this study, two kinds of concrete columns reinforced with steel pipes (a simplified form of the steel plate lining method) are tested, as follows: ①Concrete columns that have been subjected to momentary axial compression stress just to the point of breaking strength by means of an external force (ante-peak), and ②Concrete columns that have not been subjected to stress.

Having prepared the test pieces, we analyzed steel pipe deformation in relation to axial compressive force. Also, we used two kinds of filling material in the gap between the concrete and the reinforcing steel pipe, and we evaluated the differences in reinforcing effectiveness and impact on deformation properties that resulted from the physical characteristics of these filling materials.

To analyze the effectiveness of confining the main rebar of the reinforced concrete column member in the lateral direction with tie bars, we prepared a set of seven test pieces with different tie bar spacings for each of two tie bar materials, ordinary steel reinforcement (SD) and high-strength steel reinforcement (SBPD). We then conducted axial compression tests and studied improvement in the load-carrying capacity of the test pieces resulting from the confining effect of the different tie bar materials and tie bar spacings.

2. PROCEDURES

2.1 Materials

1) Concrete

Ordinary Portland cement was used for the concrete column model and the coarse aggregate is of a 20 mm maximum size and 2.63 g/cm^3 density. The 28 days compressive strength was 24 N/mm^2. **Table 2** shows the mix proportions of the structural concrete.

2) Filler

The two kinds of fillers were used: that is, the epoxy resin and the shrinkage compensating mortar (the main ingredient: alkylene oxide accompanied by the low-grade alcohol). Both agents are used frequently for reinforcement work on existing bridge piers. **Table 3** indicates the characteristics of the epoxy resin. **Tables 4** and **5** denote the mix proportions and characteristics of the shrinkage compensating mortar, respectively.

3) Steel pipe

In the actual reinforcement of reinforced concrete piers, reinforcing steel plates are arranged around the piers and then welded together. However, since welding would be difficult at the scale of the test model, JIS

Table 2 Mix propotions

W/C (%)	s/a (%)	Unit content (kg/m^3) W	C	S	G	Ad*
56	43.8	144	258	820	1065	2.75

* : Shrinkage reducing agent

Table 3 Characteristics of epoxy resin

Compressive strength	71.9 N/mm^2
Modulus of elasticity	2080 N/mm^2
Flexural strength	68.6 N/mm^2
Tensile strength	52.5 N/mm^2
Adhesive shear strength	16.3 N/mm^2

Table 4 Mix proportion of shrinkage compensating mortar

Unit content (kg/m^3) W	C	S	SCA*
193	334	668	24.4

* : Shrinkage compensating agent (Tetragurd As_{20})

Table 5 Characteristics of shrinkage compensating mortar

Compressive strength	21.7 N/mm^2
Tensile strength	1.5 N/mm^2

Table 6 Shape of steel pipe

Steel	Filler	Inner diameter (mm)	Thickness (mm)	Length (mm)	Young's modulus (kN/mm^2)	Yield point (N/mm^2)	Tensile strength (N/mm^2)
Type I	Epoxy resin	107.9	3.2	160	200	235	400
Type II	Shrinkage compensating mortar	132.6	3.5	160	200	235	400

G 3444 carbon steel pipe for general structures was used instead. **Table 6** shows the type of steel pipe used.

2.2 Preparation of Stress Hysteresis Concrete Columns

Using the compression testing system, we applied axial compressive force until just before the compressive strength limit to concrete columns 10 centimeters in diameter and 20 centimeters tall. The load was removed just before the compressive strength limit was reached, so the columns did not burst, but maintained their original form. During loading, we used an ultrasonic-scope to measure ultrasonic propagation time within the concrete columns, and found that the lateral propagation time across the column varied according to the magnitude of compressive force. In general, propagation time increased as compressive force increased within the elasticity range of the concrete used in the columns. However, our tests revealed sudden changes in propagation time at singular points[5)].

