Proposal of Design Equation for Shear Strength of Reinforced Concrete Columns Without Transverse Reinforcement

横補強筋のない鉄筋コンクリート柱のせん断耐力設計式の提案

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ABSTRACT

The Great Hanshin Earthquake in 1995 had caused devastating damage to many reinforced concrete structures. Among these serious damages, catastrophic shear failure was the significant failure mode to column serving highway, railway viaduct, underground subway and reinforced concrete building. These fatal damages and losses are the main motivation for the author to conduct the research in shear design.

The current JSCE shear design equation for linear member without web reinforcement had formulated from the experimental results of rectangular reinforced concrete beams without web reinforcement. The first form of shear design equation had elaborately proposed by Okamura and Higai (1980). Niwa et al. (1986) had reconsidered Okamura's equation and proposed revaluation of shear design equation, which later was adopted by JSCE as "Standard Specification for Concrete Structure" (JSCE 1986) and used until present.

To apply the current JSCE shear design equation for column member, there are two differences between reinforced concrete beam and column members. The first different is contribution of side reinforcement in the case of column member. The second different is the section shape for column member, which may be circular, octagonal, or square section, while the one for beam member is only rectangular section. The variation in section geometry is resulting in problem of appropriate definition of effective depth. For circular cross section, current JSCE specification adopts the concept of transform section to equivalent square sections, which has the same cross sectional area and the effective depth is defined as the distance from compression face to centroid of tensile reinforcement arranged in 90-degree portion.

From 23 collected experimental results of circular column, it was found that current JSCE specification for circular column based on transform section concept is quite conservative with average Vexp/Vcal=1.40. By consider the fact that current JSCE shear design equation is very accurate for the case of rectangular reinforced concrete beam with no side reinforcement and this current form of shear design equation is widely used and accepted, thus the objective of this research is to obtain the proposal of shear design equation for reinforced concrete column with side reinforcement and with various cross section by no changing the general form of current JSCE shear design equation.

The first consideration is definition of effective depth, since it affects many functions; a/d function, size effect function, reinforcement ratio (As/bd), and effective concrete area (bd). For the case of circular section, the effective depth defined as full section depth is seem overestimate and effective depth for equivalent square section is seem underestimate. Thus the appropriate effect depth should be the one between these two extreme cases. Hence, the author proposed another definition of effective depth as the distance from compression face up to lowest level of tensile reinforcement.

The second consideration is the appropriate portion of tensile reinforcement accounted for reinforcement effect. Compare to JSCE specification that account the reinforcement arranged in 90-degree portion, the cases in analysis were expanded to the one in 120, 150 and 180-degree portion. The last case in analysis was the one proposed by Ishibashi et al. (1985) in summation form of reinforcement at each layer multiplying with distance from compression face to that layer and normalizing by distance from compression face to lowest layer of tensile reinforcement.

From these two considerations, there are four parameters in the analysis; effective concrete area, effective depth for a/d function, effective depth for size effect function and effective longitudinal reinforcement. These four parameters are leading to 3*3*3*5 =135 combination cases. Among these 135 combination cases, there are four cases, which have the lowest coefficient of variation (COV) in the identical level of accuracy. Among these four cases, the most accurate of Vexp/Vcal is the case that uses effective concrete area up to lowest tensile reinforcement, effective depth up to lowest tensile reinforcement for a/d function and size effect function, and effective reinforcement in summation form as proposed by Ishibashi et al. (1987).

However, by considering the consistency of parameter in practice, the author propose that the effective concrete area is defined as the area above lowest tensile reinforcement, and the effective depth is defined as the distance from compression face up to lowest layer of tensile reinforcement and used it both in a/d and size effect function. For simplicity in practice, the author proposed the effective longitudinal reinforcement as the half of total longitudinal reinforcement. The verification with experimental results of 23 circular columns show good calculated results in mean and variation comparing to current JSCE specification. Lastly the author is applied the proposal to octagonal and square column. The collected experimental results of 3 octagonal and 25 square RC columns with side reinforcement were used in verification.

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CONTENTS

Chapter 1

INTRODUCTION

1.1 BACKGROUND

1.1.1 Inspiration and Motivation from the Hanshin Earthquake to Research in Shear

The Great Hanshin Earthquake in 1995 had caused devastating damage to many reinforced concrete structures. A large number of reinforced concrete bridge piers were destroyed or severely damaged. Railway lines, including Shikansen lines, were severely damages. Many reinforced concrete pillars of underground subways and stations were collapsed. Among these serious damages, catastrophic shear failure was the significant failure mode to column serving highway, railway viaduct, underground subway and reinforced concrete building (see Figure 1.1a~d). These fatal damages and losses are the main inspiration and motivation for the author to conduct the research in shear design.

1.1.2 Reviews of Current JSCE Specification for Shear Design Equation

The current JSCE shear design equation for liner member without web reinforcement was originally formulated from the experimental results of reinforced concrete beam without web reinforcement. Before obtaining the current form of shear design equation, the previous form of shear design equation had elaborately proposed by Okamura and Higai (1980) as follows,

$$V_c = 0.20 f_c'^{1/3}(1+\beta_p+\beta_d)[0.75+1.4/(a/d)]\cdot b_w d \tag{1.1}$$

$$\beta_p = \sqrt{P_w} - 1 \quad : P_w \le 3\% \ (P_w \text{ in } \%) \tag{1.2}$$

$$\beta_d = d^{-1/4} - 1 \quad : d \le 1.1\,\text{m} \ (d \text{ in m}) \tag{1.3}$$

a) Shear failure of highway bridge pier

b) Shear failure of railway viaduct

c) Shear failure of intermediate column leading to collapse of underground subway

d) Shear failure and collapse of first floor building column

Figure 1.1 Shear failures of reinforced concrete structures due to Great Hanshin Earthquake

Okamura and Higai's equation later was adopted by JSCE as "Recommendation for Limit State Design of Concrete Structure" (JSCE 1983). Okamura and Higai's equation was formulated from the experimental results of reinforced concrete beam without web reinforcement conducted by the past research both in Japan and abroad. Most of the data were the beam with effective depth 0.10~0.50 m and with reinforcement ratio more than 0.50%. There was few data for the beam with effective depth more than 1.0 m and with reinforcement ratio less than 0.50%.

Niwa et al. (1986) had investigated the experimental results in the past and proposed revaluation of the equation for shear strength of reinforced concrete beam without web reinforcement as follows,

$$V_c = 0.20(P_w f_c')^{1/3} d^{-1/4}[0.75 + 1.4/(a/d)] \cdot b_w d \tag{1.4}$$

Niwa's equation was replaced the summation form of Okamura and Higai's equation by the multiplication form of size effect and tensile reinforcement effect. By consider the fact that some reinforcement concrete structures like footings and culvers are commonly large size with low reinforcement. Therefore, to verify the proposed equation and cover this application range, Niwa et al. (1986) had conducted the experiment of large-scale reinforced concrete beams with low tensile reinforcement.

Niwa's equation later was adopted by JSCE as "Standard Specifications for Concrete Structure" (JSCE 1986) as follows,

$$V_c = \beta_d \cdot \beta_p \cdot \beta_n \cdot 0.20\sqrt[3]{f_c'} \cdot b_w d \tag{1.5}$$

$$\beta_d = \sqrt[4]{1/d} \quad (\mathrm{d:m}) \qquad \le 1.5 \tag{1.6}$$

$$\beta_p = \sqrt[3]{100 P_w} \qquad \le 1.5 \tag{1.7}$$

$$\beta_n = 1 + M_o/M_d \qquad \le 2.0 \quad (N_d \ge 0) \tag{1.8}$$

$$\beta_n = 1 + 2M_o/M_d \qquad \ge 0 \quad (N_d < 0) \tag{1.9}$$

where $P_w = A_s/bd$

f_c' is concrete compressive strength (N/mm^2)

N_d is design axial compressive force

b_w is web width (mm)

d is effective depth (mm)

M_0 is decompression moment to cancel axial stress

M_d is design moment.

For circular cross sections, JSCE specification defined web width as the width of equivalent square section, which has the same area. The effective depth for circular section was defined as the distance from the edge of equivalent square section at compression side to the centroid of the tensile reinforcement arranged in 90-degree portion. The tensile reinforcement accounted for reinforcement ratio was defined as the reinforcement arranged in 90-degree portion of tension side.

1.1.3 Summary of Other Major Code Expressions and Empirical Equations for Shear Strength without Web Reinforcement

ACI 318-89 (1989)

In 1962, the ACI-ASCE Committee 326 on Shear and Diagonal Tension presented the shear design equation at inclined cracking of reinforced concrete beams without web reinforcement as follows,

$$V_c = \left(\frac{\sqrt{f'_c}}{7} + \frac{120}{7} \rho_w \frac{V_u d}{M_u} \right) b_w d \quad \le 0.29\sqrt{f'_c} \cdot b_w d \qquad (1.10)$$

where $\rho_w = A_s / b_w d$

$V_u d / M_u \le 1.0$

The derivation of this equation is obtained from the rudimentary analysis of the stresses at the head of a flexural crack to identify the significant parameters. Then the experimental results of reinforced concrete beams without web reinforcement were statistically analyzed to establish the constant 1/7 and 120/7. The data used in the statistical analysis included shot and slender beams; therefore mixing data from two different behavior types. Most of the data were the beam with high reinforcement ratio. The term $V_u d/M_u$ was represented the variable a/d.

In 1977, the ACI-ASCE Committee 424 on Shear and Diagonal Tension recommended that Equation (1.10) should no longer use. Macgregor (1992) suggested that Equation (1.10) underestimates the effect of tensile reinforcement ratio for beam without web reinforcement and is not entirely correct in its treatment of the variable a/d, expressed as $V_u d/M_u$ in Equation (1.10). For a number of reasons, the ACI-ASCE Committee 445 (1998) suggested that this equation is considered inappropriate.

For the normal range of variables, the second terms in the parentheses in Equation (1.10) is approximately equal to $0.008\sqrt{f_c'}$. Substituting this into Equation (1.10) gets,

$$V_c = \frac{\sqrt{f_c'}}{6} \cdot b_w d \tag{1.11}$$

Equation (1.11) intentionally presents the lower-bound average shear stress at diagonal crack. The ACI-ASCE Committee 445 (1998) suggested that Equation (1.11) is a reasonable lower-bound for slender beams that are not subject to axial load and have at least 1% longitudinal reinforcement. In conclusion, for members subject to shear and flexure, shear strength provided by concrete for nonprestressed members is computed by Equation (1.11).

For members subject to axial compression and tension, shear strength provided by concrete for nonprestressed member is expressed as follows respectively,

$$V_c = \left(1 + \frac{N_u}{14A_g}\right)\frac{\sqrt{f_c'}}{6} \cdot b_w d \qquad , N_u \geq 0 \tag{1.12}$$

$$V_c = \left(1 + \frac{0.3N_u}{A_g}\right)\frac{\sqrt{f_c'}}{6} \cdot b_w d \qquad , N_u < 0 \tag{1.13}$$

where N_u is axial load; to be taken as positive for compression, negative for tension

A_g is gross area of section

For circular cross section, web width is defined as diameter of circular section. The effective depth for circular section is the distance form extreme compression fiber to centroid of longitudinal tension reinforcement, but need not be less than the distance from extreme compression fiber to centroid of tension reinforcement in opposite half of member. It seems that there is no clear definition of effective depth for circular section.

<u>CIB-FIP Model Code (1990)</u>

The CEB-FIP Model Code (1990) suggests shear design equation causing shear cracking as follows,

$$V_c = 0.15(3d/a)^{1/3}(1 + \sqrt{200/d})(100\rho \cdot f_c')^{1/3} \cdot b_w d \tag{1.14}$$

Zsutty (1968, 1971)

The first attempt to derive empirical shear formula was due to Zsutty (1968). Using the data of 86 slender beams, Zsutty (1968) proposed the empirical equation based on dimensional and regression analysis as follows,

$$V_{c1} = 2.17\left(f_c'\rho \frac{d}{a} \right)^{1/3} b_w d \qquad ,a/d \geq 2.5 \tag{1.15}$$

Based on the experimental data for 108 short beams, Zsutty (1971) developed the slightly different formula for short shear span beam by multiplying the Equation (1.15) with [2.5/(a/d)], and gets

$$\mathrm{V}_{c2} = 5.43(f_c'\rho)^{1/3}(d/a)^{4/3} \qquad ,a/d < 2.5 \tag{1.16}$$

Ishibashi et al. (1983)

Ishibashi et al. (1983) had conduct the experiments for shear strength of reinforced concrete footing by using 1/5 scale-down specimen. From the experimental results of 37 footing test, Ishibashi et al. (1983) proposed the rational shear strength equation applicable for short to medium shear span member, since these types of footings behave like deep beam behavior.

$$V_c = 0.76 f_c'^{1/3}(a/d)^{-1.166}(1+\beta_p+\beta_d)\cdot b_w d \tag{1.17}$$

1.2 STATEMENT OF PROBLEMS AND RESEARCH OBJECTIVE

The current JSCE shear design equation for liner member without web reinforcement was originally formulated from the experimental results of reinforced concrete beam without web reinforcement. This equation is used both for beam and column members. However, there are two different, concerning shear strength, between reinforced concrete beams and columns member.

The first aspect is contribution of side reinforcement in column member. Column members are contained side reinforcements, which can treat as multi-layer of longitudinal bar, while beam members are contained only single layer of tensile reinforcements. As we know that longitudinal reinforcements are contributed for dowel force, during shear stresses transfer across the dowel bars. Therefore the appropriate amount of longitudinal reinforcement using in shear design equation is necessary to evaluate.

The second different between beams and columns is effect of section shape. The section shape for column members may be square, rectangular, octagonal or circular cross section, while the one for beam members is only rectangular or square cross section. The difference in section shape is arising in the problem of definition for effective depth and definition for effective concrete area. Since effective depth (d) is used in size effect function, a/d function, and calculation of tensile reinforcement (As/bd), different definition of effective depth is resulting in different shear strength. By the same way, different definition of effective concrete area is also resulting in different shear strength. To handle the effect of cross section, the current JSCE specification uses the concept of equivalent section to transform circular section into equivalent square section and treat equivalent square section as typical square section. By this concept of equivalent section, JSCE specification defined effective depth as the depth to centroil of longitudinal reinforcement in the portion of 90-degree. The longitudinal reinforcement contributed for dowel fore is defined as dowel bar in 90-degree portion.

By collecting experimental results of reinforced concrete circular column without transverse reinforcement, it was found that the ratio of experimental shear strength (Vexp) to calculated shear strength (Vcal) using current JSCE specification with including a/d function is average Vexp/Vcal=1.40 with coefficient of variation 15.5% (see Figure 1.2).

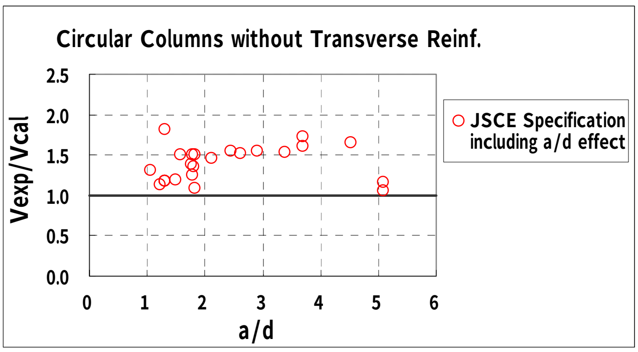

Figure 1.2 Comparison of shear strength calculated by current JSCE Specification including a/d effect to experimental results for circular column without transverse reinforcement

As the author reviews previously the development history of shear design equation before obtaining the current JSCE specification, this form of equation had already widely used and accepted. Therefore, the author had the rigid target to keep the pattern of current JSCE shear design equation and expanded this equation to column with side reinforcement and various cross sections by no changing the general form of current equation. In brief, the objectives of this research are as follows,

1) To obtain the shear design equation for reinforced concrete column with no changing the general form of current JSCE shear design equation
2) To take effect of side reinforcement into account
3) To take effect of section shape into account

1.4 RESEARCH STRATEGY AND CONTENTS

Since the objective of this research is to obtain the empirical formula, the analysis based on regression technique is appropriate and powerful. The overview of research strategy and contents of the dissertation are outlined in Figure 1.3.

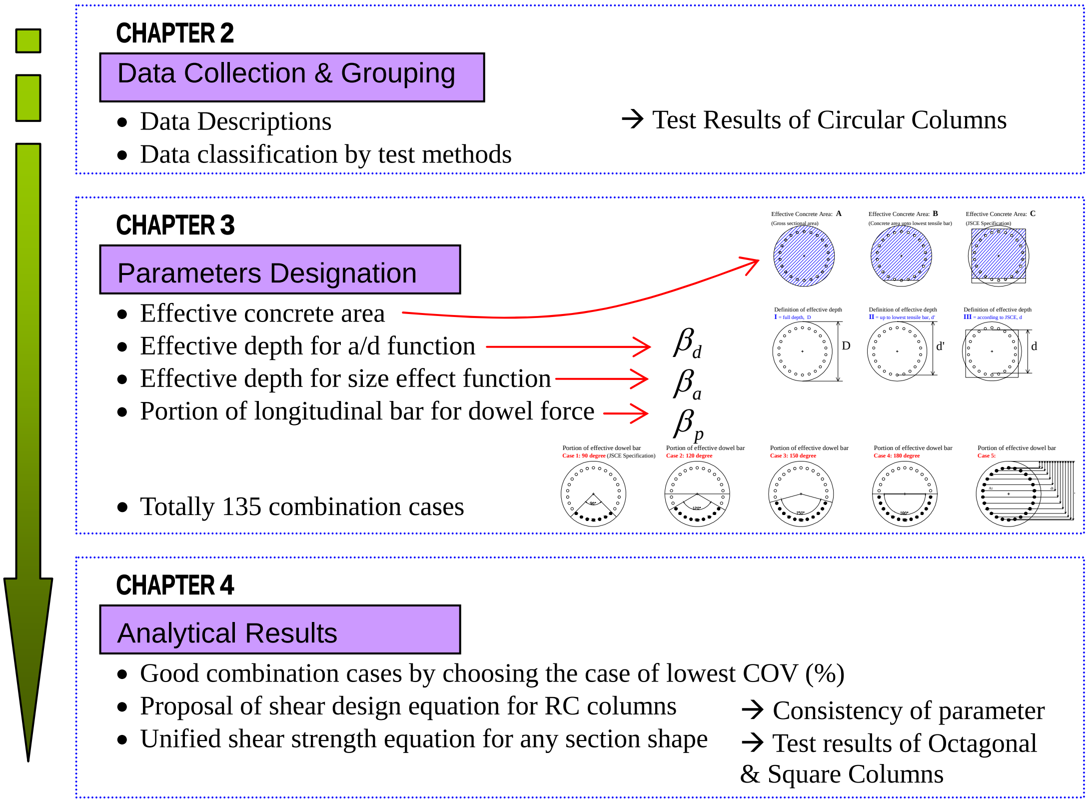

Figure 1.3 Research strategy and contents of the dissertation

The flow of research is started from data collection. The consistency of collected data is attempted to access by classifying the collected data according to test methods. The summary of data collection and grouping are briefly explained as follows,

Chapter 2: *This chapter described characteristic of collected experimental results of reinforced concrete column with using circular cross section. The geometries and properties of column specimens are typically designed according to target structure and experiment objective. The test methods for column specimen are typically conducted by cantilever setup, beam test, double curvature setup, or four-point loading in shear. To access the consistency of experimental results, the data were classified according to test method. If the biases from the experimental setup are low or negligible, the experimental results from different researchers at similar specimen geometry and same testing method should have the same tendency and consistency. The comparison of experimental and design shear strength according to current JSCE specification with adding a/d function were plotted. It was found that current JSCE specification with adding a/d effect is conservative results approximately Vexp/Vtest=1.40 with coefficient of variation 15.5%.*

The analysis is started from the elaborately consider the parameters affecting the shear strength when section shape is changed from rectangular like beam member. The case studies for each parameter were decided to cover extreme upper and lower limit. The summary of parameter designation are briefly explained as follows,

Chapter 3: *This chapter discussed the parameters affecting the shear strength of circular column. There first parameter is effective concrete area, which have three cases; gross area, concrete area up to lowest tensile bar, and area according current JSCE specification based on transform section concept. The second and third parameter is definition of effective depth, which is necessary to use in a/d and size effect function. There are three cases for this parameter; full depth, the distance up to lowest tensile reinforcement, and effective depth according to current JSCE specification. Finally the fourth parameter is portion of longitudinal reinforcement contributed for dowel force. There are five cases for this parameter; amount of tensile reinforcements in 90, 120, 150, and 180-degree portion, and the last case is the summation of longitudinal reinforcement at each lever multiplying with* *distance from extreme compression face to that level normalized by distance to lowest tensile reinforcement. There are totally 3*3*3*5=135 combination cases.*

Based on four parameters with totally 135 combination cases, the analytical results are briefly described and discussed in Chapter 4. The final form of proposal for shear strength of reinforced concrete column without web reinforcement was obtained by adjusting the analytical results. The unified shear equation for various section shapes were verified by experimental results of octagonal and square column. The details of Chapter 4 are summarized as follows,

Chapter 4: *This chapter described the analytical results from the 135 combination cases. Since the data are combined between short shear span specimen and slender specimen, the analytical results are then plotted separately at a/d=2.0. The combination case with less variation is preferable. However the good combination case for short column specimen may not the good one for slender column specimen. The good combination case for those two data ranges was decided. Hence the shear strength for circular RC column was obtained firstly. The proposal then was applied to octagonal RC column. The verification with experimental results showed good agreement. Finally the proposal was applied for square RC column. The comparison between the case of neglecting side reinforcement, case of consider side reinforcement in summation form, and case of effective reinforcement as half of total reinforcement, were performed. Finally unified shear strength equation for any cross section shape were obtained. The improvement in accuracy and mean were obtained comparing to the one using current JSCE.*

In summary, the flow of this research is started by collecting the experimental result of circular reinforced concrete column. The parameters affecting shear strength are elaborately considered for column case with various section shapes and containing side reinforcement. The case studies were decided for each parameter to cover upper and lower range. From the analytical result of 135 combination cases, the final form of proposal shear design equation for reinforced concrete column without transverse reinforcement was obtained. The unified shear design equation was verified by experimental results of octagonal and square column.