The method used in this study concerning the relationship between ultrasonic propagation time and the magnitude of compressive force is as follows. In the first iteration, compressive force is applied to sound concrete columns until the point just before the compressive strength limit is reached. This is the moment when the rate of increase in propagation time shows major deviation from the proportional relationship. Loading is stopped at this point, and the load is removed. The compressive strength at this point is that of a sound concrete column. Next in the second iteration, compressive force is again applied to the same concrete column. Propagation times are different than in the first iteration, but still increase in proportion to compressive force. However, now that the concrete column has already been subjected to a degree of stress approaching its compressive strength limit, it no longer exhibits the compressive strength of a sound column. The proportional relationship between propagation time and compressive force is lost sooner than in the first iteration, and the load-carrying capacity is considered to be the flow stress point at which a large increase in plastic deformation is observed. As this loading procedure is repeated, the load-carrying capacity of the concrete column continues to fall gradually. After a number of iterations, the result is concrete columns that have been subjected to stress hysteresis.

There is a more or less linear relationship between load-carrying capacity at each iteration and the difference in ultrasonic propagation time between the start of loading and the time at which the load-carrying capacity is reached. Therefore, by successively observing propagation times and predicting the difference, it is possible to estimate the load-carrying capacity of a concrete column. This makes it possible to prepare stress hysteresis concrete columns, or columns which have been subjected to repeated stress to reduce their load-carrying capacity to a certain level. **Table 7** shows the compressive strength and load-carrying capacity of the concrete columns used in this study.

2.3 Preparation of Test Pieces

The stress hysteresis concrete columns and unstressed concrete columns were reinforced with lengths of steel pipe. To ensure that the axial compressive force would not act directly on the steel pipe, detecting only the confining effect, the concrete columns protruded 20 millimeters above and below the steel pipe. A filling material was injected into the gap between the steel pipe and the concrete, producing a model column test piece. In a practical implementation of the steel plate lining method, a gap is left between the concrete and the steel plate because of filler flow properties. This gap is about 4 millimeters in the case of epoxy resin and about 25 to 30 millimeters in the case of shrinkage compensating mortar. In this study, test pieces using epoxy resin filler were prepared with a gap of 3.5 millimeters while those using shrinkage-compensating mortar were prepared with a gap of 20 millimeters. Preliminary tests confirmed that axial compressive force would cause lantern buckling, in which the center of the test piece shows the greatest deformation. Therefore, biaxial strain gauges were affixed to the steel pipe surface in four spots, equally spaced around the circumference at a position midway up the pipe. **Fig. 1** shows the detailed dimensions of the test pieces.

Table 7 Compressive strength and provisional load-carrying capacity of concrete column model (Unit in N/mm²)

	For epoxy resin grouted specimen			
	D*1	D2	D3	ND1~ND3
Compressive Strength	31.8	29.7	33.9	(Av.) 31.8
Provisional load-carrying capacity	22.5	20.4	22.0	———
Strength reduction ratio (%)	29.2	30.3	35.1	———

	For shrinkage compensating mortar grouted specimen			
	D*1	D2	D3	ND1~ND3
Compressive Strength	26.7	26.7	27.1	(Av.) 26.9
Provisional load-carrying capacity	18.5	18.2	18.8	———
Strength reduction ratio (%)	30.7	31.8	30.6	———

* damaged

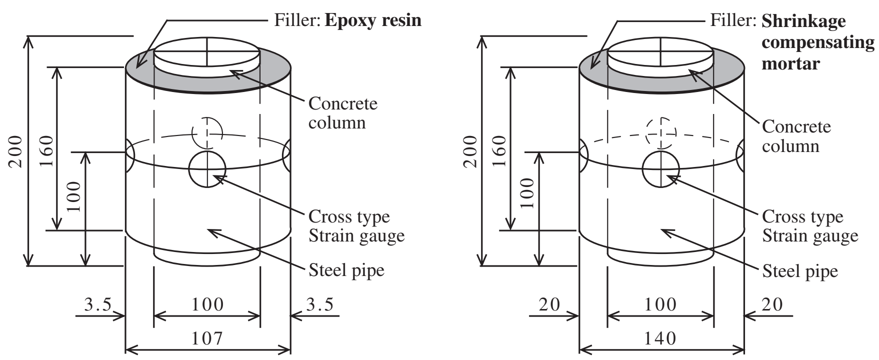


Fig.1 Detail of confined column model (Unit in mm)

2.4 Loading and Measurement Method

A compression load testing device (capacity: 2,000 kN) was used to apply a load at a fixed rate, and strain in the axial and circumferential directions was measured with strain gauges at each 10 kN compression step. **Fig. 1** illustrates the placement of strain gauges.

3. EXPERIMENTAL RESULTS AND DISCUSSION

In this test, we prepared three test pieces that had been subjected to stress hysteresis (D1-D3) and three unstressed test pieces (ND1-ND3). We measured the resulting strain when axial compressive force was applied to each test piece. Based on the relationship between load and strain, we studied the deformation properties of the test pieces and compared them against differences in filling materials (test pieces with epoxy resin filler: EP; test pieces with shrinkage compensating mortar: MO). Since the deformation properties of a test piece vary according to whether or not it has been subjected to stress hysteresis, the stress in the concrete inside the steel pipe was not uniform. For this reason, we used the macro compression stress divided by the cross sectional area of concrete to obtain the relationship between stress and strain.

3.1 Stress -Volumetric Strain Relation

Representing the longitudinal and lateral strains in the case of simple compression of a test piece as ε_C and ε_T, and assuming isotropy in volumetric strain ε_V, the theory of elasticity gives us equation (1). Since it has been experimentally confirmed that volumetric deformation is an effective quantitative index of non-elastic behavior and rupture[6)], in this study we calculated volumetric strain using equation (1) based on the average strain in each direction as obtained by testing. The relationship between stress and volumetric strain was then studied. To clarify the confining effect of the steel pipe, we also investigated the relationship between stress and volumetric strain in concrete columns without confinement.

$$\varepsilon_V = \varepsilon_C - 2\varepsilon_T \qquad (1)$$

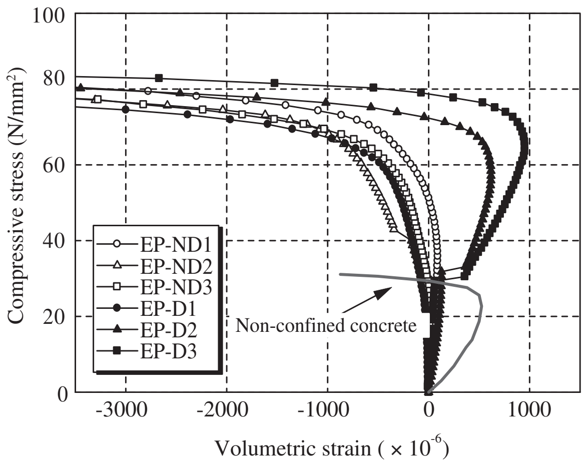


Fig.2 Stress-volumetric strain relation when epoxy resin

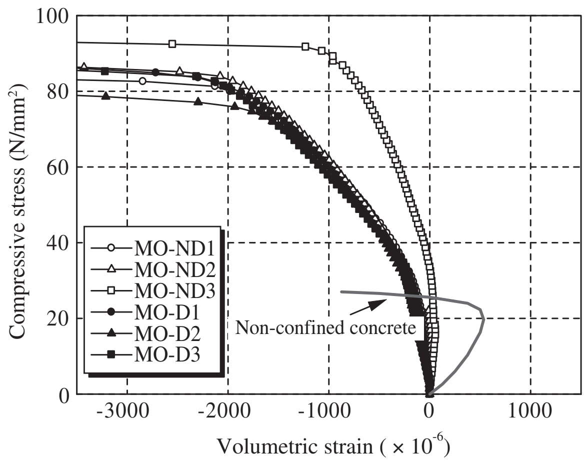


Fig.3 Stress-volumetric strain relation when shrinkage compensating mortar

where, ε_V is the volumetric strain, ε_C and ε_T is the axial strain and the transverse strain, respectively.

Fig. 2 shows that volumetric strain in test pieces with epoxy resin filler varies according to whether or not the test piece has been subjected to stress hysteresis. Test pieces that had been subjected to stress hysteresis exhibit significant compressive deformation. The apparent reason for this is that stress hysteresis causes structural relaxation (internal cracking) within the concrete column, so loading leads to greater compression of the column and a denser structure. Further, the epoxy resin used as a filling material has a low Young's modulus. Next, **Fig.3** shows the case of test pieces with shrinkage-compensating mortar filler. There is a relatively similar increase in strain regardless of whether or not the test piece has been subjected to stress hysteresis. Beginning at an early stage, volumetric expansion tends to result in a large amount of strain.