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Chapter 2

DATA COLLECTION AND CLASSIFICATION

2.1 DATA DESCRIPTIONS

The experiments of shear strength for reinforced column in the past are typically conducted for column with transverse reinforcement, since the test objective is mostly to evaluate ductility of column member under reversed cyclic loading. Experiments of reinforced concrete column with no or low transverse reinforcement are not much conducted in the past research. The geometry and properties of column specimens are typically varied depend upon target structure and test objective. The summary of collected experimental results assorting by structure type are described as follows,

<u>Reinforced Concrete Pile</u>

After Kobe Earthquake in 1995, many experiments on shear strength of reinforced concrete pile were conducted in Japan, since it was found that many reinforced concrete piles were damaged due to excessive lateral soil movement. These experimental results were referred to Yoshida et al. (1999), Kimura et al. (1998), Nagae et al. (1999), Arai et al. (2000), and Sako et al. (1999, 2000, 2001).

<u>Reinforced Concrete Building Column</u>

The columns for reinforced concrete building are typically low to medium slenderness, since the shear span is relatively short compare to its cross section. The both ends of building column are fixed with slab, thus columns are deformed under double curvature.

Hence, the specimens for column building are typically constructed as double curvature specimen. These experimental results were referred to Suzuki et al. (1988), and Kokusho et al. (1978).

Reinforced Concrete Bridge Pier

The columns for reinforced concrete bridge pier are typically medium to high slenderness with low axial stress and low longitudinal reinforcement. The diameter of bridge pier for real dimension is typically more than 1 meter, hence it is economical to use low longitudinal reinforcement with enough ductility. The specimens of bridge pier are constructed as cantilever column. Only one published experimental results were obtained, which conducted the test by Ang et al. (1985).

Reinforced Concrete Member with Circular Section

In this type of specimen, the target of experiment is to study the effect of section shape. At the early period of research in shear strength of reinforced concrete member, the Joint ASCE-ACI Task Committee 426 (1973) explained the effect of section shape by observing the experimental results conducted by Faradji and Dias de Cossio (1965). Originally this report was written in Spanish and translated into English by Portland Cement Association, Foreign Literature Study No. 466. Other experimental results were conducted by Kuroiwa and Okamoto (1999) and Yamada et al. (2003).

Totally 23 column specimens were collected from past researches. The summary of specimen geometry and experimental results were shown in Table 2.1 and specimen details were attached in Appendix A.

Table 2.1 Summary of experimental results of circular reinforced concrete column

No	Specimen	Test Setup	Concrete Strength fc' N/mm^2	Axial Load N KN	Axial Level N/Agfc'	Geometry: Clear Covering c mm	Geometry: Shear Span a mm	Geometry: Dia. D mm	Geometry: a/D	Longitudinal Reinf.: Reinf. Ratio ρ %	Longitudinal Reinf.: Yield fy N/mm^2	Longitudinal Reinf.: Bar Dia ϕ mm	Ultimate Shear Strength: Experiment kN	Ultimate Shear Strength: JSCE including a/d effect kN	Ultimate Shear Strength: Proposal kN	Vexp/Vcal: JSCE	Vexp/Vcal: Proposal
1	Ang et al. 1985, No. 25	Cantilever	32.8	0	0	15.00	600	400	1.50	3.20	296	20-D16	239	160	223	1.50	1.07
2	Yoshida 1999, No. 3	Cantilever	28.8	0	0	13.65	450	300	1.50	2.51	339	14-D13	113	82	118	1.38	0.96
3	Yamada 2003, C-000	Cantilever	25.8	530	0.29	12.05	450	300	1.50	3.37	436	12-D16	150	119	166	1.26	0.90
4	Kokusho et al. 1978, C-10-0	4-Point Loading	25.3	0	0	17.05	200	250	0.80	2.06	340	8-D13	117	89	139	1.32	0.84
5	Kokusho et al. 1978, C-15-0	4-Point Loading	25.2	0	0	17.05	300	250	1.20	2.06	340	8-D13	89	59	92	1.51	0.96
6	Kokusho et al. 1978, C-20-0	4-Point Loading	25.3	0	0	17.05	400	250	1.60	2.06	340	8-D13	64	44	69	1.45	0.93
7	Kokusho et al. 1979, C-0-0	4-Point Loading	20.1	0	0	21.15	300	250	1.20	3.10	827	12-D13	81	68	96	1.19	0.84
8	Sako 1999, SP-00 (SD295-00)	4-Point Loading	28.3	0	0	36.00	450	300	1.50	2.51	339	14-D13	107	78	106	1.36	1.01
9	Sako 2000, L90-00 (SD390-00)	4-Point Loading	26.9	0	0	36.00	450	300	1.50	3.37	426	12-D16	95	88	113	1.08	0.84
10	Sako 2000, L60-00	4-Point Loading	26.9	0	0	36.00	300	300	1.00	3.37	426	12-D16	148	132	170	1.13	0.87
11	Suzuki 1988, No. 1	Double Curvature	48.8	0	0	8.65	275	250	1.10	3.10	803	12-D13	127	108	150	1.18	0.85
12	Suzuki 1988, No. 4	Double Curvature	40.7	599	0.30	8.65	275	250	1.10	3.10	803	12-D13	265	146	202	1.82	1.31
13	Suzuki 1988, No. 9	Double Curvature	49.6	1460	0.60	8.65	275	250	1.10	3.10	803	12-D13	255	218	302	1.17	0.84
14	Nagae 1999, No. 1	Double Curvature	29.8	212	0	20.65	450	300	1.50	2.15	422	12-D13	141	94	131	1.51	1.08
15	Arai 2000, No. 3	Double Curvature	29.5	0	0	19.05	600	300	2.00	3.37	415	12-D16	114	73	97	1.55	1.18
16	Faradji et al. 1965, 25-3-C	Beam Test by 3 Point Loading	29.3	603	0.42	15.65	1050	251	4.18	3.07	406	12-D13	70	60	78	1.16	0.90
17	Faradji et al. 1965, 25-3-D	Beam Test by 3 Point Loading	34.2	571	0.34	15.65	1050	251	4.18	3.07	406	12-D13	66	63	82	1.05	0.81
18	Faradji et al. 1965, F-25-3-A	Beam Test by 3 Point Loading	29.0	0	0	15.65	700	251	2.79	3.07	406	12-D13	70	46	60	1.54	1.18
19	Faradji et al. 1965, F-25-3-B	Beam Test by 3 Point Loading	30.0	0	0	15.65	600	252	2.38	3.05	406	12-D13	76	49	64	1.55	1.18
20	Kimura 1988, No. 1	Beam Test by 3 Point Loading	27.3	0	0	118.00	2000	1000	2.00	2.22	371	22-D32	711	470	652	1.51	1.09
21	Fukushima 1992, No. 1	Beam Test by 3 Point Loading	19.3	0	0	92.05	1500	500	3.00	2.43	376	24-D16	170	102	143	1.65	1.18
22	Kuroiwa and Okamoto 1999, No. 1	Beam Test by 3 Point Loading	26.3	0	0	84.10	1900	700	2.71	4.13	545	20-D32	437	272	371	1.60	1.18
23	Kuroiwa and Okamoto 1999, No. 2	Beam Test by 3 Point Loading	27.0	0	0	88.50	1900	700	2.71	2.16	1004	20-D23	383	221	302	1.73	1.27

2.2 DATA CLASSIFICATIONS

To access the data consistency, all collected experimental results were classified by test methods. If bias from test setup is low or negligible, the experimental results of similar specimen geometry and properties tested under the same test method should show similar tendency. From the collected experimental results, there are four test setups for column specimen described as follow,

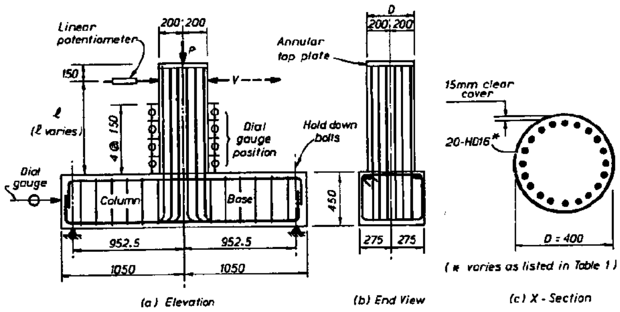
Cantilever Setup: This experimental setup is typically for specimen of civil structure including reinforced concrete bridge pier. The large and rigid footings are necessary for this test method.

Beam Setup: The experiments are conducted in beam test fashion. Since the specimen is circular cross section, there are two methods to place the support. The first method is cast the concrete stub at support position and test the specimen as usual reinforced concrete beam. Another method is making hemispherical support.

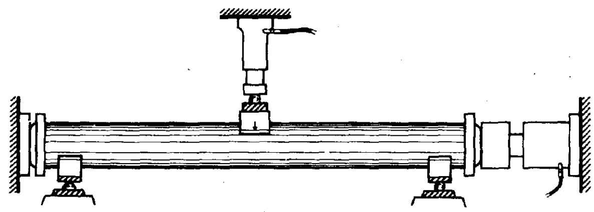
Double Curvature Setup: This is the typical test method for building column, which aim to maintain the axial load vertically, as similar as gravity force, throughout the experiment and test under double curvature setup. The large loading frames are necessary for this test method.

4-Point Loading in Shear: The experiments are conduct in beam test fashion. The big stubs were cast at both end of column specimen, which aims to obtain double curvature behavior. The supports are placed in diagonal pattern leading to induced shear force in shear span. The benefit of beam test fashion is no loading frame required.

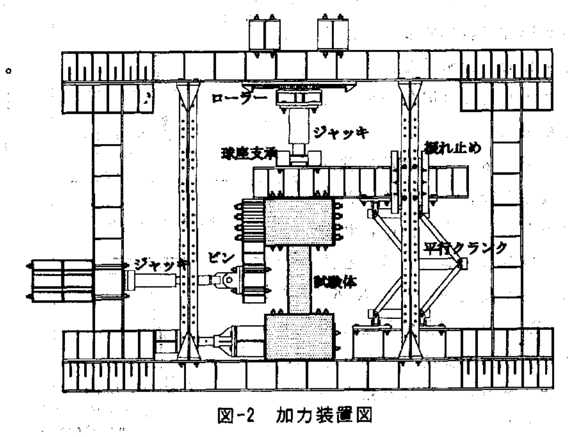
Figure 2.1 shows the experimental setup by various methods using in collected data.



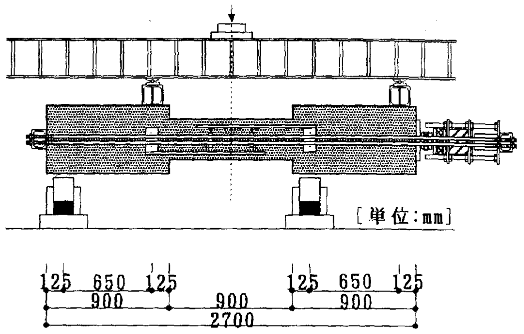
a) Cantilever Setup (Ang 1985)



b) Beam Setup (Faradji 1965)



c) Double Curvature Setup (Nagae 1999)



d) 4-Point Loading in Shear (Sako 1999, 2000)

Figure 2.1 Experimental setups for column specimens

To evaluate the accuracy of current JSCE specification for circular reinforced concrete member, the ratio of experimental results to calculated shear strength according to current JSCE specification including a/d effect were plotted in Figure 2.2~2.5 with parameter slenderness ration (a/d), axial load level (P/Agfc'), concrete compressive strength, and effective depth. Based on 23 collected specimens, it was found that the current JSCE specification is conservative results average Vexp/Vcal=1.40 with coefficient of variation 15.5%. It was implied that each parameters still need to refine to reach the higher accuracy.

Figure 2.2 shows the comparison of shear strength calculated by current JSCE specification with adding a/d effect. In this figure, three specimens, which conducted by different researchers showed very good consistency.

In Figure 2.3 and 2.4, the variations of calculated shear strength compared to experimental results are the case of specimen with high axial load. It was revealed that shear and axial load interaction is the complex behavior, which still needs further research.

In Figure 2.5, the good consistency of experimental results was obtained for specimen with comparable longitudinal reinforcement ratio.

Finally Figure 2.6 plots the all collected experimental results with all parameters; slenderness ratio, concrete compressive strength, effective depth, and longitudinal reinforcement ratio.

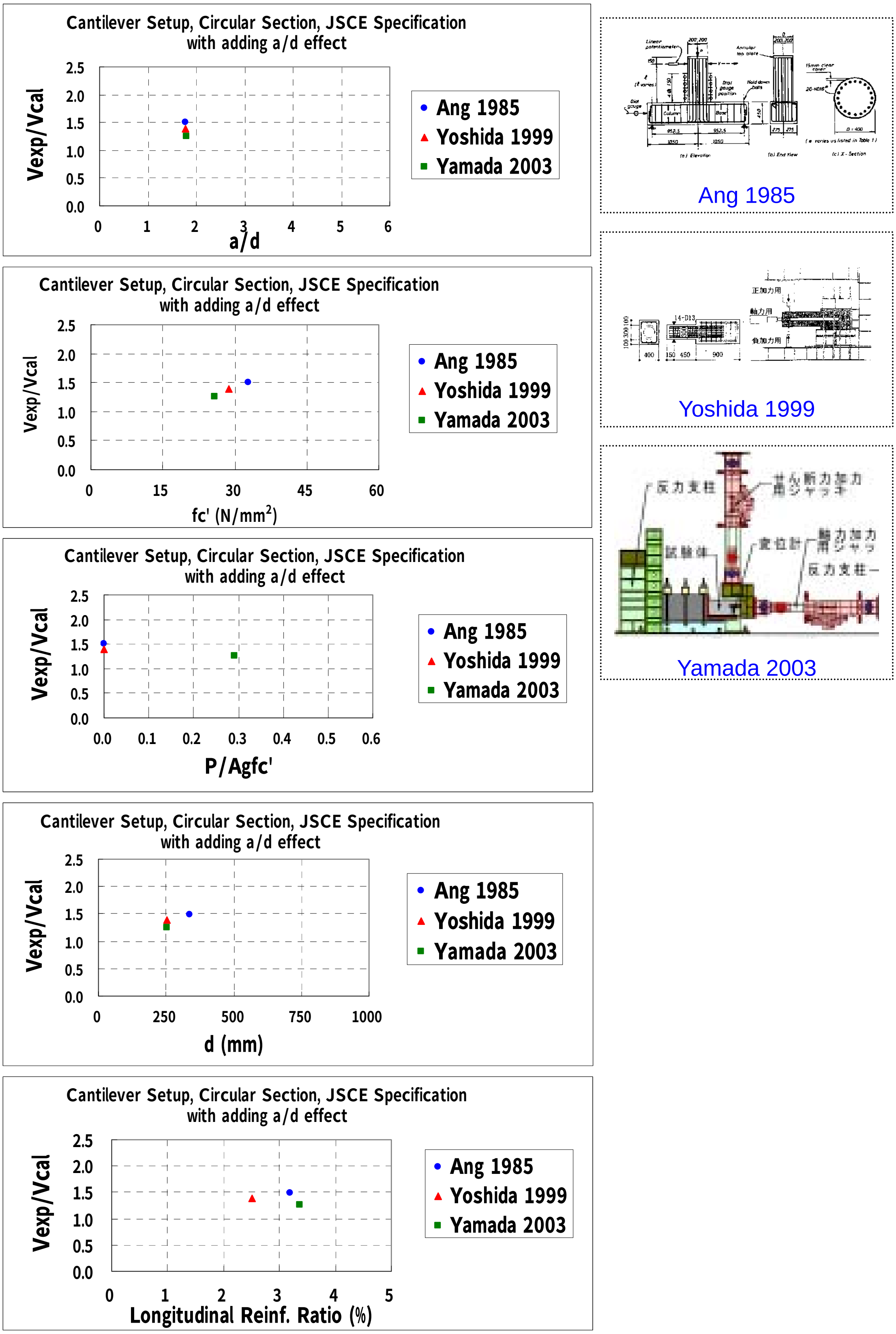


Figure 2.2 Comparison of shear strength according to current JSCE specification including a/d effect to experimental results testing by cantilever setup

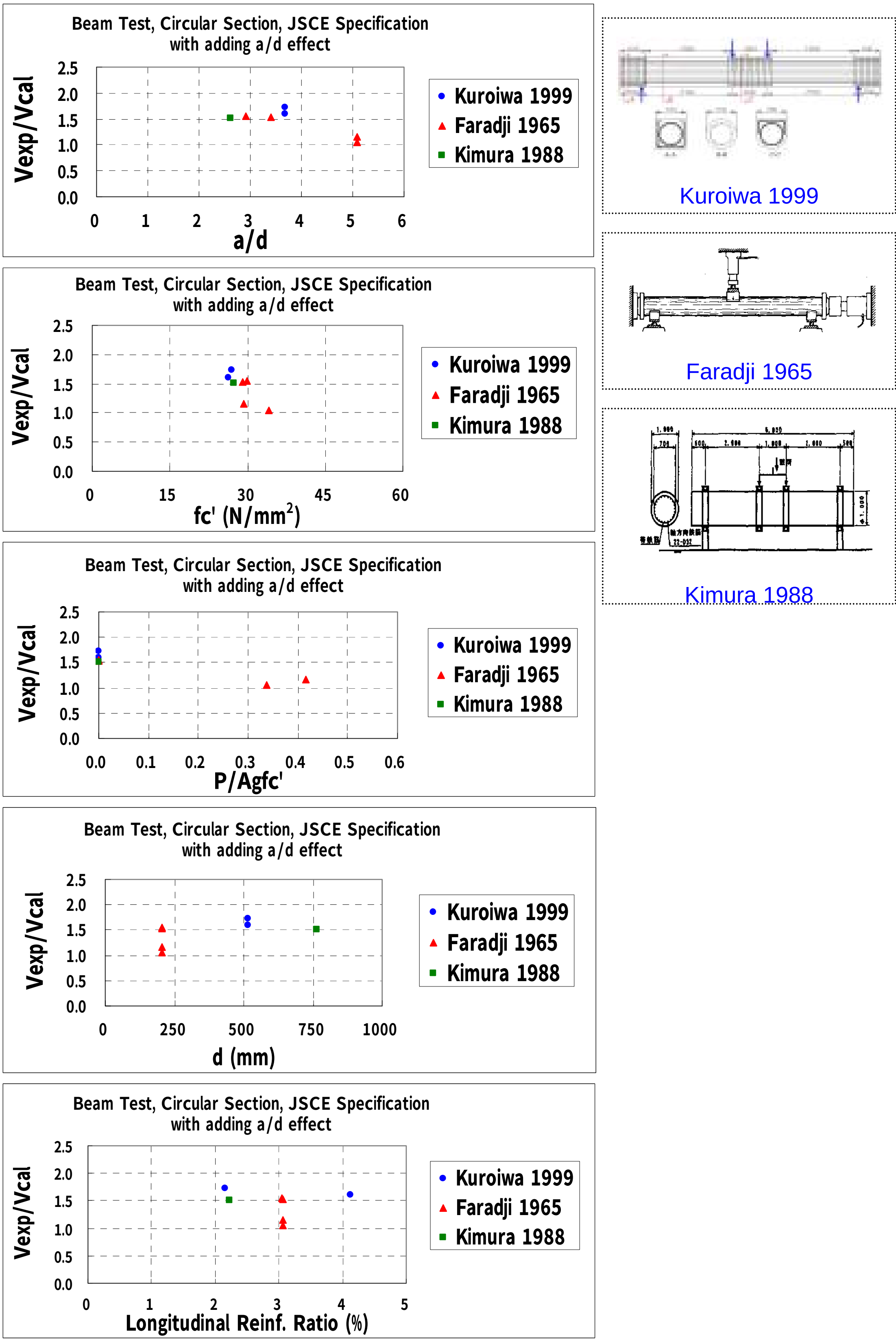


Figure 2.3 Comparison of shear strength according to current JSCE specification including a/d effect to experimental results testing by beam setup