The differences between the two filler materials demonstrate that a shrinkage compensating mortar filler shows a greater tendency to transmit strain to the steel pipe than epoxy resin filler. Because the epoxy resin has a lower Young's modulus and good adhesion, overall integration is maintained between the concrete and the steel pipe; and since the volumetric elastic modulus increases, the strain increases in a straight line. In contrast, since the volumetric elastic modulus is low in the case of shrinkage compensating mortar, the increase in strain describes a curve, with plastic deformation behavior observed from an early stage. Comparing volumetric strain in both types of test pieces, we see greater deformation in test pieces with epoxy resin filler. That is, because more energy is absorbed, and there is better deformation capacity.

3.2 Determination of Load-Carrying Capacity

Figures 4 and **5** are log-log plots of the relationship between stress and strain due to strain in the circumferential direction in our tests. We have used this method here because the kinks obtained by double logarithmic plotting of stress and strain[7] represent singular points of the physical characteristics[8]. We evaluated the load-carrying capacity of test pieces by this method.

Fig. 4 shows that the load-carrying capacity of a test piece with epoxy resin filler is approximately 80 N/mm², and there is no significant difference in load-carrying capacity if the test piece has been subjected to stress hysteresis. However, there is a difference in yield strain according to whether or not the test piece has been subjected to stress hysteresis: the yield strain is approximately 880×10^{-6} with stress hysteresis and 600×10^{-6} without stress hysteresis. The reason for this seems to be that stress hysteresis causes internal structural relaxation in the concrete columns, increasing their tendency for volumetric expansion. **Fig. 5** shows that the load-carrying capacity of a test piece with shrinkage compensating mortar filler is approximately 85 N/mm², and as in the case of test pieces with epoxy resin filler, there is no significant difference in the load-carrying capacity of the test piece that has been subjected to stress hysteresis. Again, though, yield strain is different, at approximately 600×10^{-6} with stress hysteresis and approximately 350×10^{-6} without stress hysteresis. This is thought to be for the same reason as in the case of test pieces with epoxy resin filler.

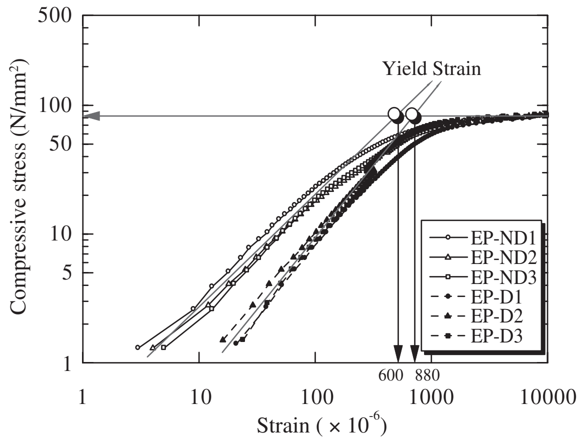


Fig.4 Logarithmic expression of stress-strain relation [epoxy resin]

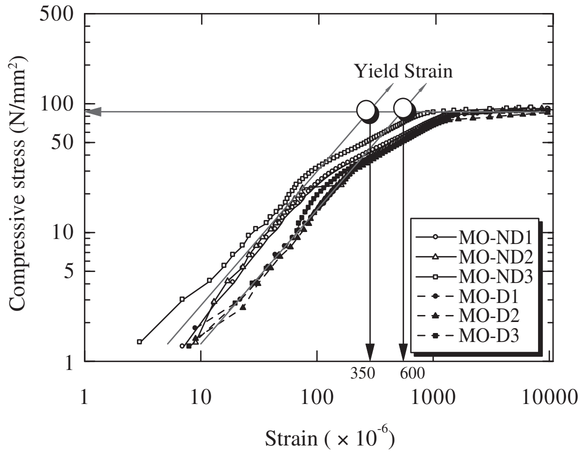


Fig.5 Logarithmic expression of stress-strain relation [shrinkage compensating mortar]

4. COMPRESSION TESTING OF COLUMN MODEL WITH CONSIDERATION OF TIE BAR SPACING

The new earthquake resistance standards require closer spacing of tie bars in new bridge piers. We studied the effects of tie bar spacing in reinforced concrete column members on the compressive strength of the core concrete. Model test pieces of reinforced concrete columns were prepared with two kinds of tie bars, ordinary steel reinforcement (SD) and high-strength steel reinforcement (SBPD), conducted axial compression tests, and studied the improvement in strength resulting from closer tie bar spacing under triaxial compression stress.