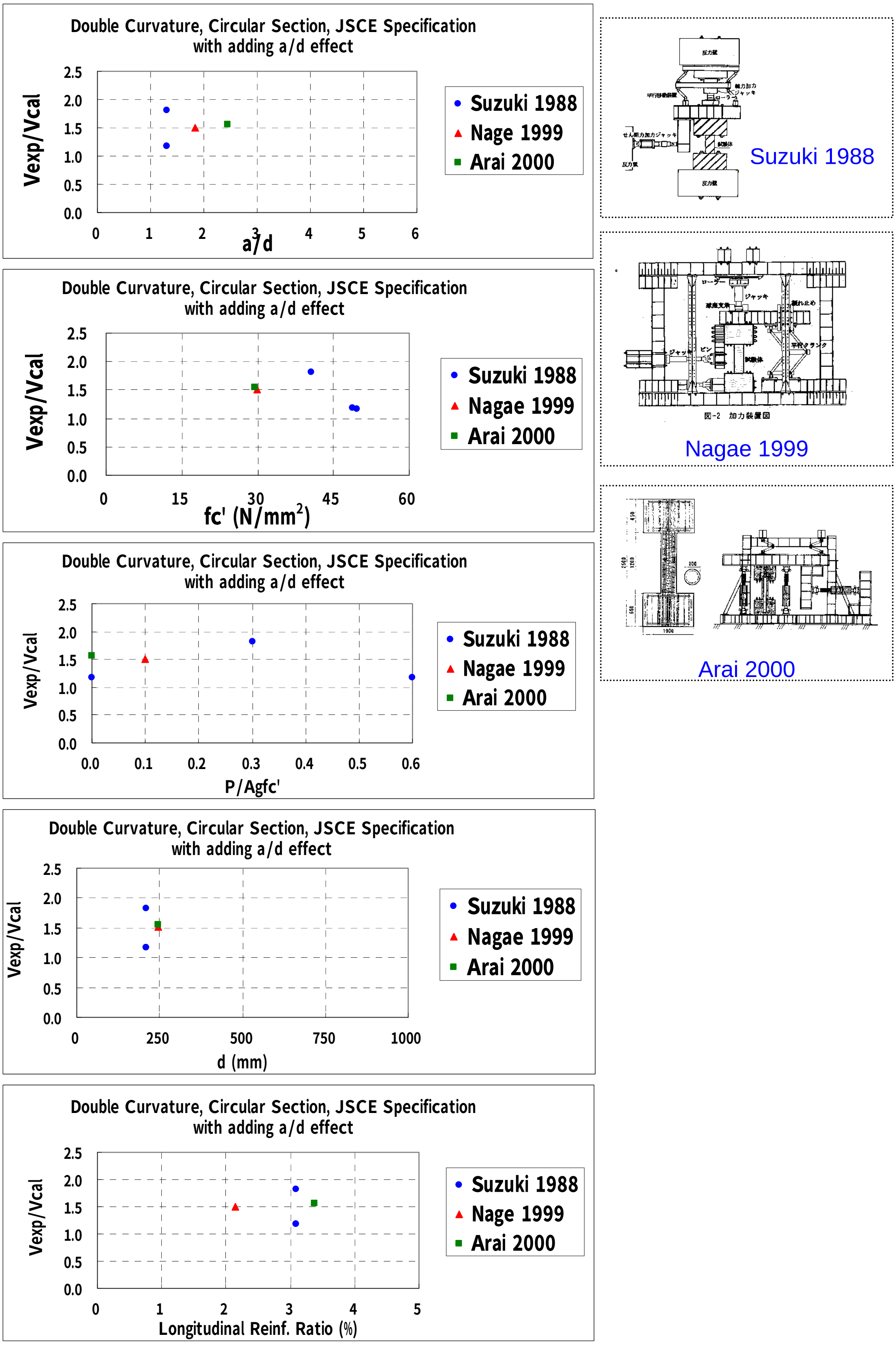


Figure 2.4 Comparison of shear strength according to current JSCE specification including a/d effect to experimental results testing by double curvature setup

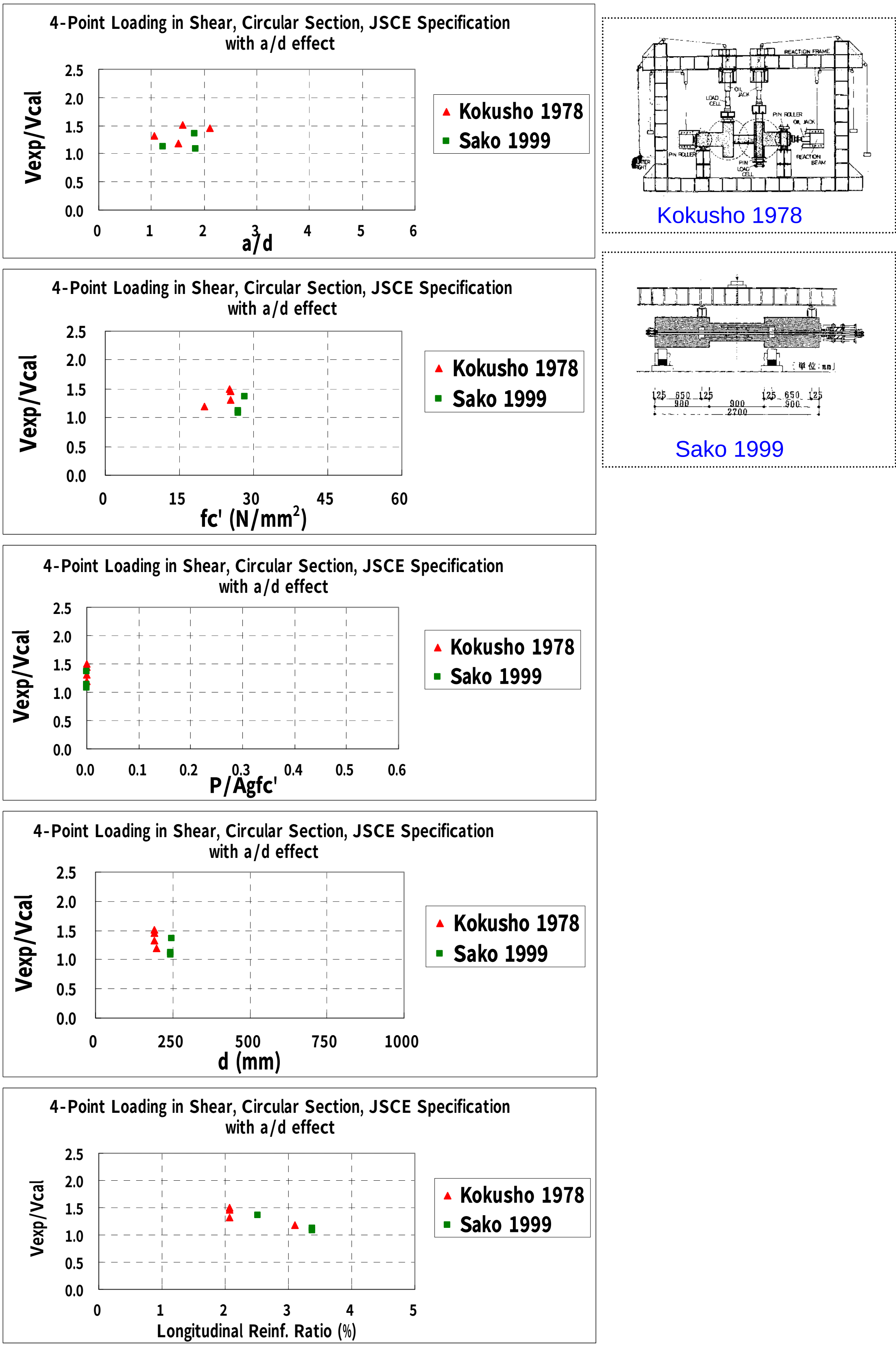


Figure 2.5 Comparison of shear strength according to current JSCE specification including a/d effect to experimental results testing by 4-point loading in shear

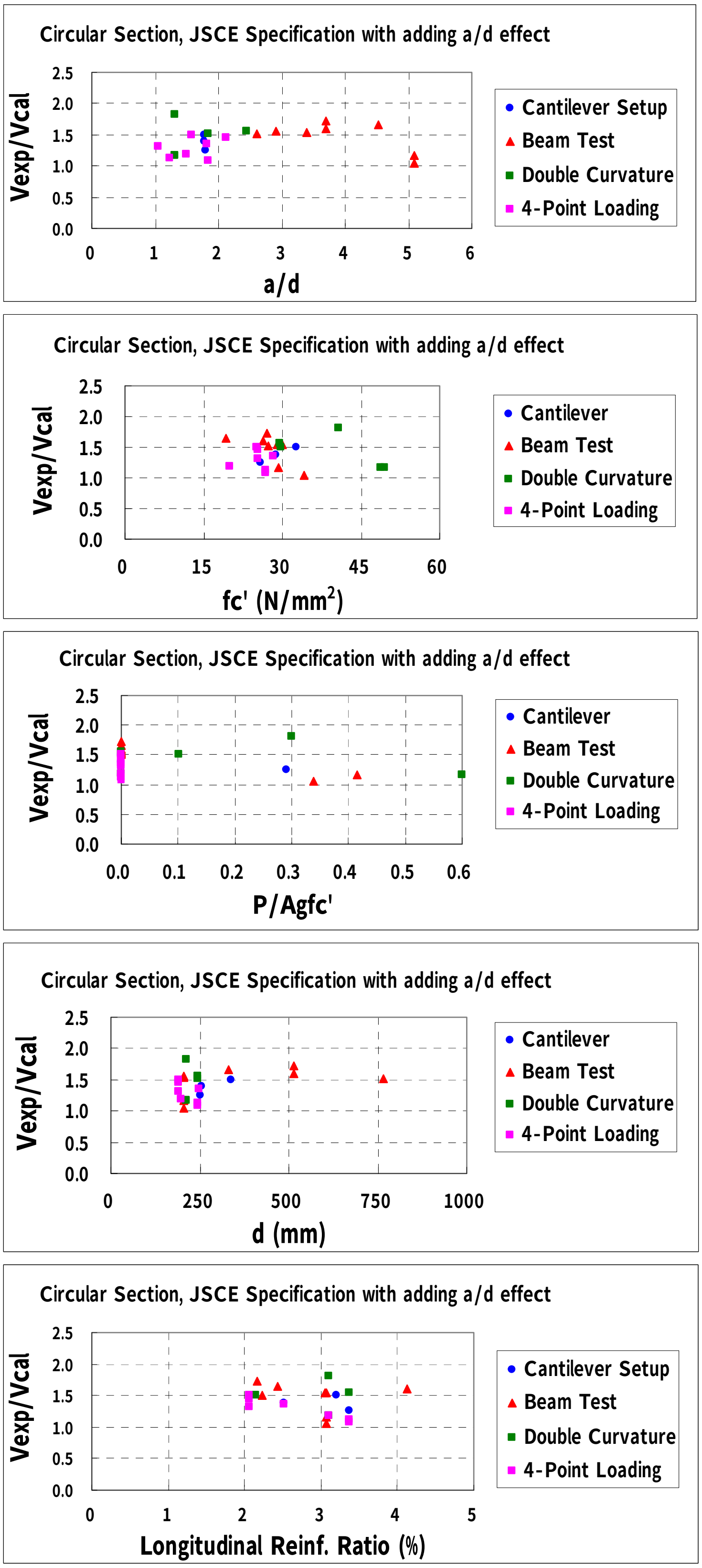


Figure 2.6 Comparison of shear strength according to current JSCE specification including a/d effect to experimental results for all specimens

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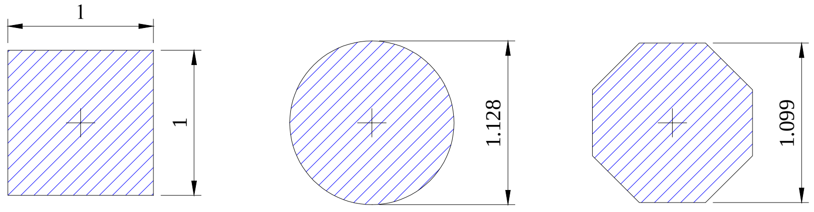
Chapter 3

DESIGNATION OF PARAMETERS IN ANALYSIS

3.1 EFFECT OF SECTION SHAPE ON SHEAR STREGNTH EQUATION

As review previously, the current JSCE shear design equation was formulated from experimental results of rectangular reinforced concrete beam. The effective depth according to this geometry was defined as the distance from the edge at compression side to tensile reinforcements. This effective depth is then use in calculation for a/d function, size effect function, tensile reinforcement ratio, and effective concrete area. For the case of non-uniformed section width like circular shape, the appropriate definition of effective depth is the main consideration from the view point of empirical formulation, since it affects many functions.

Figure 3.1 illustrates the unit gross area with three kind of section; square, circular, and octagonal section. It can be seen that under the same unit cross sectional area, the longest dimension for each case is different. Thus the appropriate definition of effective depth for the case of circular section is first parameter in analysis.



a) Square b) Circular c) Octogonal

Figure 3.1 Unit areas with various cross sections

Since the scale effect is described in function of effective depth, the changing in definition of effect depth is also affected size effect function. Hence, the definition of effective depth for size effect function is the second parameter in this analysis.

Accompany with definition of effective depth, effective concrete area (bd) is also affected when definition of effective depth is changed. Thus the effective concrete area is the third parameter in this analysis.

As the cross section is changed from rectangular to circular section, the longitudinal bars arrangement is also changed. Since longitudinal reinforcements are contributed for dowel force during shear forced transfer across the bar and controlling crack width, which imply the aggregate interlocking, the effective portion of tension bar used in calculation is the fourth parameter in analysis.

In summary, by considering all effects of section shape on shear strength equation, the parameters in this analysis are systematically decided as follows,

- Definition of effective depth for a/d function
- Definition of effective depth for size effect function
- Effective concrete area
- Portion of longitudinal reinforcement contribution for dowel force.

From the view point of member consideration, column members are different from beam member in three aspects,

1) Shape of section. The cross section for column member may square, rectangular, circular and octagonal section, while the one for beam member is typically rectangular section.
2) Placement of side reinforcement. Column members are typically contained side reinforcement, which contribute for dowel force and restraining crack width resulted in beneficial of aggregate interlocking, while beam members are typically no side reinforcement.
3) Source of axial load. The axial load for column members are directly came from self-weight of structure and live load in the gravitational direction, while the one for beam members are came from prestressed force.

3.2 PARAMETER 1: DEFINITION OF EFFECTIVE DEPTH FOR a/d FUNCTION

At short shear span member like short column and deep beam, the shear forces can transfer directly from loading point to bearing support in diagonal direction. The shear strength is increase inversed proportional to shear span-to-depth ratio and increase sharply for short shear span member.

Okamura and Higai (1980) had proposed a precise equation to describe effect of slenderness for slender member as follows,

$$\beta_a = 0.75 + 1.4(a/d)^{-1} \quad , \text{a/d} > 2.5 \tag{3.1}$$

For short reinforced concrete beam, Kennedy (1967) had proposed a/d effect with function of $(a/d)^{-1.166}$ and Zsutty (1971) had proposed the different function as $(a/d)^{-4/3}$. Ishibashi et al. (1983) had conducted the experiments for shear strength of reinforced concrete footing by using 1/5 scale-down specimen. From the experimental results of 37 footing test, Ishibashi et al. (1983) had concluded that $(a/d)^{-1.166}$ is the best fitting. Thus Ishibashi et al (1983) had proposed the shear design equation for reinforced concrete footing as follows,

$$V_c = 0.76 f_c'^{1/3} (a/d)^{-1.166} (1 + \beta_p + \beta_d) \cdot b_w d \tag{3.2}$$

By comparing with current JSCE shear design equation, the coefficient 0.76 can split into coefficient for concrete and a/d function as 0.76=0.20*3.8. Hence, the function of a/d solely is as follows,

$$\beta_a = 3.8(a/d)^{-1.166} \quad , \text{a/d} < 2.5 \tag{3.3}$$

JSCE Committee Report (1997) had proposed more conservative a/d function,

$$\beta_a = 3(a/d)^{-1} \quad , 0.5 < \text{a/d} < 2.0 \tag{3.4}$$

These three forms of a/d function are plotted in Figure 3.2. For footing specimen, the principle shear crack is in diagonal direction started from column fact to steel bearing support. Comparing to double curvature specimen at same a/d ratio, the principle shear crack is still in diagonal direction started from upper column fact to opposite lower column face. However, even both of specimen types are same a/d ratio, the failure angle of principal shear cracks are different. As the shear span is increase to a/d=2.0 for double curvature specimen, there are two separated principal shear cracks, which individual crack may similar to the crack from footing test.

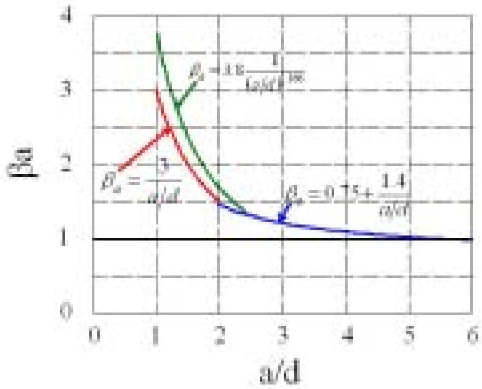


Figure 3.2 Effect of member slenderness on shear strength

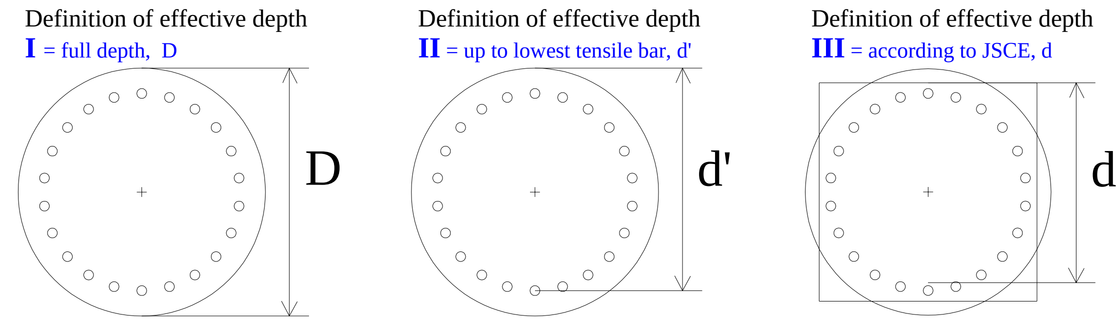
By considering the slenderness effect on shear strength of double curvature column at low a/d ratio, the conservative function as proposed by JSCE Committee Report (1997) was adopted in this analysis. In conclusion, the effect of slenderness on shear strength adopted in this analysis were rewritten as follows,

$$\beta_a = 0.75 + 1.4(a/d)^{-1} \qquad , \text{a/d} > 2.0 \qquad (3.5)$$

$$\beta_a = 3(a/d)^{-1} \qquad , \text{a/d} \leq 2.0 \qquad (3.6)$$

The definition of effective depth is the main consideration for a/d function applying to circular section. The upper limit for effective depth should be full section depth and lower limit should be effective depth according to current JSCE specification based on transform section concepts, since the current method of JSCE shear design equation for circular section show conservative results compare to experimental results. Between these two extreme cases, the author proposed the effective depth as the distance from compression face to lowest layer of tensile reinforcement. Therefore, there are three cases of definition of effective depth as follows,

Case I: Using full depth, D
Case II: The depth up to lowest tensile bar, d'
Case III: Effective depth according to JSCE specification based on transformed section



$$\beta_a = 0.75 + 1.4(a/D)^{-1}, a/D > 2.0$$
$$\beta_a = 3(a/D)^{-1}, a/D \le 2.0$$

$$\beta_a = 0.75 + 1.4(a/d')^{-1}, a/d' > 2.0$$
$$\beta_a = 3(a/d')^{-1}, a/d' \le 2.0$$

$$\beta_a = 0.75 + 1.4(a/d)^{-1}, a/d > 2.0$$
$$\beta_a = 3(a/d)^{-1}, a/d \le 2.0$$

Figure 3.3 Parameters for a/d function

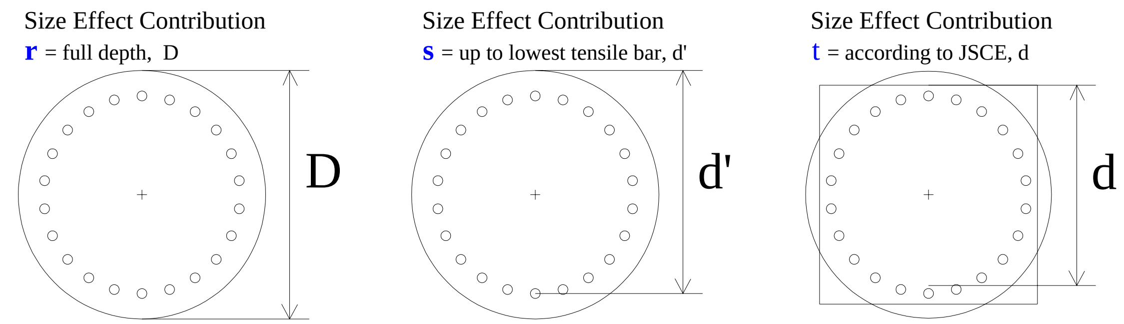
3.3 PARAMETER 2: DEFINITION OF EFFECTIVE DEPTH FOR SCALE FUNCTION

Consequently from three definitions of effective depth used in a/d function, those three cases are also adopted in size effect as follows,

Case r: Using full depth, D

Case s: The depth up to lowest tensile bar, d’

Case t: Effective depth according to JSCE specification based on transformed section



$$\beta_d = \sqrt[4]{1000/D}$$
$$\beta_d = \sqrt[4]{1000/d'}$$
$$\beta_d = \sqrt[4]{1000/d}$$

Figure 3.4 Parameters for size effect function

3.4 PARAMETER 3: EFFECTIVE CONCRETE AREA

The upper limit of effective concrete area is fully gross area. Typically effective concrete area is defined as area over effective depth. JSCE specification uses concept of transform section from circular to square section, while effective depth is defined as the depth up to centroid of tensile reinforcement arranged in 90degree portion. This effective depth seem too small compare to fully depth. Therefore, the author proposes the effective depth between this two ranges and defined effective depth as the depth up to lowest tensile reinforcement.