4.1 Current Calculations for Design Strength upper-bound for Reinforced Concrete Columns

1) Constitutive Equations[1)]

To ensure that the ratio of design maximum axial compression strength N'_{oud} to the design axial compression load N'_d is not lower than the structure coefficient γ_i (ranging from 1.0 to 1.2), members must be designed to handle the expected axial compression load. For members with a very small ratio of design flexural moment M_d to design compression load, the ultimate resistive force fall off with just a small increase in flexural moment due to inconsistencies introduced into members at the construction stage. To avoid this kind of problem, an upper limit is placed on axial compression load-carrying capacity, and a member coefficient of 1.3 is used. The upper limit on axial compression load-carrying capacity under axial compressive force N'_{oud} is obtained with equation (2) if tie bars are used, or with either equation (2) or equation (3) if spiral reinforcement is used, whichever yields the greater strength.

$$N'_{oud} = (k_1\, f'_{cd}\, A_c + f'_{yd} A_{st}) \,/\, \gamma_b \qquad (2)$$

$$N'_{oud} = (k_1\, f'_{cd}\, A_e + f'_{yd} A_{st} + f'_{pyd} A_{spe}) / \gamma_b \qquad (3)$$

The variables used here are as follows. A_c is Cross sectional area of concrete, A_e is Cross sectional area of concrete surrounded by spiral reinforcement, A_{st} is Total cross sectional area of steel reinforcement in the axial direction, A_{spe} is Converted cross sectional area of spiral reinforcement ($= \pi d_{sp} A_{sp}/s$), d_{sp} is Diameter surrounded by spiral reinforcement, A_{sp} is Cross sectional area of spiral reinforcement, s is Pitch of spiral reinforcement, f'_{cd} is Design compressive strength of concrete, f'_{yd} is Design compressive yield strength of reinforcement steel in the axial direction, f_{pyd} is Design tensile yield strength of spiral reinforcement, K_1 is Strength reduction coefficient ($= 1 - 0.003f'_{ck} < 0.85$), f'_{ck} is Physical property of concrete strength (N/mm^2) and γ_b is Member coefficient (generally 1.3).

Fig.6 An example of damaged pier when great earthquake

2) Stress capacity of concrete

The design standard strength of concrete in the specifications (former regulations), or f'_{ck} has been replaced by f'_{cd} as obtained using equations (1) and (2). This is generally reasonable, but the following is observed in the case of $0.85f'_{ck}/\gamma_c$, which is in common use.

$$f'_{cd} = 0.85f'_{ck}/1.3 = 0.65f'_{ck} \text{【when} \leqq 50\text{N/mm}^2\text{】}$$
$$= 0.57f'_{ck} \text{【when} \leqq 60\text{N/mm}^2\text{】}$$

Here, γ_c is the concrete member coefficient and has a value of 1.3 (or 1.5 if $f'_{ck} \geq 60$ N/mm^2).

For these reasons, the design compressive strength of concrete is generally taken to be 57% to 65% of the design standard strength. This stress level corresponds to the limit of proportionality in the relationship between concrete stress and strain. This technique does not conform to the ultimate limit state design concept, nor to design theory since it is an amalgamation of stress tolerance design techniques, service limit state design techniques, and ultimate limit state design techniques.

3) Upper-bound of axial compressive load-carrying capacity with consideration of main rebar buckling

It is well known that main rebar fail to exhibit any significant effect when reinforced concrete columns are subjected to axial compression. This is simply because main rebar do not possess compressive strength. Instead, they undergo elastic failure through buckling.

Fig. 6 shows an expressway bridge pier that suffered damage in the Great Hanshin Earthquake of 1995. This seems to back up the observation that, in addition to experiencing an excessively large seismic load, the column had a significantly lower load-carrying capacity than estimated. In fact, if buckling is taken into consideration, the load-carrying capacity of main rebar depends on the buckling load, which it turn is a function of the slenderness ratio of the main rebar. Here, the slenderness ratio λ is expressed as in equation (4).

$$\lambda = \ell/(\phi/4) \qquad (4)$$

Here, ℓ is the length of the main rebar, and ϕ is their diameter.

The critical slenderness ratio Λ and buckling stress σ_s, in the case that both ends of the main rebar are coupled with pins, are given by equations (4) and (5) respectively[9].

$$\Lambda = (\pi^2 E_s/f'_{yd})^{1/2} \qquad (5)$$
$$\sigma_s = f'_{yd}/[1+f'_{yd}\lambda^2/(\pi^2 E_s)] \qquad (6)$$

Here, f'_{yd} is the design yield strength of the main rebar, and E_s is their elasticity modulus.

Therefore, the maximum load-carrying capacity N'_{oub} if the buckling effect is taken into consideration is generally as given by equation (7).

$$N'_{oub} = A_e f'_c + A_s \sigma_s \qquad (7)$$

Here, A_e is the core cross sectional area of the concrete.

4.2 Testing of Reinforced Concrete Columns with Consideration for Tie Bar Spacing

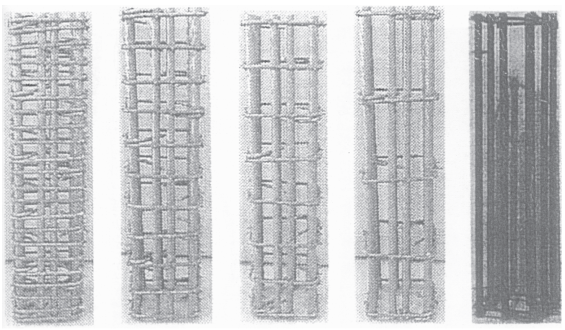
(1) Preparation of reinforced concrete columns

We prepared tie bar reinforcement frames as follows: D13 (diameter 12.7 mm, ordinary reinforcement: SD type, f'_{yd} = 333 N/mm^2), U13 (diameter 13.1 mm, high strength reinforcement: SBPD type, f'_{yd} = 1,424 N/mm^2), and U6.4 (SBPD type, f'_{yd} = 1,446 N/mm^2).

The column model test pieces had dimensions 150 × 150 × 530 millimeters and core dimensions 120 × 120 millimeters. We used seven different nominal tie bar spacings: 25, 50, 75, 125, 170, 250, and 500 millimeters. **Fig. 7** is an example of a hollow reinforcement frame. The maximum size of coarse aggregate in the concrete was 10 millimeters, and the structural concrete had an average compressive strength of f'_c = 64 N/mm^2 after 28 days of water curing. In these tests, we used concrete with high compressive strength in view of the use of higher strength concrete in recent years.

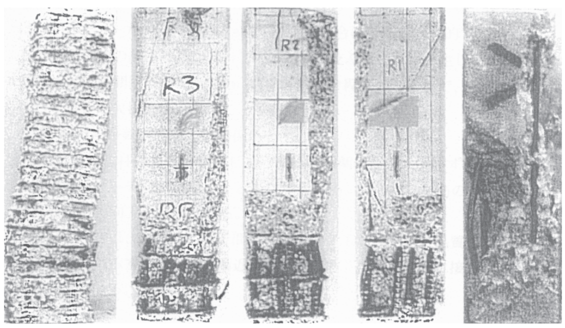
2) Failure mode of column models

Fig. 9 shows the failure mode of SBPD type steel reinforcement for each tie bar spacing. As described in Reference 10, spalling of the cover concrete increased significantly with larger tie bar spacing, and the effective cross section tended to suffer deeper damage. It should be noted in particular that in the case of intermediate tie bar spacing, at least 170 millimeters, the buckling main rebar did not influence the core concrete and so no effective lateral pressure was exerted. The same failure mode was observed with the SD type steel (**Fig. 8**). We formulated the following three deformation categories based on tie bar spacing and failure mode observed in these tests.



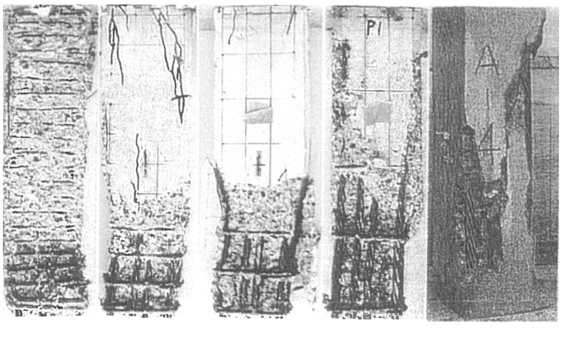
① s=25 ② s=50 ③ s=75 ④ s=125 ⑤ s=500

Fig.7 Example of hollow reinforcement cages [s:mm]