The parameters of effective concrete area have three cases as here after:

Case A: Gross sectional area

Case B: Concrete area up to lowest tensile bar

Case C: Effective concrete area according to JSCE specification

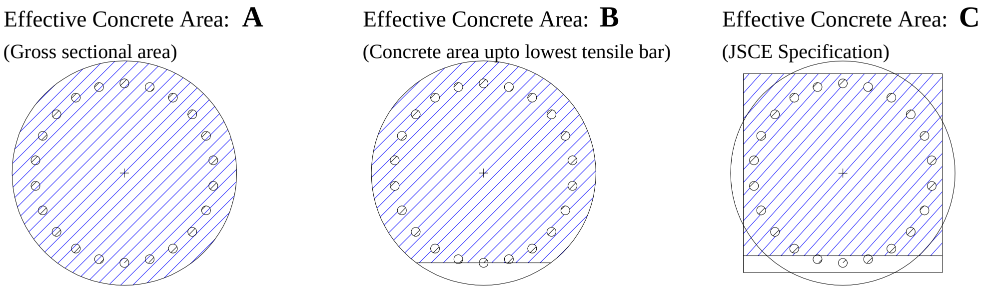


Figure 3.5 Parameters for effective concrete area

The area of circular segment can be calculated as hereafter,

$$\theta = \cos^{1}\left(\frac{D/2 - a}{D/2}\right) \quad \text{, in radian} \tag{3.7}$$

$$A = D^2\left(\frac{\theta - \sin\theta \cdot \cos\theta}{4}\right) \tag{3.8}$$

where θ is angle from vertical axis (see Figure 3.6)

A is segment area

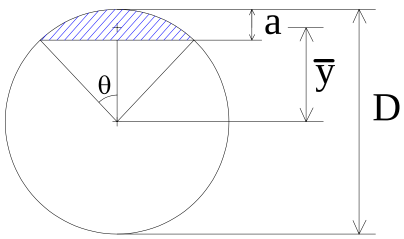


Figure 3.6 Linear algebra for circular segment

3.5 PARAMETER 4: EFFECTIVE LONGITUDINAL REINFORCEMENT

The experiment of reinforced concrete beams width multi-layer of longitudinal reinforcement had conducted by Ishibashi et al. (1985b), and Matsuyoshi and Machida (1987). It was found that the position of longitudinal reinforcement has relationship with increment of shear strength. To accumulate reinforcement effect for all layers, Ishibashi et al. (1985b) had proposed the summation of reinforcement at each layer multiplying with distance from compression face to that layer and normalize by the distance up to lowest tension bar. This form of equation, in fact, is applied the concept of linear interpolation.

$$A_s = \sum \left(A_i \, d_i / d_1 \right) \qquad (3.9)$$

where d_i is distance from compression face to each layer of longitudinal reinforcement
d_1 is distance from compression face to lowest tensile reinforcement

By considering the current JSCE specification, which account tensile bar arranging in 90-degree portion, in this analysis the effective dowel bar is expanded to 120, 150 and 180-degree portion. The method as proposed by Ishibashi et al. (1985b) is also included.

Case 1: Dowel Bar in 90-degree portion
Case 2: Dowel Bar in 120-degree portion
Case 3: Dowel Bar in 150-degree portion
Case 4: Dowel Bar in 150-degree portion
Case 5: Summation form as proposed by Ishibashi et al. (1985b)

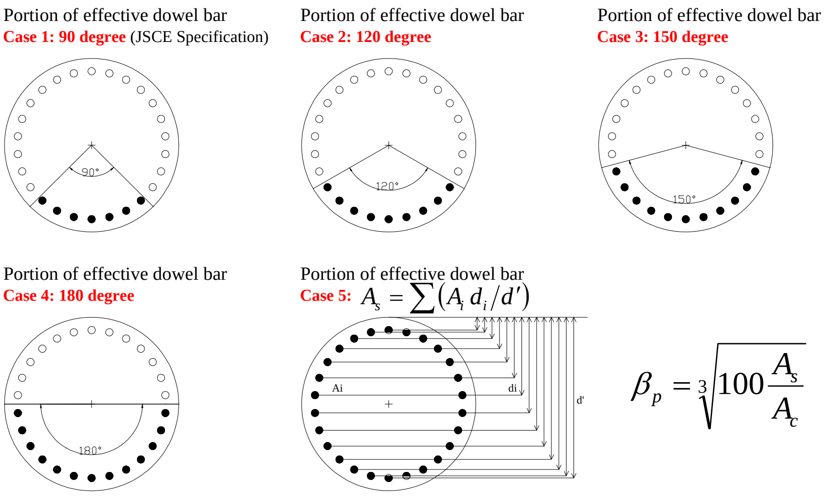


Figure 3.9 Parameters for portion of dowel bar

3.6 EFFECT OF AXIAL LOAD ON SHEAR STRENGTH

For design purpose, the current JSCE shear design equation is accounted the axial load effect by the following functions,

$$\beta_n = 1 + M_o / M_d \qquad \leq 2.0 \quad (N_d \geq 0) \qquad (3.10)$$

where M_o is decompression moment to cancel extreme axial stress
M_d is design moment
N_d is design axial load (positive sign convention for compression force).

Moreover, JSCE specifications also suggest that $\beta_n = 1 + 2M_o / M_u$ may show better results compared with experimental results in laboratory. Hence for analysis purpose, this equation was adopted in analysis.

$$\beta_n = 1 + 2M_o / M_u \qquad \leq 2.0 \quad (N_d \geq 0) \qquad (3.11)$$

where M_u is ultimate moment

The ultimate moment was calculated from linear strain distribution over section at the extreme concrete compressive strain ε_c=0.0035. Decompression moment, M_o, can be calculated from relationship of stress distribution over section, $\sigma_o = M_o y / I$, where σ_o is axial stress, y is distance at extreme fiber from neutral axis, and I is moment of inertia. Moment of inertia is changed according to cross section that is $I=bh^3/12$ for rectangular section, $I=\pi D^4/64$ for circular section, and $I = (8\sqrt{2}+11)S^4/12$ for octagonal section. By substituting moment of inertia and distance at extreme fiber, the decompression moment can be calculated as follows,

$$M_o = \sigma_o \cdot \frac{1}{6} bh^2 \qquad \text{, for rectangular section} \qquad (3.12)$$

$$M_o = \sigma_o \cdot \frac{1}{32} \pi D^3 \qquad \text{, for circular section} \qquad (3.13)$$

$$M_o = \sigma_o \cdot \frac{5+3\sqrt{2}}{6} S^3 \qquad \text{, for octagonal section} \qquad (3.14)$$

where σ_o is axial compressive stress
S is a side length of octagonal section

3.7 COMBINATION CASES FOR ALL PARAMTERS

From four parameters in this analysis, there are totally 3*3*3*5=135 combination cases as follows,

Table 3.1 List of combination case

Case	Parameter	Case	Parameter	Case	Parameter
1	A1-I-r	46	B1-I-r	91	C1-I-r
2	A1-I-s	47	B1-I-s	92	C1-I-s
3	A1-I-t	48	B1-I-t	93	C1-I-t
4	A1-II-r	49	B1-II-r	94	C1-II-r
5	A1-II-s	50	B1-II-s	95	C1-II-s
6	A1-II-t	51	B1-II-t	96	C1-II-t
7	A1-III-r	52	B1-III-r	97	C1-III-r
8	A1-III-s	53	B1-III-s	98	C1-III-s
9	A1-III-t	54	B1-III-t	99	C1-III-t
10	A2-I-r	55	B2-I-r	100	C2-I-r
11	A2-I-s	56	B2-I-s	101	C2-I-s
12	A2-I-t	57	B2-I-t	102	C2-I-t
13	A2-II-r	58	B2-II-r	103	C2-II-r
14	A2-II-s	59	B2-II-s	104	C2-II-s
15	A2-II-t	60	B2-II-t	105	C2-II-t
16	A2-III-r	61	B2-III-r	106	C2-III-r
17	A2-III-s	62	B2-III-s	107	C2-III-s
18	A2-III-t	63	B2-III-t	108	C2-III-t
19	A3-I-r	64	B3-I-r	109	C3-I-r
20	A3-I-s	65	B3-I-s	110	C3-I-s
21	A3-I-t	66	B3-I-t	111	C3-I-t
22	A3-II-r	67	B3-II-r	112	C3-II-r
23	A3-II-s	68	B3-II-s	113	C3-II-s
24	A3-II-t	69	B3-II-t	114	C3-II-t
25	A3-III-r	70	B3-III-r	115	C3-III-r
26	A3-III-s	71	B3-III-s	116	C3-III-s
27	A3-III-t	72	B3-III-t	117	C3-III-t
28	A4-I-r	73	B4-I-r	118	C4-I-r
29	A4-I-s	74	B4-I-s	119	C4-I-s
30	A4-I-t	75	B4-I-t	120	C4-I-t
31	A4-II-r	76	B4-II-r	121	C4-II-r
32	A4-II-s	77	B4-II-s	122	C4-II-s
33	A4-II-t	78	B4-II-t	123	C4-II-t
34	A4-III-r	79	B4-III-r	124	C4-III-r
35	A4-III-s	80	B4-III-s	125	C4-III-s
36	A4-III-t	81	B4-III-t	126	C4-III-t
37	A5-I-r	82	B5-I-r	127	C5-I-r
38	A5-I-s	83	B5-I-s	128	C5-I-s
39	A5-I-t	84	B5-I-t	129	C5-I-t
40	A5-II-r	85	B5-II-r	130	C5-II-r
41	A5-II-s	86	B5-II-s	131	C5-II-s
42	A5-II-t	87	B5-II-t	132	C5-II-t
43	A5-III-r	88	B5-III-r	133	C5-III-r
44	A5-III-s	89	B5-III-s	134	C5-III-s
45	A5-III-t	90	B5-III-t	135	C5-III-t

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Chapter 4

ANALYTICAL RESULTS OF SHEAR STRENGTH FOR REINFORCED CONCRETE COLUMNS

4.1 ANALYTICAL RESULTS FOR ALL COMBINATION CASES

There are two mechanisms namely beam action and arch action providing the shear strength for slender and short beam/column. These two mechanisms were taken into account empirically by a/d function. Therefore to consider the tendency of accuracy, the analysis were separate between slender and short columns at a/d=2. The all analytical results were plot in term of coefficient of variation (COV) as shown in Figure 4.1. The combination case with lowest COV is preferable. However, the combination case with lowest COV for data group a/d < 2 may not the one for data group a/d >2. The details of mean and COV for combination case were shown in Table 4.1.

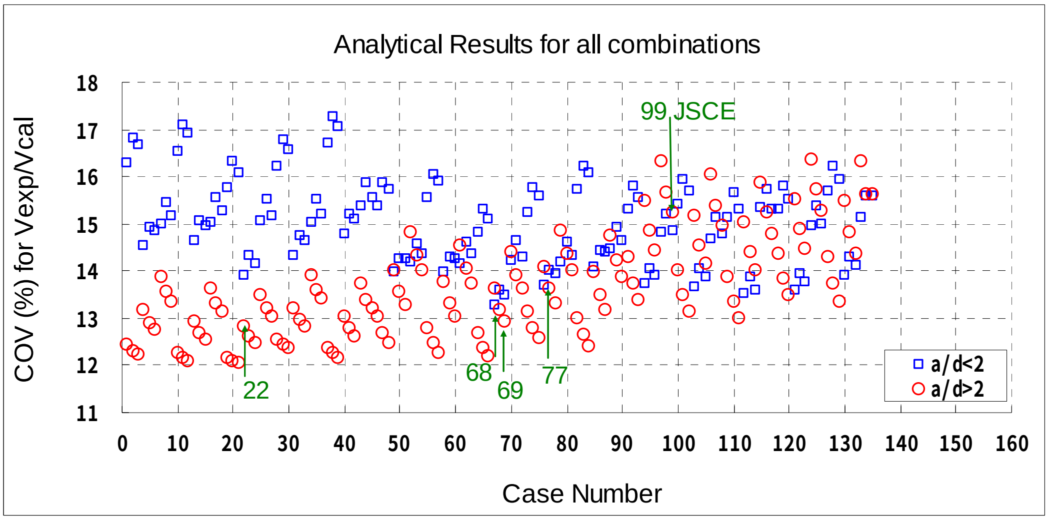


Figure 4.1 Analytical results for all combination cases

Table 4.1 Analytical results for combination cases sorting by coefficient of variation

		a/d < 2	
	Case No	Average Vtest/Vcal	COV %
	67	1.038	13.280
	69	0.988	13.481
	112	1.064	13.511
	121	0.997	13.587
	68	1.014	13.592
	114	1.013	13.606
	103	1.126	13.658
	76	0.973	13.685
	94	1.265	13.715
	123	0.949	13.760
	113	1.039	13.859
	105	1.072	13.878
	96	1.205	13.910
	130	0.967	13.915
	22	1.005	13.922
	78	0.927	13.951
	122	0.973	13.952
	58	1.100	13.969
	49	1.236	13.979
(Proposal)	77	0.950	14.006
(JSCE)	99	1.345	14.863

		a/d > 2	
	Case No	Average Vtest/Vcal	COV %
	21	1.028	12.047
	20	1.061	12.089
	12	1.111	12.099
	11	1.147	12.153
	19	1.095	12.165
	39	0.939	12.166
	66	1.078	12.184
	3	1.229	12.233
	10	1.184	12.247
	38	0.969	12.248
	57	1.165	12.271
	2	1.269	12.308
	30	0.972	12.352
	37	1.000	12.370
	65	1.113	12.378
	84	0.985	12.412
	29	1.004	12.430
	1	1.310	12.433
	24	1.080	12.462
	48	1.289	12.466
	56	1.203	12.475
	28	1.036	12.543
	15	1.167	12.547
	75	1.020	12.562
	23	1.115	12.593
	42	0.986	12.601
	83	1.017	12.647
	47	1.332	12.685
	14	1.205	12.688
	64	1.149	12.691
	6	1.292	12.737
	41	1.019	12.772
	74	1.053	12.785
	55	1.242	12.801
	33	1.022	12.805
	22	1.151	12.812
	5	1.334	12.896
	69	1.133	12.912
	13	1.244	12.923
	32	1.055	12.967
	82	1.050	13.007
	111	1.125	13.007
	60	1.225	13.026
	46	1.375	13.034
	40	1.052	13.038
	27	1.131	13.044
	73	1.088	13.123
	102	1.216	13.128
	18	1.222	13.144
	4	1.377	13.156
	87	1.035	13.159
	68	1.170	13.187
	45	1.033	13.190
	26	1.168	13.212
	31	1.089	13.216
	51	1.356	13.263
	59	1.265	13.308
	78	1.072	13.311
	17	1.263	13.322
	110	1.162	13.347
	129	1.028	13.356
	9	1.353	13.358
	93	1.346	13.378
	44	1.067	13.397
	36	1.070	13.406
	101	1.256	13.474
	86	1.069	13.474
	120	1.065	13.474
	25	1.206	13.488
	8	1.398	13.551
	50	1.400	13.557
	35	1.105	13.604
	16	1.304	13.610
(Proposal)	77	1.108	13.610
(JSCE)	99	1.483	15.230

There are many combination patterns to obtain less variation. Among these forms, there are 4 combination cases that shown good results for all range of data (see Table 4.2). It is surprisingly that these 4 cases use the same definition of effective depth for a/d function (case II). For simplicity and consistency in practice, the effective depth for size effective function should use the same definition as the one for a/d function (case S). There are two cases that use parameter II-S which are case number 68 and 77. Both of the case number 68 and 77 use the same effective concrete area (case B). Therefore the last parameter is only effective longitudinal reinforcement. By consider the ease of use, the case number 77, which occupied the half of longitudinal reinforcement for effective reinforcement, was decided. The proposal of case number 77 still show improvement in mean and variation compare to current JSCE specification. Figure 4.2 illustrate the parameters for good combination including current JSCE specification.

Table 4.2 Summary of good combination case for all range of data

Case	Parameter	a/d < 2		a/d > 2	
		Average Vtest/Vcal	COV %	Average Vtest/Vcal	COV %
22	A3-II-r	1.01	13.92	1.03	12.81
68	B3-II-s	1.01	13.59	1.21	13.19
69	B3-II-t	0.99	13.48	1.17	12.91
77 (Proposal)	B4-II-s	0.95	14.01	1.11	13.61
99 (JSCE)	C1-III-t	1.35	14.86	1.48	15.23

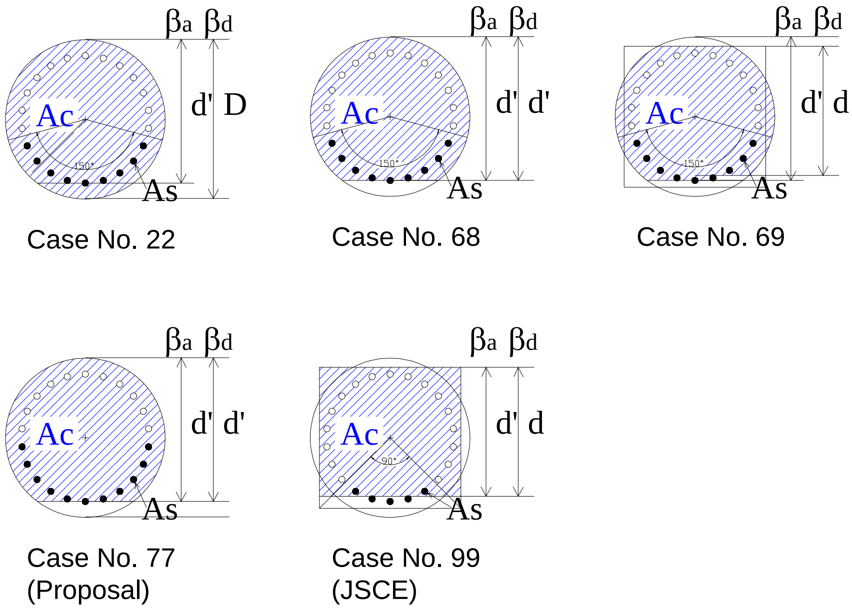


Figure 4.2 Illustration of parameters for good combination case

4.2 PROPOSAL OF SHEAR STRENGTH EQUATION FOR CIRCULAR REINFORCED CONCRETE COLUMN

By considering the consistency of parameter in practice, the author proposes that the effective concrete area is defined as the area above lowest tensile reinforcement, and the effective depth is defined as the distance from compression face up to lowest layer of tensile reinforcement and used it both in a/d and size effect function. For simplicity in practice, the author proposes effective longitudinal reinforcement as the half of total longitudinal reinforcement (see Figure 4.3).

$$V_c = 0.20 \cdot \sqrt[3]{f_c'} \cdot \beta_p \cdot \beta_d \cdot \beta_n \cdot \beta_a \cdot A_c \qquad (4.1)$$

$$\beta_p = \sqrt[3]{100 P_w} \qquad \leq 1.50 \qquad (4.2)$$

$$\beta_d = \sqrt[4]{1000/d} \qquad \leq 1.50 \qquad (4.3)$$

$$\beta_n = 1 + 2M_o / M_u \qquad \leq 2.0 \quad \text{(in compression)} \qquad (4.4)$$

$$\beta_a = 0.75 + 1.4/(a/d)^{-1} \qquad , \text{a/d} > 2.0 \qquad (4.5a)$$

$$\beta_a = 3 \cdot (a/d)^{-1} \qquad , \text{a/d} \leq 2.0 \qquad (4.5b)$$

The parameter using in proposal are defined as follow:

A_c : effective concrete area defined as area above lowest layer of tensile reinforcement

A_s : effective longitudinal reinforcement defined as half of total reinforcement

d : effective depth defined as distance from compression face to lowest layer of tensile reinforcement

$P_w = A_s/A_c$

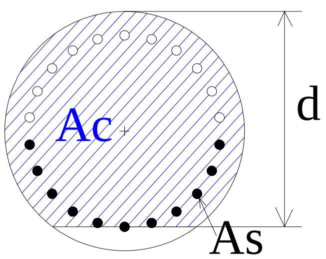


Figure 4.3 Detail parameter of proposal for circular column

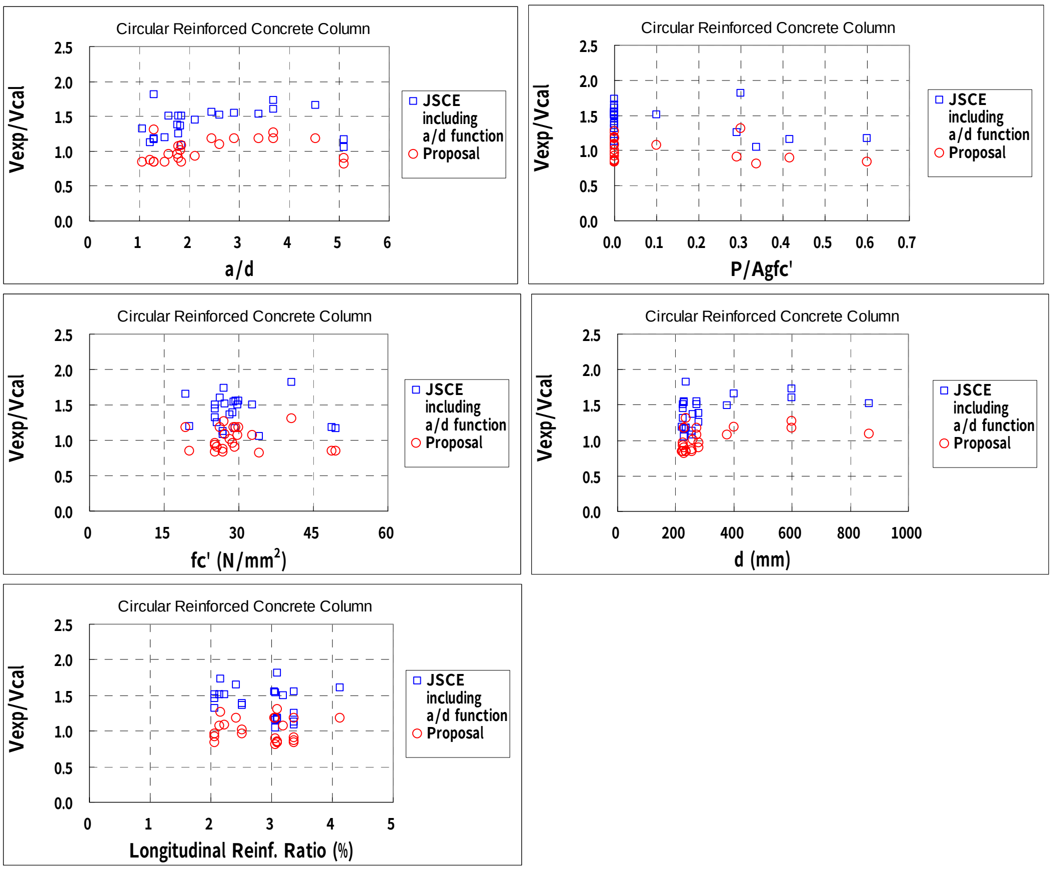
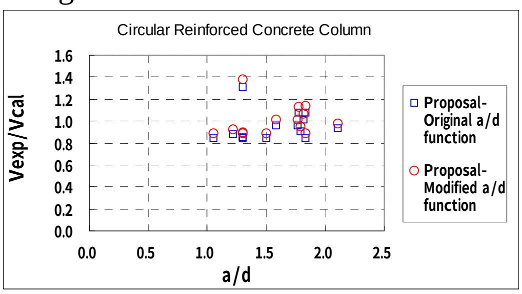


Figure 4.3 Verification of proposal for circular reinforced concrete columns

The verifications of proposal with experimental results are shown in Figure 4.3. The improvement in variation and mean are obtained compare to current JSCE specification. However, for the case of short columns (a/d < 2.0), the proposal show overestimate with mean Vexp/Vcal=0.95. Therefore to overcome overestimate mean value, the a/d function for short shear span range was modified as follow,

<u>Modified a/d function:</u> $$\beta_a = 2.85 \cdot (a/d)^{-1} \qquad ,a/d \leq 2.0 \qquad (4.6)$$

By using modified a/d function, the mean value of Vexp/Vcal was improved to 1.00. The comparison between calculation using original and modified a/d function are shown in Figure 4.4.