① s=25 ② s=50 ③ s=75 ④ s=125 ⑤ s=500

Fig.8 Failure modes of SD type RC column



① s=25 ② s=50 ③ s=75 ④ s=125 ⑤ s=500

Fig.9 Failure modes of SBPD type RC column

25mm ≦ s < 50 mm ····· "Most ductile"
50mm ≦ s < 125mm ····· "Moderately ductile"
125mm ≦ s ≦ 500mm ····· "Brittle"
(250mm ≦ s ≦ 500mm ····· "Especially most brittle")

5. STUDY OF CONFINING EFFECT

5.1 Test Pieces of Concrete Column Reinforced with Steel Pipe

Fig. 10 shows the relationship between core concrete compressive strength and ratio of load-carrying capacity. The ratio of load-carrying capacity is obtained by dividing the experimental load-carrying capacity by the compressive strength of the core concrete.

Based on **Fig. 10**, this value is 2.6 for unstressed test pieces with epoxy resin filler and 3.7 for those that had undergone stress hysteresis. Similarly in the case of test pieces with shrinkage compensating mortar filler, the value is 3.2 for unstressed test pieces and 4.6 for those that had undergone stress hysteresis. The confining strength ratio declines as core concrete strength rises. This means that even with low-strength concrete, a high load-carrying capacity is obtained as long as lateral confinement is adequate. Since these tests had a number of inadequacies, further testing will be needed to elucidate the impact of stress hysteresis on confinement effect.

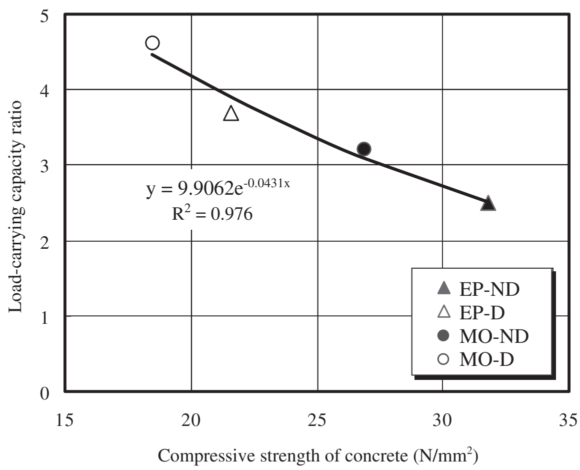


Fig.10 Relationship between load-carrying capacity ratio and compressive strength of concrete

5.2 Reinforced Concrete Column Model Test Pieces

Fig. 11 shows the relationship between tie bar spacing and reinforced concrete column load-carrying capacity for each type of steel reinforcement. This illustrates that load-carrying capacity rises as tie bar spacing decreases, and this tendency is prominent under heavier confinement (s ≤ 50 mm). Next, comparing the effects of different types of steel reinforcement, no difference in load-carrying capacity ratio was found in relation to the type of steel reinforcement at intermediate tie bar spacings of at least 170 millimeters. However, differences in load-carrying capacity ratio were observed when the tie bar spacing was less than 170 millimeters, and this effect grew more pronounced with heavier confinement. That is, regardless of the quality of the main rebar in a reinforced concrete column, its load-carrying capacity gradually approaches the upper bound. In the experiment, when the column dimensions and elasticity modulus were the same and the pitch intervals was greater than 300 millimeters, the tie bars provided no significant lateral confining effect, and with both SD and SBPD tie bars the load-carrying capacity approached the upper bound as obtained by equations (5) and (6), the basic design equations for tie bar columns.

This indicates that a higher load-carrying capacity can be achieved by the use of appropriate lateral confinement along with stronger main rebar, producing a lattice effect through the interaction of main rebar and tie bars.

6. CONCLUSIONS

In this study, we carried out axial compression experiments to investigate the confinement effect of steel plates and tie bars on concrete columns. The experiments involved test columns reinforced with steel pipe (and with two types of filler: epoxy resin and shrinkage compensating mortar) and reinforced concrete columns with various types of tie bar confinement (using SD and SBPD). Within the scope of the experiments, our analysis leads to the following conclusions:

(1) Test columns showed different deformation response at an early stage depending on whether or not they had been subjected to stress hysteresis, but the ultimate load-carrying capacity was found to be about the same. However, significant differences were observed in the rate of reinforcement effectiveness.