Case	a/d function	Parameter	a/d < 2	
			Average Vtest/Vcal	COV %
Proposal	original	B4-II-s	0.95	14.01
JSCE	original	C1-III-t	1.35	14.86
Proposal	Modified	B4-II-s	1.00	14.01
JSCE	Modified	C1-III-t	1.42	14.86

Figure 4.4 Comparison between calculation using original and modified a/d function

Table 4.3 Summary of column properties for circular reinforced concrete column

No	Specimen	Test Setup	Concrete Strength	Axial Load	Axial Level	Geometry				Longitudinal Reinf.			Ultimate Shear Strength			Vexp/Vcal	
			fc' N/mm^2	N KN	N/Agfc'	Clear Covering c mm	Shear Span a mm	Dia. D mm	a/D	Reinf. Ratio ρ %	Yield fy N/mm^2	Bar Dia φ mm	Experiment kN	JSCE including a/d effect kN	Proposal kN	JSCE	Proposal
1	Ang et al. 1985, No. 25	Cantilever	32.8	0	0	15.00	600	400	1.50	3.20	296	20-D16	239	160	223	1.50	1.07
2	Yoshida 1999, No. 3	Cantilever	28.8	0	0	13.65	450	300	1.50	2.51	339	14-D13	113	82	118	1.38	0.96
3	Yamada 2003, C-000	Cantilever	25.8	530	0.29	12.05	450	300	1.50	3.37	436	12-D16	150	119	166	1.26	0.90
4	Kokusho et al. 1978, C-10-0	4-Point Loading	25.3	0	0	17.05	200	250	0.80	2.06	340	8-D13	117	89	139	1.32	0.84
5	Kokusho et al. 1978, C-15-0	4-Point Loading	25.2	0	0	17.05	300	250	1.20	2.06	340	8-D13	89	59	92	1.51	0.96
6	Kokusho et al. 1978, C-20-0	4-Point Loading	25.3	0	0	17.05	400	250	1.60	2.06	340	8-D13	64	44	69	1.45	0.93
7	Kokusho et al. 1979, C-0-0	4-Point Loading	20.1	0	0	21.15	300	250	1.20	3.10	827	12-D13	81	68	96	1.19	0.84
8	Sako 1999, SP-00 (SD295-00)	4-Point Loading	28.3	0	0	36.00	450	300	1.50	2.51	339	14-D13	107	78	106	1.36	1.01
9	Sako 2000, L90-00 (SD390-00)	4-Point Loading	26.9	0	0	36.00	450	300	1.50	3.37	426	12-D16	95	88	113	1.08	0.84
10	Sako 2000, L60-00	4-Point Loading	26.9	0	0	36.00	300	300	1.00	3.37	426	12-D16	148	132	170	1.13	0.87
11	Suzuki 1988, No. 1	Double Curvature	48.8	0	0	8.65	275	250	1.10	3.10	803	12-D13	127	108	150	1.18	0.85
12	Suzuki 1988, No. 4	Double Curvature	40.7	599	0.30	8.65	275	250	1.10	3.10	803	12-D13	265	146	202	1.82	1.31
13	Suzuki 1988, No. 9	Double Curvature	49.6	1460	0.60	8.65	275	250	1.10	3.10	803	12-D13	255	218	302	1.17	0.84
14	Nagae 1999, No. 1	Double Curvature	29.8	212	0	20.65	450	300	1.50	2.15	422	12-D13	141	94	131	1.51	1.08
15	Arai 2000, No. 3	Double Curvature	29.5	0	0	19.05	600	300	2.00	3.37	415	12-D16	114	73	97	1.55	1.18
16	Faradji et al. 1965, 25-3-C	Beam Test by 3 Point Loading	29.3	603	0.42	15.65	1050	251	4.18	3.07	406	12-D13	70	60	78	1.16	0.90
17	Faradji et al. 1965, 25-3-D	Beam Test by 3 Point Loading	34.2	571	0.34	15.65	1050	251	4.18	3.07	406	12-D13	66	63	82	1.05	0.81
18	Faradji et al. 1965, F-25-3-A	Beam Test by 3 Point Loading	29.0	0	0	15.65	700	251	2.79	3.07	406	12-D13	70	46	60	1.54	1.18
19	Faradji et al. 1965, F-25-3-B	Beam Test by 3 Point Loading	30.0	0	0	15.65	600	252	2.38	3.05	406	12-D13	76	49	64	1.55	1.18
20	Kimura 1988, No. 1	Beam Test by 3 Point Loading	27.3	0	0	118.00	2000	1000	2.00	2.22	371	22-D32	711	470	652	1.51	1.09
21	Fukushima 1992, No. 1	Beam Test by 3 Point Loading	19.3	0	0	92.05	1500	500	3.00	2.43	376	24-D16	170	102	143	1.65	1.18
22	Kuroiwa and Okamoto 1999, No. 1	Beam Test by 3 Point Loading	26.3	0	0	84.10	1900	700	2.71	4.13	545	20-D32	437	272	371	1.60	1.18
23	Kuroiwa and Okamoto 1999, No. 2	Beam Test by 3 Point Loading	27.0	0	0	88.50	1900	700	2.71	2.16	1004	20-D23	383	221	302	1.73	1.27

Table 4.4 Summary of column properties for octagonal reinforced concrete column

No	Specimen	Section Shape and Test setup	Concrete Strength	Axial Load	Axial Level	Geometry				Longitudinal Reinf.			Ultimate Shear Strength			Vexp/Vcal	
			fc' N/mm^2	N KN	N/Agfc'	Clear Covering c mm	Shear Span a mm	Height H mm	a/H	Reinf. Ratio ρ %	Yield fy N/mm^2	Bar Dia φ mm	Experiment kN	Equivalent Square Section kN	Proposal kN	Equivalent Square Section	Proposal
1	Arakawa et al. 1987, No. 3	Octagonal-Double Curvature	28.6	215	0.12	18.0	300	275	1.09	3.80	366	12-D16	158	131	188	1.21	0.84
2	Arakawa et al. 1987, No. 11	Octagonal-Double Curvature	28.7	430	0.24	18.0	300	275	1.09	3.80	366	12-D16	188	149	213	1.27	0.89
3	Arakawa et al. 1987, No. 18	Octagonal-Double Curvature	31.1	215	0.11	18.0	450	275	1.64	3.80	363	12-D16	132	90	129	1.47	1.03

4.3 APPLICATION OF PROPOSAL TO OCTAGONAL REINFORCED CONCRETE COLUMN

To obtain unified shear strength equation, the author is attempted to apply the proposal of shear strength equation to octagonal reinforced concrete column. Since octagonal column is quite similar to circular column in geometry, the detail parameter for octagonal still defined as same as the case of circular column. The effective depth is defined as the distance from compression face up to lowest layer of tensile reinforcement and effective concrete area is the area above the lowest tensile reinforcement (see Figure 4.5). The effective longitudinal reinforcement is half of total longitudinal reinforcement.

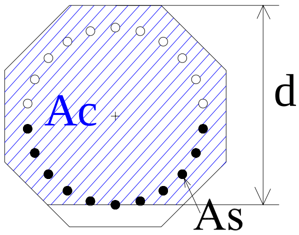


Figure 4.5 Detail parameter of proposal for octagonal column

The experiment of reinforced concrete octagonal column had conducted by Arakawa et al. (1987). The details specimens are attached in Appendix B. Table 4.4 summarize the column properties using in verification. For octagonal column, there is no explanation directly in JSCE specification. However, the calculation method for circular section is adopted for octagonal one in the analysis for comparison purpose. Thus octagonal section is transformed into equivalent square section. The width of equivalent section is calculated from square root of gross area. The effective depth of transform section is still calculated as the depth up to centroid of tensile reinforcement in 90-degree portion. The tensile reinforcement is account for the reinforcement arranged in 90-degree portion. The comparison of calculated shear strength to experimental results is shown in Figure 4.6.

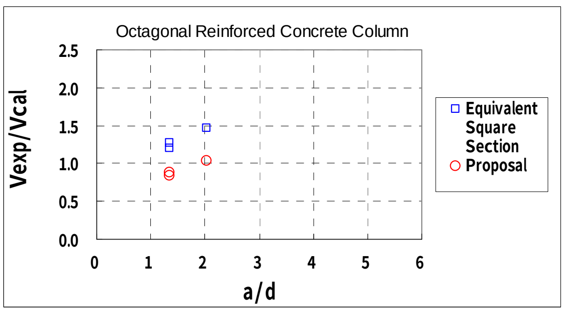


Figure 4.6 Verification of proposal for octagonal reinforced concrete column

4.4 APPLICATION OF PROPOSAL TO SQUARE REINFORCED CONCRETE COLUMN

The current JSCE shear design equation had formulated from the experimental results of reinforced concrete beam, which contain no side reinforcement. To investigate the amount of effective longitudinal reinforcement for the case of square column, the analysis were compared between the case of neglecting side reinforcement by accounting only lowest layer of tensile reinforcement, case of all layer taken into account in summation form as proposed by Ishibashi in 1985, and lastly case of only half of total reinforcement taken into account as proposed by author (see Figure 4.7).

The experimental results of square and rectangular reinforced concrete column containing side reinforcement were collected. The details of these specimens were attached in Appendix C and the specimen properties were summarized in Table 4.6.

From analytical results (Figure 4.8 and Table 4.5), it was found that amount of effective longitudinal reinforcement calculated by summation form show closely results in mean and variation with the one using effective reinforcement as half of total reinforcement.

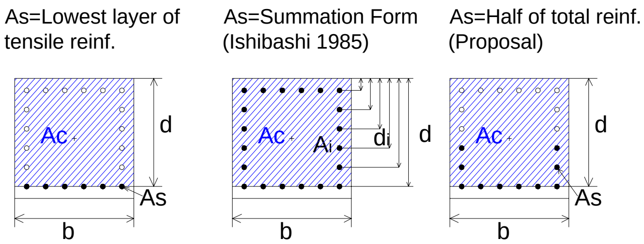


Figure 4.7 Parameter to compare effect of side reinforcement

Table 4.5 Summary of good combination case for all range of data

Case	Average Vtest/Vcal	COV %
As=Single layer of tensile reinf.	1.16	17.41
As=Multilayer in summation form	0.99	16.57
As=Half of total reinf. (Proposal)	1.02	16.55

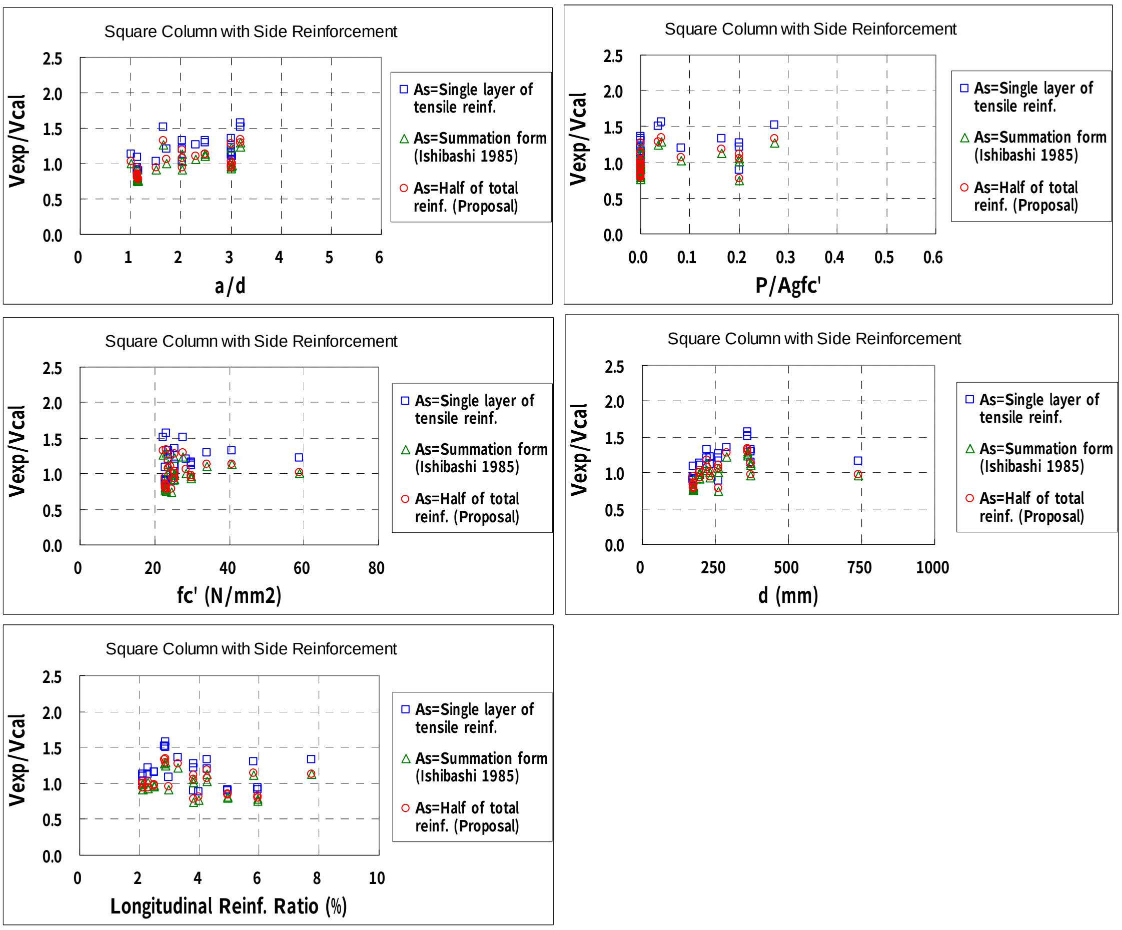


Figure 4.8 Comparison of effective reinforcement for square reinforced concrete column

To clarify the relationship between reinforcement in summation form and half of total reinforcement, the geometry of square section with *n*-number of reinforcement were calculated. It was assumed that the section is square shape with height *h* and covering 0.1*h*. The spacing at each reinforcement can calculated as $0.8h/(n/4)=3.2h/n$.

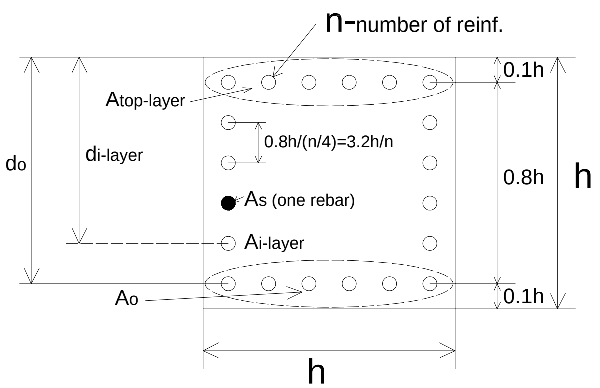


Figure 4.9 Geometry of assumed square section

The effective reinforcement in summation form can calculated as hereafter,

$$A_{S,Summation} = A_o + \sum A_i \frac{d_i}{d_o} \tag{4.7}$$

$$A_o = A_s \cdot (n/4+1) \tag{4.8}$$

$$\sum A_i \frac{d_i}{d_o} = 2A_s \cdot \sum_{i=1}^{i=(n/4)-1} \left(1 - i\frac{32}{9n}\right) + A_s \cdot (n/4+1)\frac{1}{9} \tag{4.9}$$

where n is number of longitudinal reinforcement

A_s is cross sectional area for one rebar

i is layer number counting from bottom to top

d_i is distance from compression face to layer-i

d_o is distance from compression face to lowest layer of tensile reinforcement

By substitute equation (4.8) and (4.9) into (4.7), we get solution:

$$A_{effective,\,Ishibashi} = A_s \cdot \frac{5}{9} n \tag{4.10}$$

The effective reinforcement for proposal is defined as half of total reinforcement,

$$A_{effective,\,proposal} = A_s \cdot \frac{1}{2} n \tag{4.11}$$

Therefore, the ratio of effective reinforcement calculated by proposal method as half of total reinforcement over the summation method as proposed by Ishibashi in 1985, is as follow

$$\frac{A_{effective,Ishibashi}}{A_{effective,\,proposal}} = \frac{10}{9} \tag{4.12}$$

By substitute this ratio into reinforcement function, βp, we can get the relationship that

$$V_{C,summation} = \sqrt[3]{10/9} \cdot V_{C,\,proposal} = 1.036 \cdot V_{C,\,proposal} \tag{4.13}$$

From this analysis, it clarifies that by using effective reinforcement in summation form as propose by Ishibashi (1985) and by using the proposal as half of total reinforcement, the variation of both method should be identical with difference in mean by $V_{c,summation}/V_{c,proposal}$=1.036.

Table 4.4 Summary of properties for square reinforced concrete column

No	Specimen	Section Shape and Test setup	Concrete Strength fc' N/mm^2	Axial Load N KN	Axial Level N/Agfc'	Geometry: Width b mm	Geometry: Heigh H mm	Geometry: Shear Span a mm	Geometry: d mm	Geometry: a/d	Longitudinal Reinf.: Reinf. Ratio ρ %	Longitudinal Reinf.: Yield fy N/mm^2	Longitudinal Reinf.: Bar Dia ϕ mm	Ultimate Shear Strength: Experiment kN	Ultimate Shear Strength: Single Layer (Not include side reinf.) kN	Ultimate Shear Strength: Multi-Layer (Summation form) kN	Ultimate Shear Strength: Half total reinf. (Proposal) kN	Comparison: Vexp/Vcal Single Layer	Comparison: Vexp/Vcal Summation form	Comparison: Vexp/Vcal Proposal
1	Jinno 1996, No. 1	Double Curvature	22.4	980	0.27	400	400	600	360	1.67	2.85	314	12-D22	417	275	328	315	1.51	1.27	1.32
2	Kato 1996, A1-1	Double Curvature	28.5	513.32	0.20	300	300	450	260	1.73	3.82	611	12-D19	210	174	209	199	1.21	1.01	1.05
3	Kato 1996, A2-1	Double Curvature	24.5	441	0.20	300	300	300	260	1.15	3.82	611	12-D19	217	243	291	278	0.89	0.74	0.78
4	Tadehara 1985, 22	4-Point Loading	22.7	0	0	200	200	200	175	1.14	2.98	361	6-D16	88	81	97	93	1.08	0.91	0.95
5	Tadehara 1985, 32	4-Point Loading	22.9	0	0	200	200	200	175	1.14	3.97	361	8-D16	83	93	108	103	0.88	0.77	0.80
6	Tadehara 1985, 42	4-Point Loading	22.9	0	0	200	200	200	175	1.14	4.96	361	10-D16	92	103	116	111	0.90	0.79	0.83
7	Tadehara 1985, 42M	4-Point Loading	22.9	0	0	200	200	200	175	1.14	4.96	361	10-D16	94	103	116	111	0.92	0.81	0.85
8	Tadehara 1985, 44M	4-Point Loading	23.0	0	0	200	200	200	175	1.14	5.96	361	12-D16	97	103	123	118	0.94	0.79	0.82
9	Tadehara 1985, 44D	4-Point Loading	23.0	0	0	200	200	200	175	1.14	5.96	361	12-D16	93	103	123	118	0.90	0.76	0.79
10	Kokusho et al. 1978, R-10-0	4-Point Loading	25.3	0	0	220	220	200	198	1.01	2.09	340	8-D13	122	108	123	119	1.13	1.00	1.03
11	Kokusho et al. 1978, R-15-0	4-Point Loading	25.2	0	0	220	220	300	198	1.52	2.09	340	8-D13	74	72	82	79	1.03	0.91	0.94
12	Kokusho et al. 1978, R-20-0	4-Point Loading	25.3	0	0	220	220	400	198	2.02	2.09	340	8-D13	54	53	60	58	1.03	0.91	0.94
13	Kokusho et al. 1978, R-30-0	4-Point Loading	25.3	0	0	220	220	600	198	3.04	2.09	340	8-D13	49	44	50	49	1.10	0.98	1.00
14	Yamamoto 1972, IS25-0-20	4-Point Loading	23.8	126	0.08	252	255	450	222	2.03	4.26	445, 468	4D19+8D16	107	89	104	99	1.20	1.03	1.07
15	Yamamoto 1972, IS25-0-20	4-Point Loading	23.8	252	0.16	252	255	450	222	2.03	4.26	445, 468	4D19+8D16	126	95	112	107	1.33	1.13	1.18
16	Kato 1996, A3-1	Double Curvature	24.3	437	0.20	300	300	600	260	2.31	3.82	611	12-D19	160	126	151	144	1.27	1.06	1.11
17	Ishibashi 1985, S6	Beam Test	34.1	0	0	200	400	925	370	2.50	5.81	-	12-D22	134	103	121	118	1.30	1.11	1.14
18	Ishibashi 1985, S7	Beam Test	40.6	0	0	200	400	925	370	2.50	7.74	-	16-D22	145	109	128	128	1.33	1.13	1.13
19	Tsuchiya 2001, N800	Beam Test	29.7	0	0	800	800	2220	740	3.00	2.48	351	20-D32	519	447	539	530	1.16	0.96	0.98
20	Tsuchiya 2001, N400	Beam Test	29.7	0	0	400	400	1110	370	3.00	2.48	353	20-D16	153	133	160	158	1.15	0.96	0.97
21	Tsuchiya 2001, N250	Beam Test	29.7	0	0	250	250	693	231	3.00	2.28	346	20-D10	64	57	68	67	1.12	0.93	0.95
22	Tsuchiya 2001, H250	Beam Test	58.7	0	0	250	250	693	231	3.00	2.28	346	20-D10	87	71	86	85	1.21	1.01	1.02
23	Satou 1996, N12-0	Cantilever Column	25.5	0	0	660	330	875	290	3.02	3.28	300	36-D16	292	216	240	229	1.35	1.22	1.27
24	Ishibashi 2001, I-1	Cantilever Column	27.7	157	0.04	400	400	1150	360	3.19	2.87	375	16-D19	216	143	174	167	1.51	1.24	1.29
25	Ishibashi 2001, I-6	Cantilever Column	23.2	157	0.04	400	400	1150	360	3.19	2.87	370	19-D19	212	135	165	158	1.57	1.29	1.34

4.5 UNIFIED SHEAR STRENGTH EQUATION

The final goal of this research is attempted to obtain the unified shear strength equation. The proposal was formulated from the case of circular reinforced concrete column, and expanded to the case of octagonal, square and rectangular column. The verifications with experimental results were performed for each case. For analysis purpose, the unified shear strength equation is obtained as hereafter:

$$V_c = 0.20 \cdot \sqrt[3]{f'_c} \cdot \beta_p \cdot \beta_d \cdot \beta_n \cdot \beta_a \cdot A_c \qquad (4.14)$$

$$\beta_p = \sqrt[3]{100P_w} \quad \leq 1.50 \qquad (4.15)$$

$$\beta_d = \sqrt[4]{1000/d} \quad \leq 1.50 \qquad (4.16)$$

$$\beta_n = 1 + 2M_o / M_u \quad \leq 2.0 \text{ (in compression)} \qquad (4.17)$$

$$\beta_a = 0.75 + 1.4/(a/d)^{-1} \quad , \text{a/d} > 2.0 \qquad (4.18a)$$

$$\beta_a = 3(a/d)^{-1} \quad , \text{a/d} \leq 2.0 \qquad (4.18b)$$

where P_w is effective longitudinal reinforcement ratio= A_s / A_c ,
A_s is effective reinforcement defined as half of total longitudinal reinforcement
A_c is effective concrete area defined as concrete above lowest tension reinf.
d is effective depth defined as the depth up to lowest tensile reinforcement
M_u is ultimate moment calculated at concrete compressive strain ε_c=0.0035
M_o is decompression moment to balance the axial stress

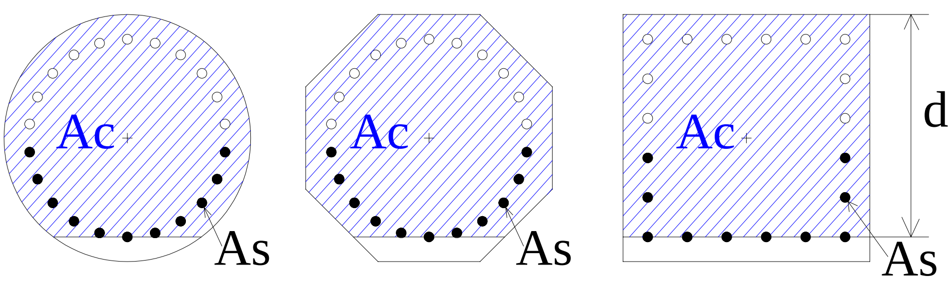


Figure 4.4 Illustration of parameters using in proposal for all type of cross section

However for design purpose, in some case, there is difficult to define the shear span-to-depth ratio like the case of distributed load and moving load. Thus the effect of shear span-to-depth ratio may be neglected in design. For the effect of axial load, from the experimental result, it was found that the data scattering is quite large. Therefore by concerning safety reason, the effect of axial load is modified to $\beta_n = 1 + M_0/M_d \le 2.0$ (in compression). Hence, the unified shear design equation is obtained as hereafter:

$$V_c = 0.20 \cdot \sqrt[3]{f'_c} \cdot \beta_p \cdot \beta_d \cdot \beta_n \cdot A_c \tag{4.19}$$

$$\beta_p = \sqrt[3]{100 P_w} \qquad \le 1.50 \tag{4.20}$$

$$\beta_d = \sqrt[4]{1000/d} \qquad \le 1.50 \tag{4.21}$$

$$\beta_n = 1 + M_0/M_d \qquad \le 2.0 \quad (N_d \ge 0) \tag{4.22a}$$

$$\beta_n = 1 + 2M_0/M_d \qquad \ge 0 \quad (N_d < 0) \tag{4.22b}$$

where P_w is effective longitudinal reinforcement ratio= A_s/A_c ,
A_s is effective reinforcement defined as half of total longitudinal reinforcement
A_c is effective concrete area defined as concrete above lowest tension reinf.
d is effective depth defined as the depth up to lowest tensile reinforcement
M_d is design moment according to JSCE Specification
M_o is decompression moment to balance the axial stress

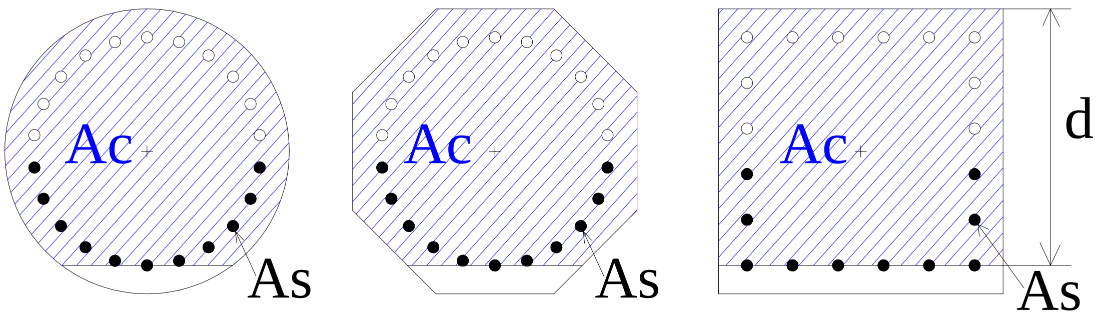


Figure 5.1 Illustration of parameters for proposal of shear design equation for column members

Chapter 5

CONCLUSIONS

To apply the current JSCE shear design equation for column member, there are three different between reinforced concrete beam and column members. The first different is contribution of side reinforcement in the case of column member. The second different is the section shape for column member, which may be circular, octagonal, or square section, while the one for beam member is only rectangular section. The third different is the source of axial load, which came from self-weight and live load in gravitational direction for column member, and from prestressed force for beam member. The variation in section geometry is resulting in problem of appropriate definition of effective depth. For circular cross section, current JSCE specification adopts the concept of transform section to equivalent square sections, which has the same cross sectional area and the effective depth is defined as the distance from compression face to centroid of tensile reinforcement arranged in 90-degree portion.

From 23 collected experimental results of reinforced concrete circular column, it was found that shear strength calculated by current JSCE specification for circular column based on transform section concept is quite conservative with average Vexp/Vcal=1.40. By consider the fact that current JSCE shear design equation is very accurate for the case of rectangular reinforced concrete beam with no side reinforcement and this current form of shear design equation was widely used and accepted, thus the objective of this research is to extend the current form of JSCE shear design equation to cover effect of side reinforcement and section shape like the case of column member with higher accuracy by no changing the general form of current JSCE shear design equation.

The first consideration is definition of effective depth, since it affects many functions; a/d function, size effect function, reinforcement ratio (As/bd), and effective concrete area (bd). For the case of circular section, the effective depth defined as full section depth is seem overestimate and effective depth for equivalent square section is seem underestimate. Thus the appropriate effect depth should be the one between these two extreme cases. Hence, the author proposed another definition of effective depth as the distance from compression face up to lowest level of tensile reinforcement.

The second consideration is the appropriate portion of tensile reinforcement accounted for reinforcement effect. Compare to JSCE specification that account the reinforcement arranged in 90-degree portion, the cases in analysis were expanded to the one in 120, 150 and 180-degree portion. The last case in analysis was summation method of reinforcement at each layer multiplying with distance from compression face to that layer and normalizing by distance from compression face to lowest layer of tensile reinforcement as proposed by Ishibashi (1985).

From these two consideration, there are four parameters in the analysis; effective concrete area, effective depth for a/d function, effective depth for size effect function and effective longitudinal reinforcement. These four parameters are leading to 3*3*3*5 =135 combination cases. Among these 135 combination cases, the case with less variation for all data ranges is preferable. The proposal was decided based on variation consideration and simplicity of parameter in practice. By considering the consistency of parameters, the author propose that the effective concrete area is defined as the area above lowest tensile reinforcement, and the effective depth is defined as the distance from compression face up to lowest layer of tensile reinforcement and used it both in a/d and size effect function. For simplicity in practice, the author proposed the effective longitudinal reinforcement as the half of total longitudinal reinforcement. The verification with experimental results of 23 circular columns show good calculated results in mean and variation comparing to current JSCE specification based upon transform section concept. The proposal was applied to octagonal and square RC column. The collected experimental results of 3 octagonal and 25 square columns with side reinforcement were used in verification. Since octagonal section column is quite similar to circular section column, the good calculated results were obtained. For square RC columns with side reinforcement, the proposal also show good accuracy with comparing to the case of neglecting side reinforcement.

The final form of proposal of shear design equation for reinforced concrete column without transverse reinforcement was mention again as follows,

$$V_c = 0.20 \cdot \sqrt[3]{f'_c} \cdot \beta_p \cdot \beta_d \cdot \beta_n \cdot A_c \qquad (5.1)$$

$$\beta_p = \sqrt[3]{100P_w} \qquad \leq 1.50 \qquad (5.2)$$

$$\beta_d = \sqrt[4]{1000/d} \qquad \leq 1.50 \qquad (5.3)$$

$$\beta_n = 1 + M_0/M_d \qquad \leq 2.0 \quad (N_d \geq 0) \qquad (5.4a)$$

$$\beta_n = 1 + 2M_0/M_d \qquad \geq 0 \quad (N_d < 0) \qquad (5.4b)$$

where P_w is effective longitudinal reinforcement ratio= A_s/A_c ,
A_s is effective reinforcement defined as half of total longitudinal reinforcement
A_c is effective concrete area defined as concrete above lowest tension reinf.
d is effective depth defined as the depth up to lowest tensile reinforcement
M_d is design moment according to JSCE Specification
M_o is decompression moment to balance the axial stress

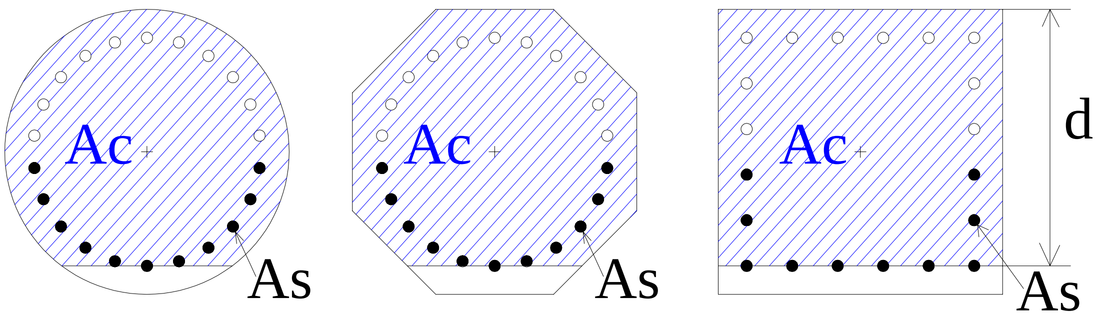


Figure 5.1 Illustration of parameters for proposal of shear design equation for column members

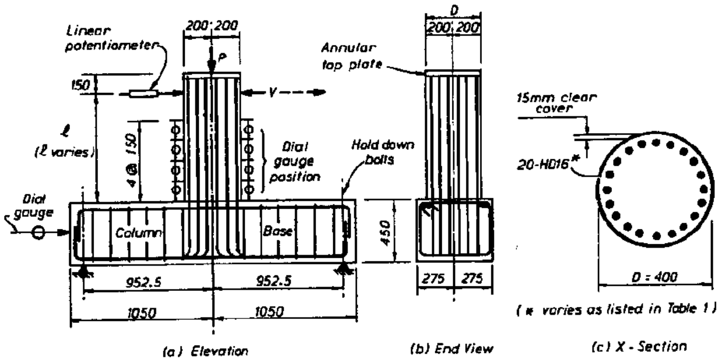
APPENDIX A

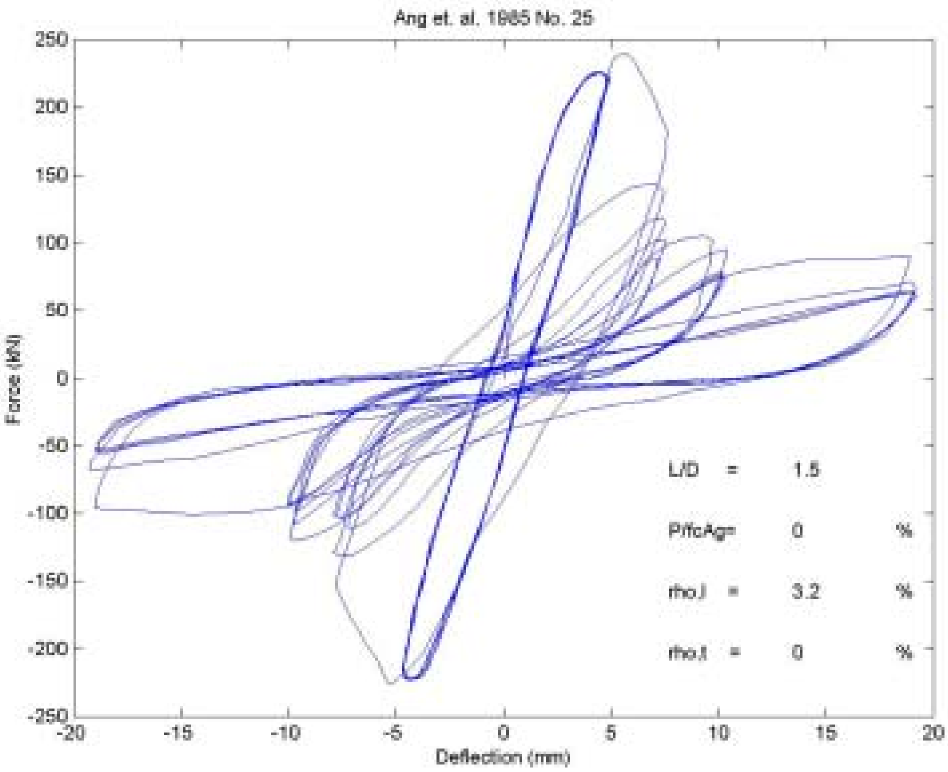
A SUMMARY OF EXPERIMENTAL RESULTS OF CIRCULAR RC COLUMNS

ANG et al. (1985)

Ang, B. G., Priestley, M. J. N., Paulay, T. (1985), Seismic shear strength of circular reinforced concrete Columns, *ACI Structural Journal*, Jan-Feb, pp.45-59.

Specimen	Concrete Strength	Axial Load	Axial Level	Geometry			Longitudinal Reinf.			Shear Strenth
				Shear Span	Dia.		Reinf. Ratio	Yield	Bar Dia	
	fc'	N	N/Agfc'	a	D	a/D	ρ	fy	ϕ	Vc
	N/mm^2	KN		mm	mm		%	N/mm^2	mm	kN
Ang 1985, No. 25	32.8	0	0.00	600	400	1.50	3.20	296	20-D16	239

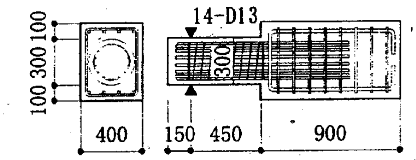


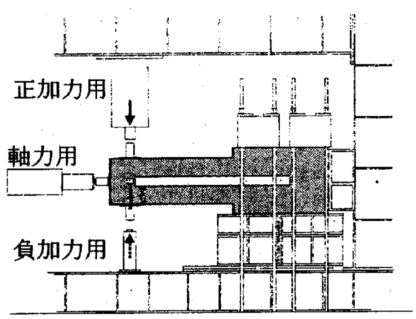


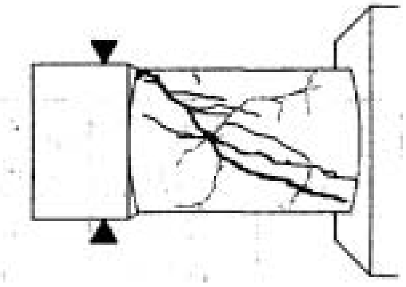
Yoshida et al. (1999)

Yoshida, M., Yamamoto, T., and Yamada, K. (1999), Experimental study on shear behavior of cast-in-place reinforced concrete pile, *Proceedings of JCI*, Vol.21, No.3, pp.487-492.

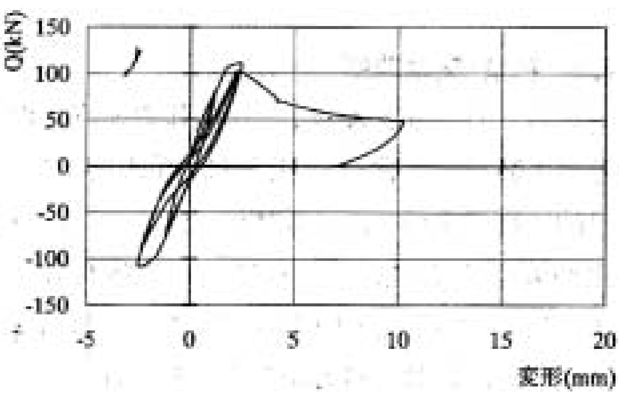
Specimen	Concrete Strength	Axial Load	Axial Level	Geometry			Longitudinal Reinf.			Shear Strenth
				Shear Span	Dia.		Reinf. Ratio	Yield	Bar Dia	
	fc'	N	N/Agfc'	a	D	a/D	ρ	fy	ϕ	Vc
	N/mm^2	KN		mm	mm		%	N/mm^2	mm	kN
Yoshida 1999, No. 3	28.8	0	0	450	300	1.5	2.51	339	14-D13	113







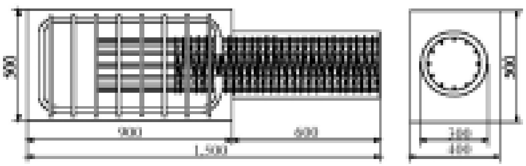
No. 3

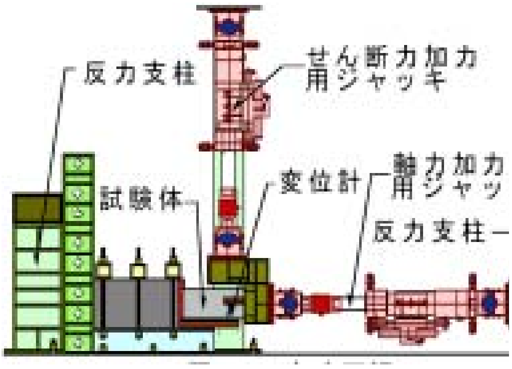


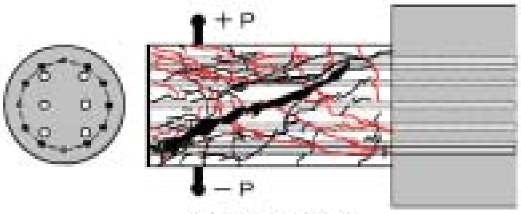
Yamada et al. (2003)

Yamada K., Yamamoto, T., and Okada, R. (2003), Shear-flexural behavior of reinforced concrete members with different section shape, *Proceedings of JCI*, Vol.25, No.2, pp.217-222

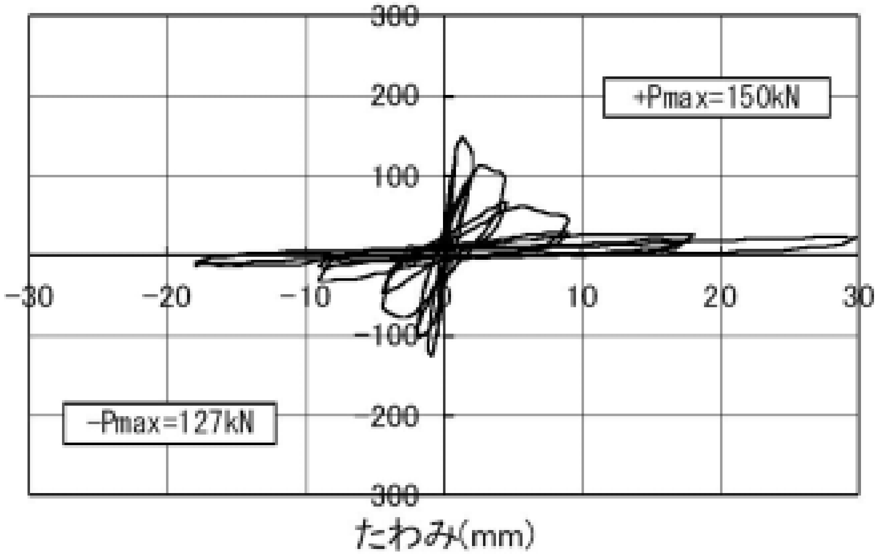
Specimen	Concrete Strength	Axial Load	Axial Level	Geometry			Longitudinal Reinf.			Shear Strenth
				Shear Span	Dia.		Reinf. Ratio	Yield	Bar Dia	
	fc'	N	N/Agfc'	a	D	a/D	ρ	fy	φ	Vc
	N/mm^2	KN		mm	mm		%	N/mm^2	mm	kN
Yamada 2003, C-000	25.75	530	0.29	450	300	1.5	3.37	436	12-D16	150







(a) C-000試験体

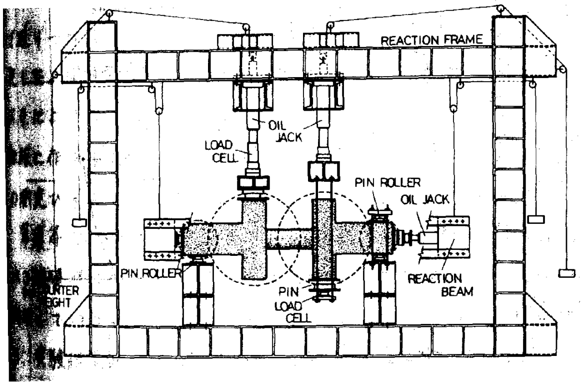


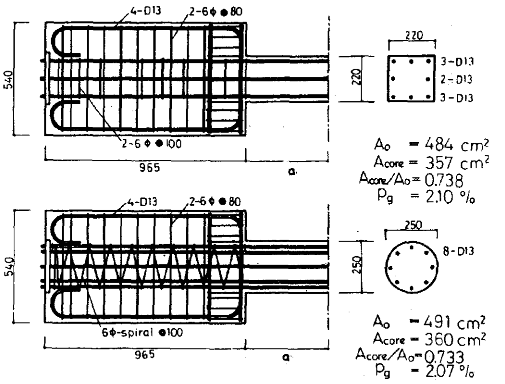
Kokusho et al. (1978)

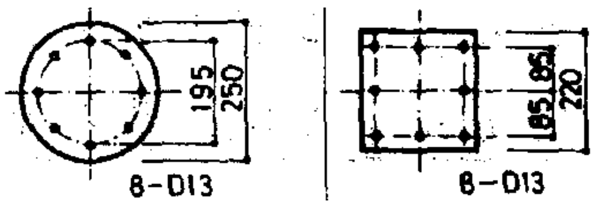
黒正清治，林静雄，雅己能森，小川幸雄 (1978)，円形断面を有する鉄筋コンクリート柱のせん断性状に関する実験，日本建築学会大会学術講演梗概集（北海道），pp.1737-1738.