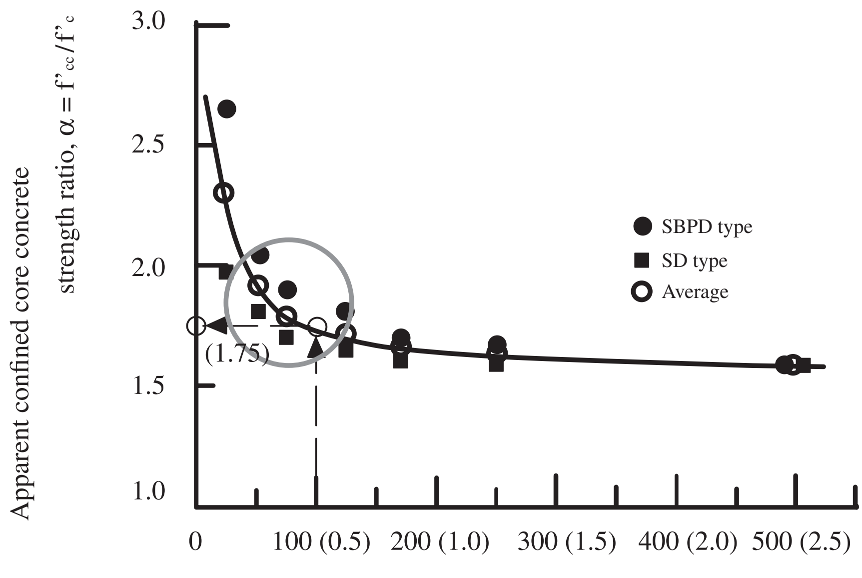


Fig.11 Relationship between load-carrying capacity ratio and spacing of tie bars

(2) There was little difference in the load-carrying capacity of test columns according to the type of filling material used. The value was approximately 80 N/mm^2 in the case of test columns with epoxy resin filler, and approximately 85 N/mm^2 in the case of test columns with shrinkage compensating mortar filler.
(3) Test pieces with epoxy resin filler had a high volumetric elastic modulus and exhibited linear behavior. Meanwhile, test pieces with shrinkage compensating mortar filler had a low volumetric elastic modulus and exhibited plastic deformation behavior. This indicates that the type of filling material has a significant effect on deformation properties.
(4) The failure mode of concrete columns with confinement tie bars were classified into three deformation categories according to tie bar spacing.
(5) In the case of test columns with epoxy resin filler, load-carrying capacity when confined with a steel pipe increased by 2.6 times if unstressed and by 3.7 times if subjected to stress hysteresis. Similarly in the case of test columns with shrinkage-compensating mortar filler, load-carrying capacity increased by 3.2 times if unstressed and by 4.6 times if subjected to stress hysteresis.

We then confirmed the following two points: (1) a highly effective reinforcement performance is obtained by applying an appropriate reinforcement method to stress-hysteresis columns of bridge piers, etc., that have sustained massive damage and (2) the failure mode of concrete columns with tie bar confined steel reinforcement can be classified into 3 types according to tie bar spacing.

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コンクリート柱の鋼管拘束とRC柱の帯鉄筋拘束が耐力向上に及ぼす影響に関する研究

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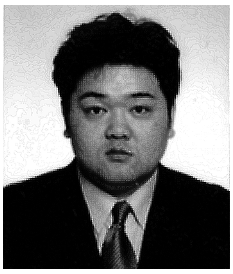
概　　要

既設橋脚の耐震補強・補修が，兵庫県南部地震を契機に精力的に行われている。本研究は，ＲＣ柱の耐震補強に関するデザインコンセプトを求めるため，応力履歴および応力上健全な２種類のコンクリート柱を鋼管で巻き立て補強を施した上で軸圧縮実験を行い，鋼管の変形性状の解析と鋼管による拘束効果の有効性を考察したものである。また，コンクリート柱と鋼管の間隙に注入する充填材の物性値が変形性状と補強効果に及ぼす影響の検討も行い，有用な知見が得られた。さらに，ＲＣ橋脚を新設する場合のＲＣ柱主鉄筋に対する横拘束鉄筋量とコア・コンクリートの耐力との関係から，横拘束鉄筋のピッチがコア・コンクリートの耐力を支配することを明らかにした。

Biographical Sketches of the Authors



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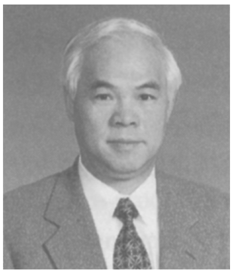

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