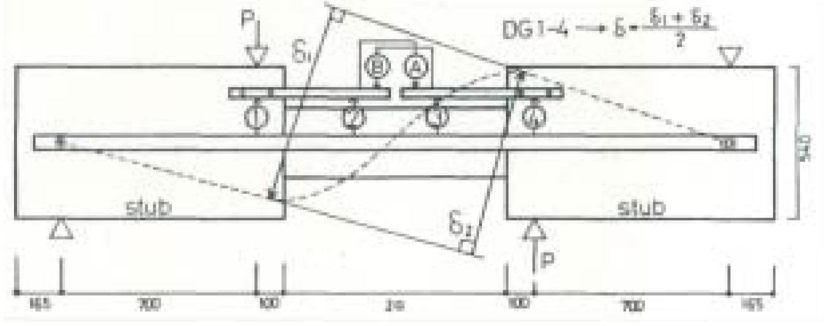
黒正清治，林静雄，小川幸雄 (1980)，軸力と曲げせん断力を受ける鉄筋コンクリート円形断面柱の強度と変形性に関する実験研究，日本建築学会大会学術講演梗概集（近畿），pp.1727-1728.

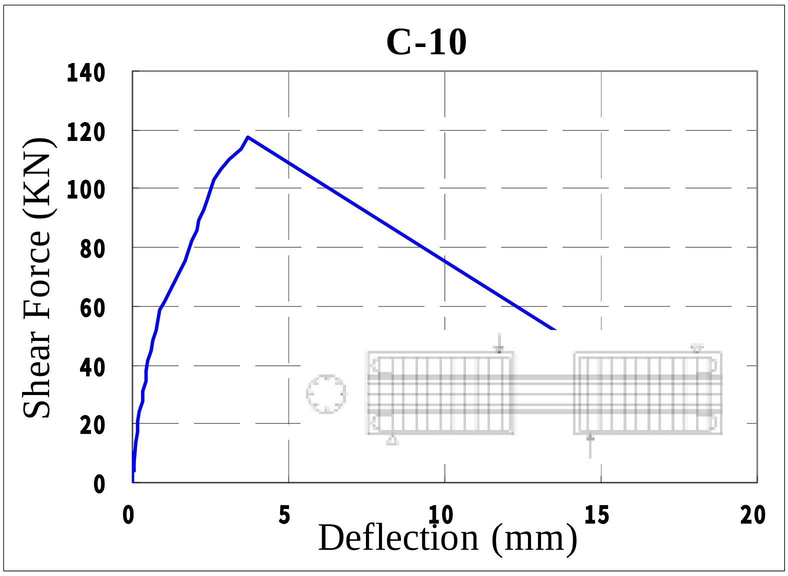
Specimen	Concrete Strength	Axial Load	Axial Level	Geometry			Longitudinal Reinf.			Shear Strenth
				Shear Span	Dia.		Reinf. Ratio	Yield	Bar Dia	
	fc'	N	N/Agfc'	a	D	a/D	ρ	fy	φ	Vc
	N/mm^2	KN		mm	mm		%	N/mm^2	mm	kN
Kokusho 1978, C-10-0	25.28	0	0	200	250	0.8	2.06	340	8-D13	117
Kokusho 1978, C-15-0	25.19	0	0	300	250	1.2	2.06	340	8-D13	89
Kokusho 1978, C-20-0	25.28	0	0	400	250	1.6	2.06	340	8-D13	64

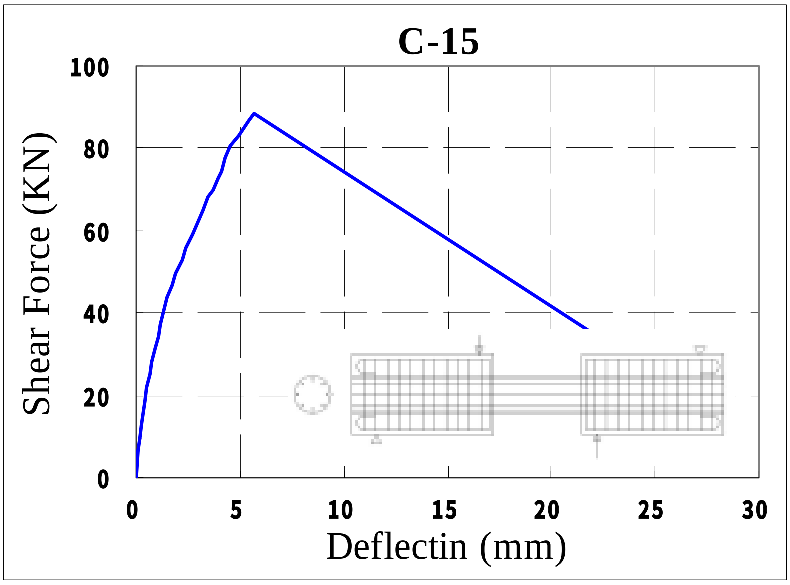


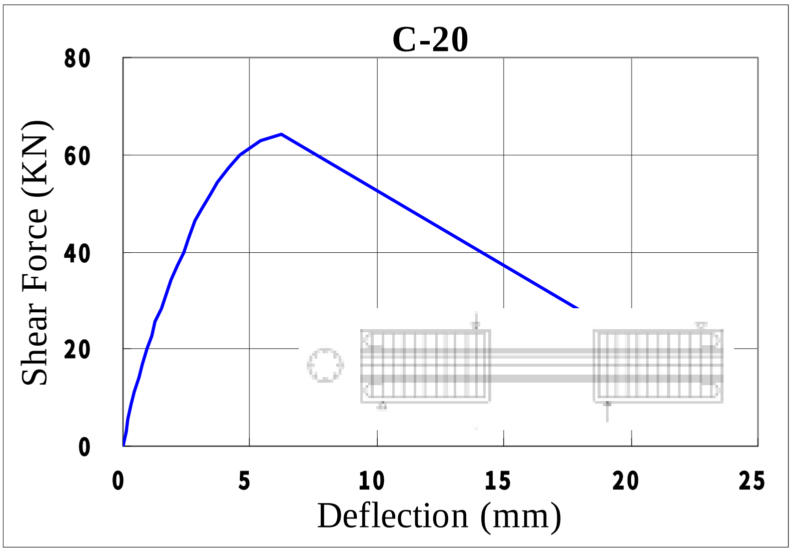







C-10
Shear Force (KN)
140
120
100
80
60
40
20
0
0
5
10
15
20
Deflection (mm)


C-15
Shear Force (KN)
100
80
60
40
20
0
0
5
10
15
20
25
30
Deflectin (mm)


C-20
Shear Force (KN)
80
60
40
20
0
0
5
10
15
20
25
Deflection (mm)

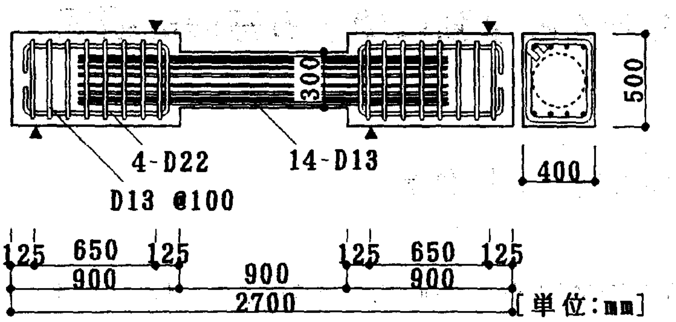
Sako et al. (1999, 2000, 2001)

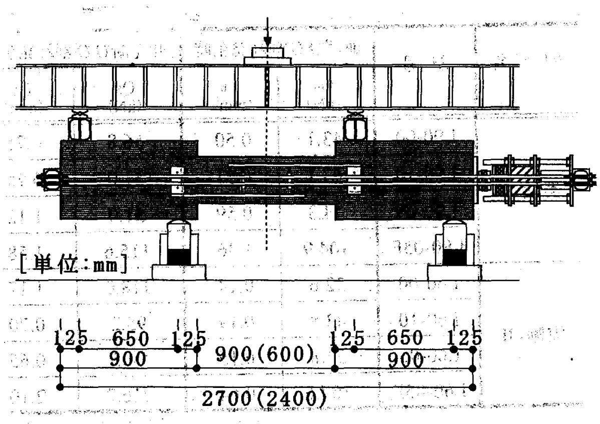
Sako, Y., Yamada, K., and Yamamoto, T. (1999), Fundamental study on shear behavior of cast-in-place reinforced concrete pile, *Proceedings of JCI*, Vol.21, No.3, pp.493-498.

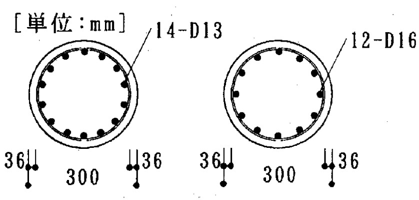
Sako, Y. et al. (2000), Effect of shear span to depth ratio on shear behavior of cast-in-place reinforced concrete pile, *Proceedings of JCI*, Vol.22, No.3, pp.673-678.

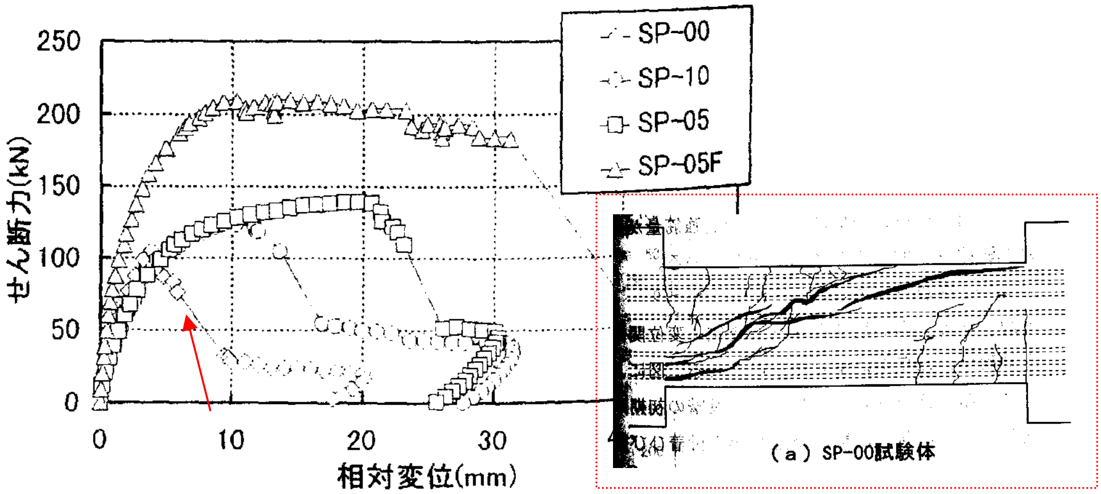
Sako, Y., et al. (2001), Experimental study on shear-flexural behavior of reinforced concrete circular members, *Proceedings of JCI*, Vol.23, No.3, pp.181-186.

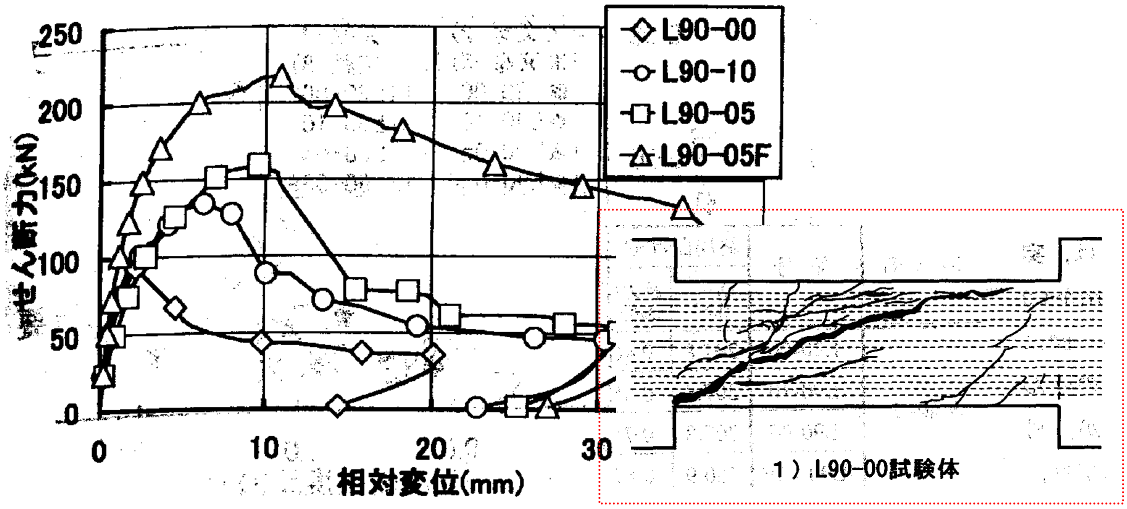
Specimen	Concrete Strength	Axial Load	Axial Level	Geometry			Longitudinal Reinf.			c Strenth
				Shear Span	Dia.		Reinf. Ratio	Yield	Bar Dia	
	fc'	N	N/Agfc'	a	D	a/D	ρ	fy	ϕ	Vc
	N/mm^2	KN		mm	mm		%	N/mm^2	mm	kN
Sako 1999, SP-00 (SD295-00	28.3	0	0	450	300	1.5	2.51	339	14-D13	107
Sako 2000, L90-00 (SD390-0	26.9	0	0	450	300	1.5	3.37	426	12-D16	95
Sako 2000, L60-00	26.9	0	0	300	300	1.0	3.37	426	12-D16	148



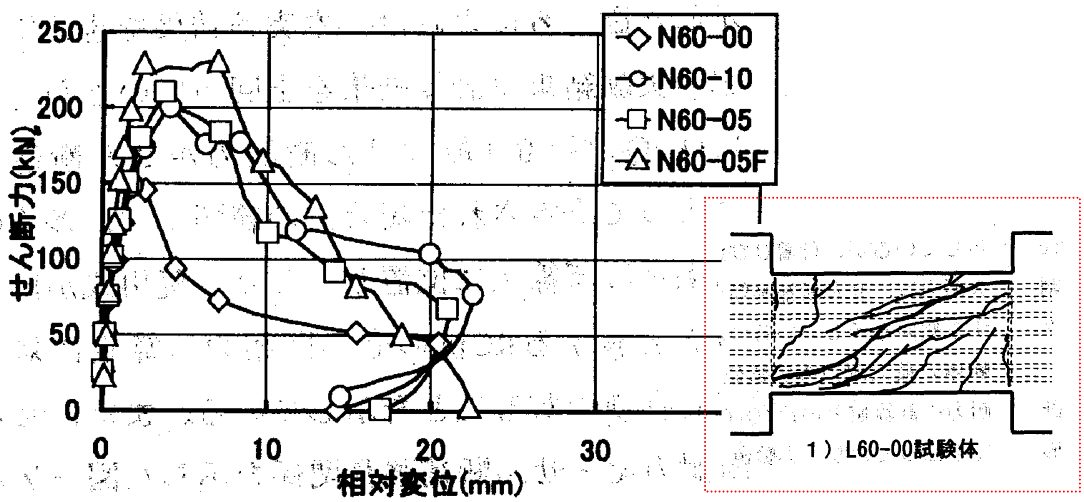





250
200
150
100
50
0
せん断力(kN)
0
10
20
30
相対変位(mm)
SP-00
SP-10
SP-05
SP-05F
（a）SP-00試験体


250
200
150
100
50
0
せん断力(kN)
0
10
20
30
相対変位(mm)
L90-00
L90-10
L90-05
L90-05F
1）L90-00試験体

（a）実験-I

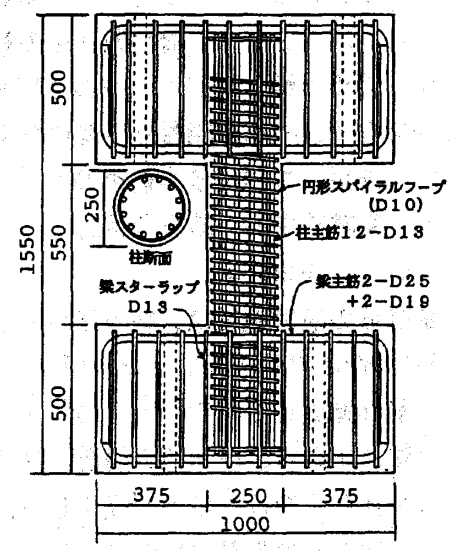

250
200
150
100
50
0
せん断力(kN)
0
10
20
30
相対変位(mm)
N60-00
N60-10
N60-05
N60-05F
1）L60-00試験体

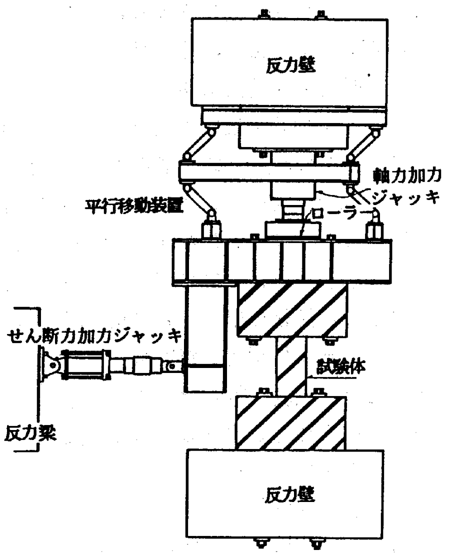
（b）実験-II

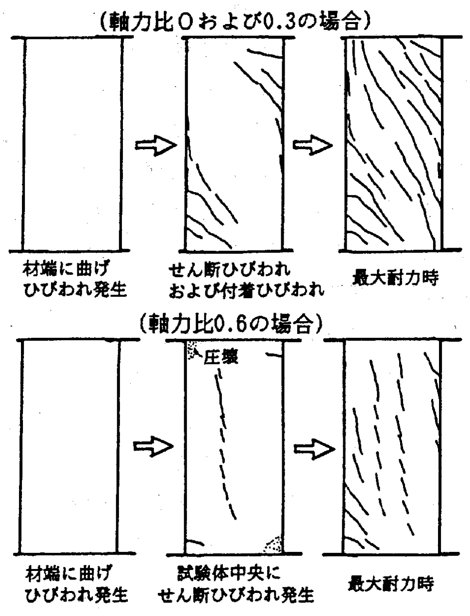
Suzuki et al. (1988)

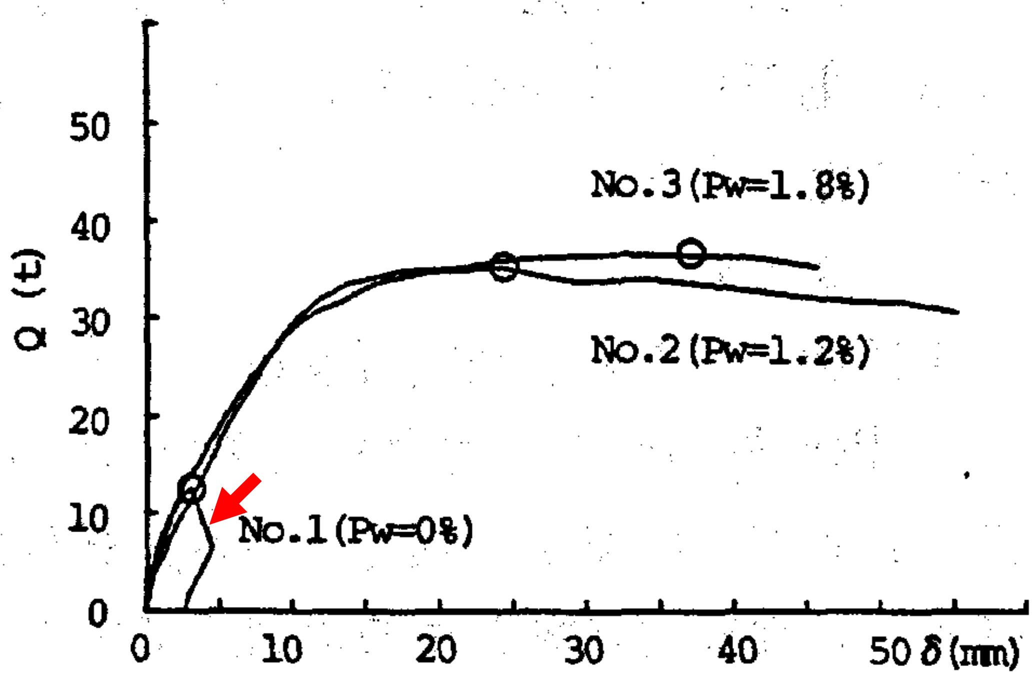
Suzuki, K. et al. (1988), Shear strength and deformation characteristics of reinforced concrete columns with circular spiral reinforcement of grade SD50, *Proceedings of JCI*, Vol.10, No.3, pp.601-606.

Specimen	Concrete Strength	Axial Load	Axial Level	Geometry			Longitudinal Reinf.			Shear Strenth
				Shear Span	Dia.		Reinf. Ratio	Yield	Bar Dia	
	fc'	N	N/Agfc'	a	D	a/D	ρ	fy	φ	Vc
	N/mm^2	KN		mm	mm		%	N/mm^2	mm	kN
Suzuki 1988, No. 1	48.80	0	0	275	250	1.1	3.10	802.5	12-D13	127
Suzuki 1988, No. 4	40.67	599	0.3	275	250	1.1	3.10	802.5	12-D13	265
Suzuki 1988, No. 9	49.59	1460	0.6	275	250	1.1	3.10	802.5	12-D13	255

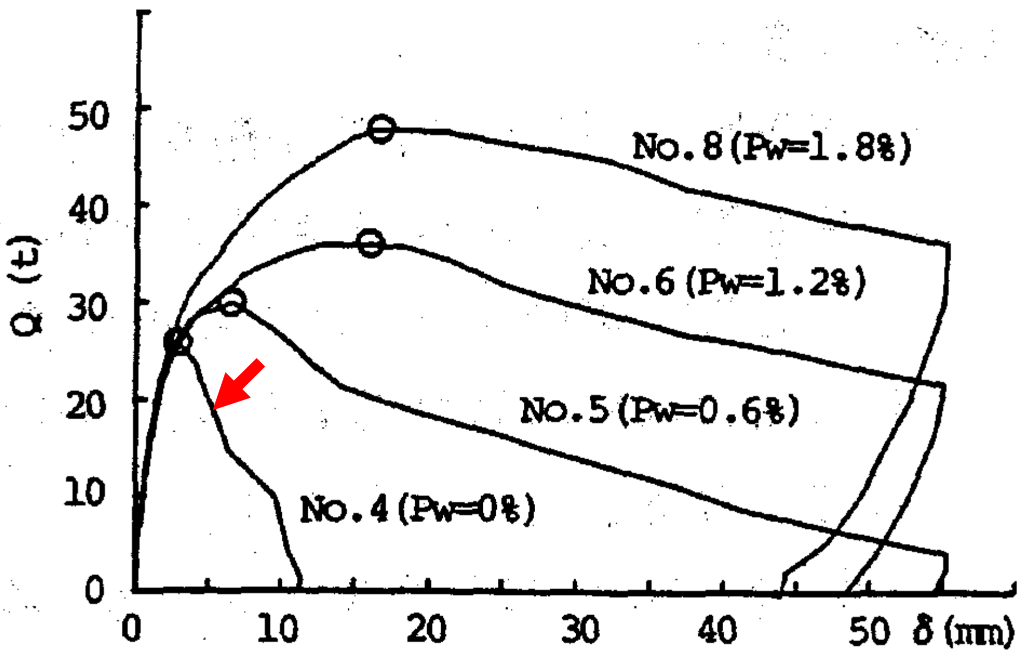




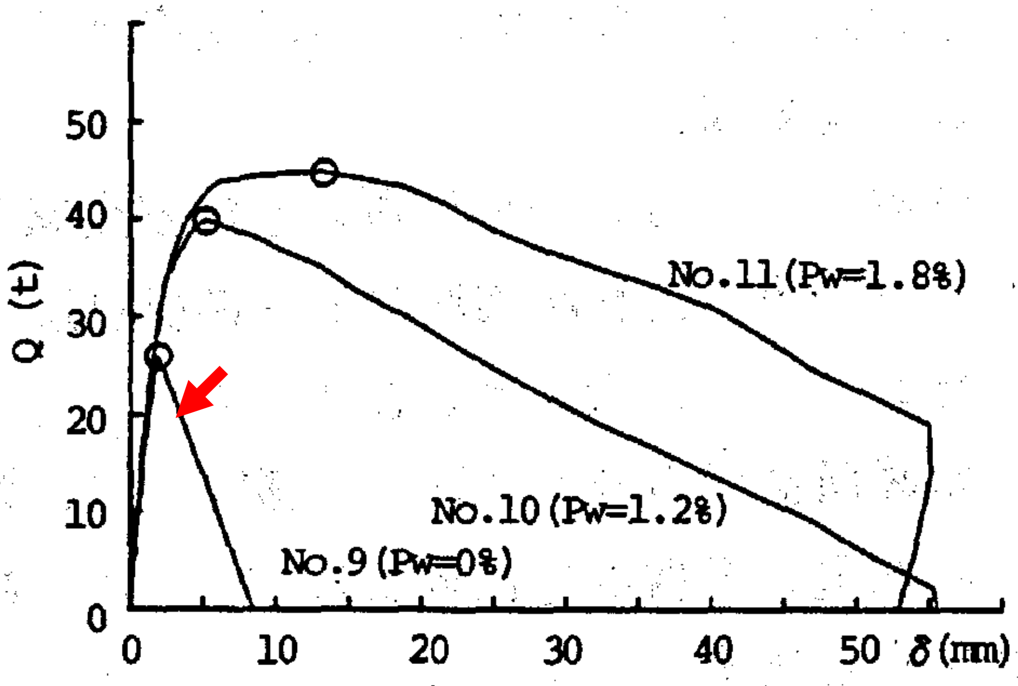




(a) $\eta = 0$



(b) $\eta = 0.3$

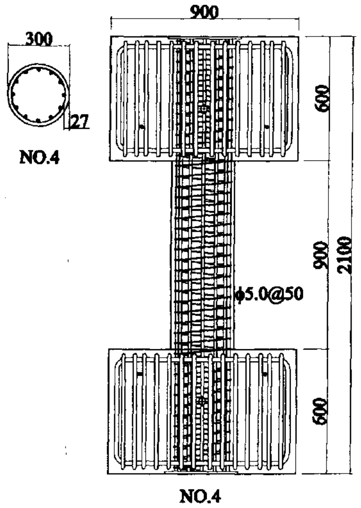


(c) η=0.6

Nagae et al. (1999)

Nagae, T., Katori, K., and Hayashi, S. (1999), Study on application of high-strength shear reinforcement to reinforcement concrete pile, *Proceedings of JCI*, Vol.21, No.3, pp.403-407.

Specimen	Concrete Strength	Axial Load	Axial Level	Geometry			Longitudinal Reinf.			Shear Strenth
				Shear Span	Dia.		Reinf. Ratio	Yield	Bar Dia	
	fc'	N	N/Agfc'	a	D	a/D	ρ	fy	ϕ	Vc
	N/mm^2	KN		mm	mm		%	N/mm^2	mm	kN
Nagae 1999, No. 1	29.8	212	0.10	450	300	1.5	2.15	422	12-D13	141



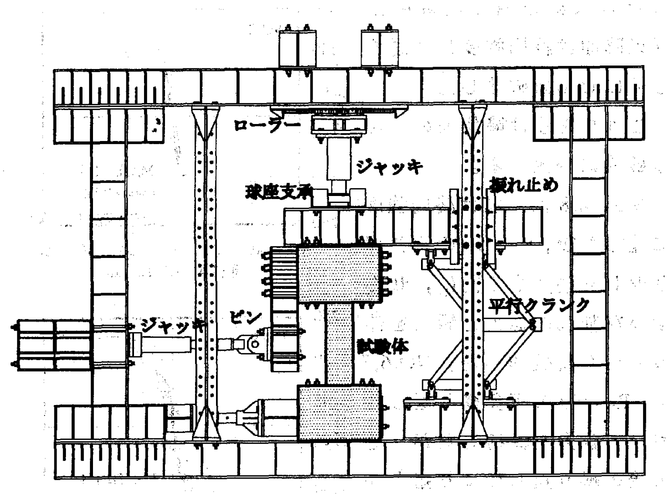
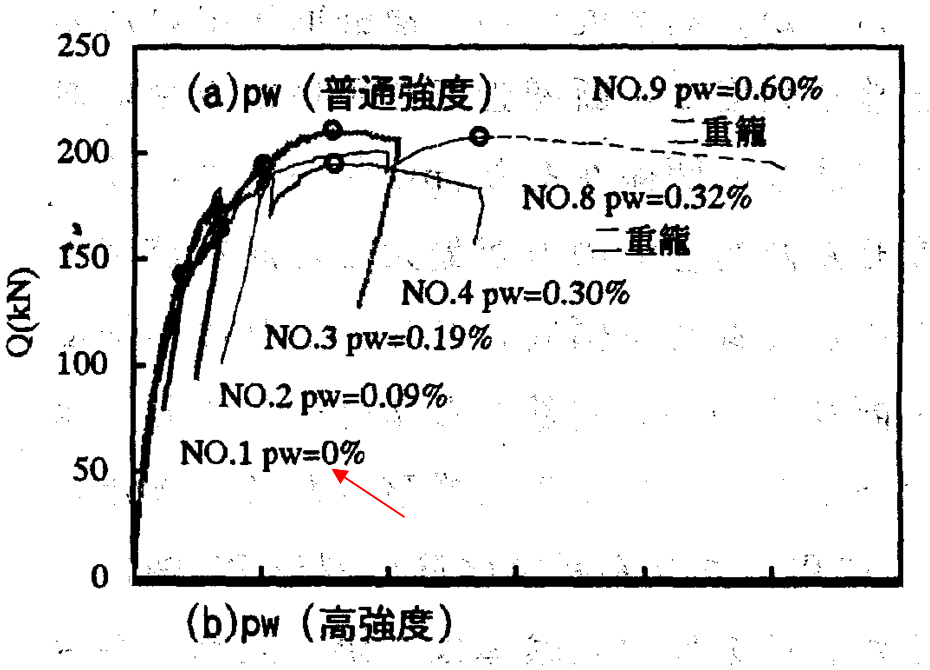


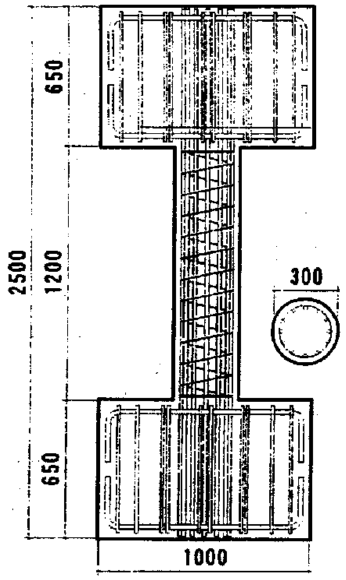
図-2 加力装置図

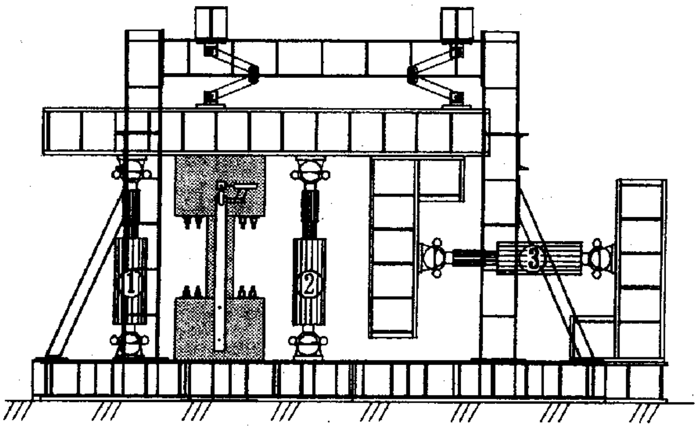


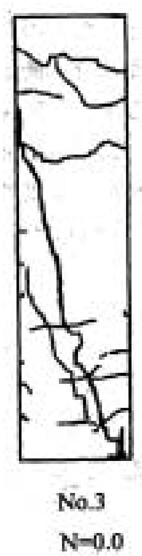
Arai et al. (2000)

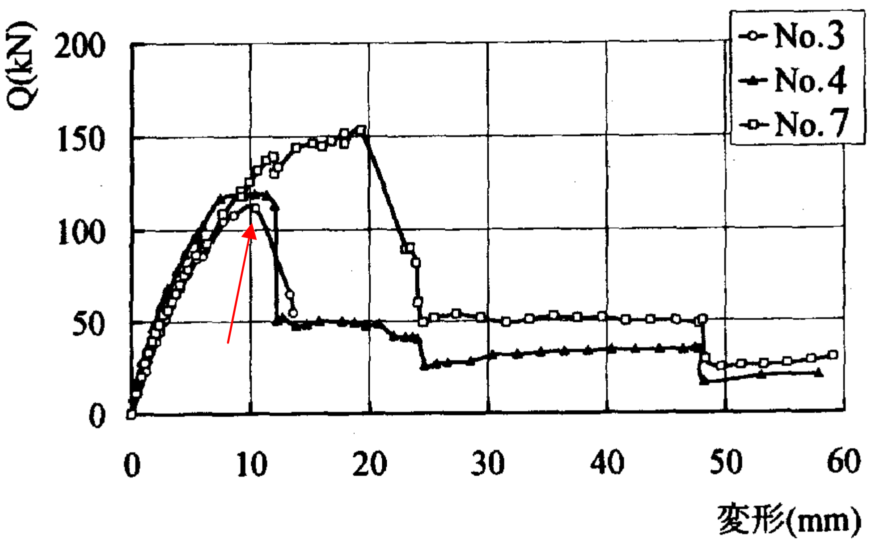
Arai, M. et al. (2000), Experimental study on shear-flexural behavior of cast-in-place reinforced concrete pile, *Proceedings of JCI*, Vol.22 No.3, pp.667-672.

Specimen	Concrete Strength	Axial Load	Axial Level	Geometry			Longitudinal Reinf.			Shear Strenth
				Shear Span	Dia.		Reinf. Ratio	Yield	Bar Dia	
	fc'	N	N/Agfc'	a	D	a/D	ρ	fy	φ	Vc
	N/mm^2	KN		mm	mm		%	N/mm^2	mm	kN
Arai 2000, No. 3	29.5	0	0	600	300	2	3.37	415	12-D16	114





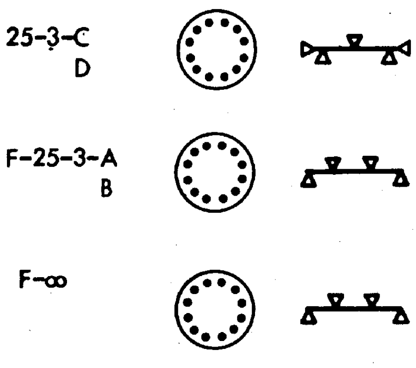


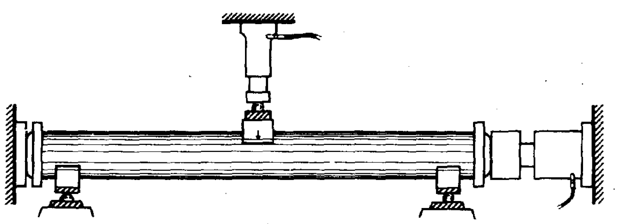


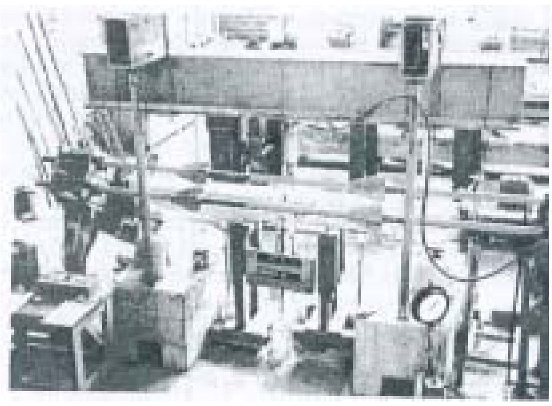
<u>Faradji and Diaz de Cossio (1965)</u>

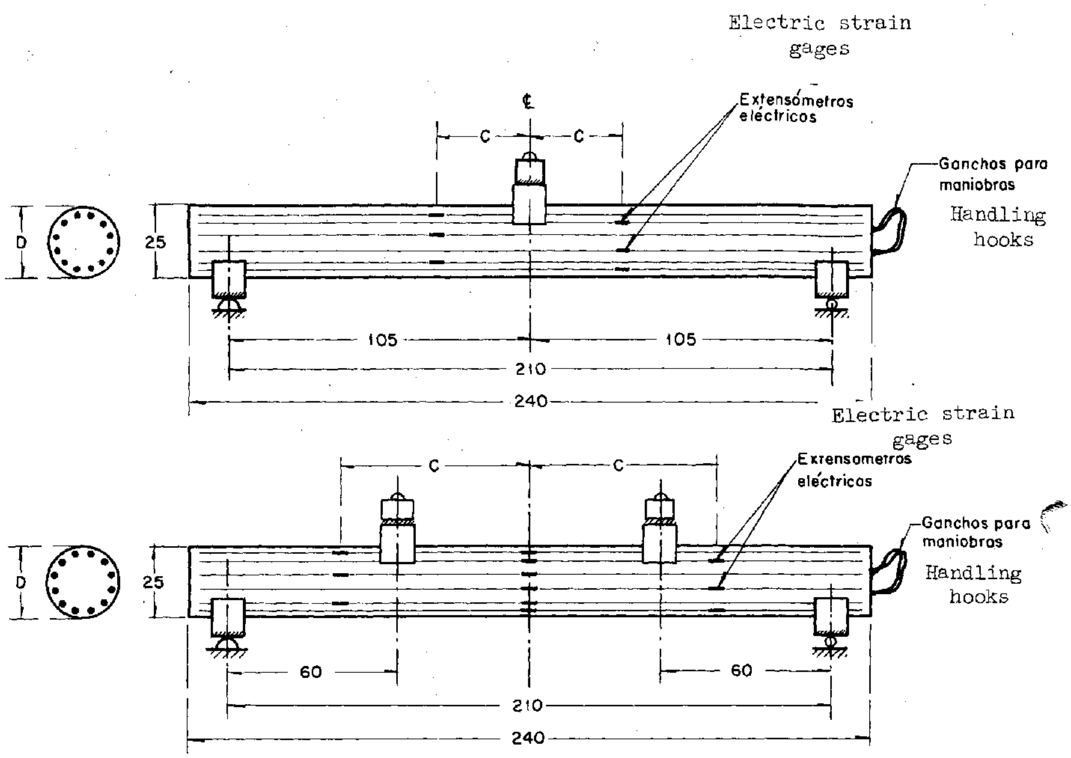
Faradji, M. J. and Diaz de Cossio, R. (1965), Diagonal Tension in Concrete Members of Circular Section, (in Spanish), *Ingenieria*, Mexico, April, pp.257-280 (Translation by Portland Cement Association, *Foreign Literature Study* No. 466).

Specimen	Concrete Strength fc' N/mm^2	Axial Load N KN	Axial Level N/Agfc'	Geometry: Shear Span a mm	Geometry: Dia. D mm	Geometry: a/D	Longitudinal Reinf.: Reinf. Ratio ρ %	Longitudinal Reinf.: Yield fy N/mm^2	Longitudinal Reinf.: Bar Dia ϕ mm	Shear Strenth Vc kN
Faradji 1965, 25-3 C	29.3	602.70	0.42	1050	251	4.18	3.07	vary	12-D13	70
Faradji 1965, 25-3 D	34.2	571.34	0.34	1050	251	4.18	3.07	vary	12-D13	66
Faradji 1965, F-25-3 A	29.0	0	0	700	251	2.79	3.07	vary	12-D13	70
Faradji 1965, F-25-3 B	30.0	0	0	600	252	2.38	3.05	vary	12-D13	76
Faradji 1965, F-alpha	13.1	0	0	600	251	2.39	3.07	vary	12-D13	69

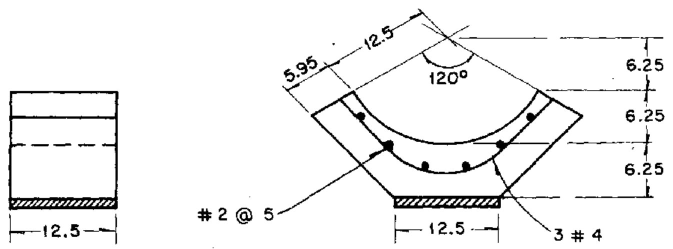








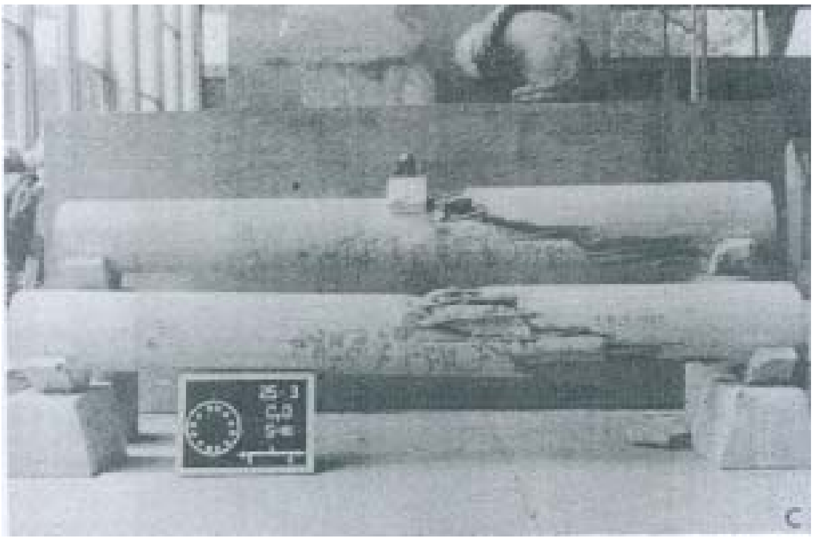
a) Croquis de los especímenes ensayados
Sketch of test specimen

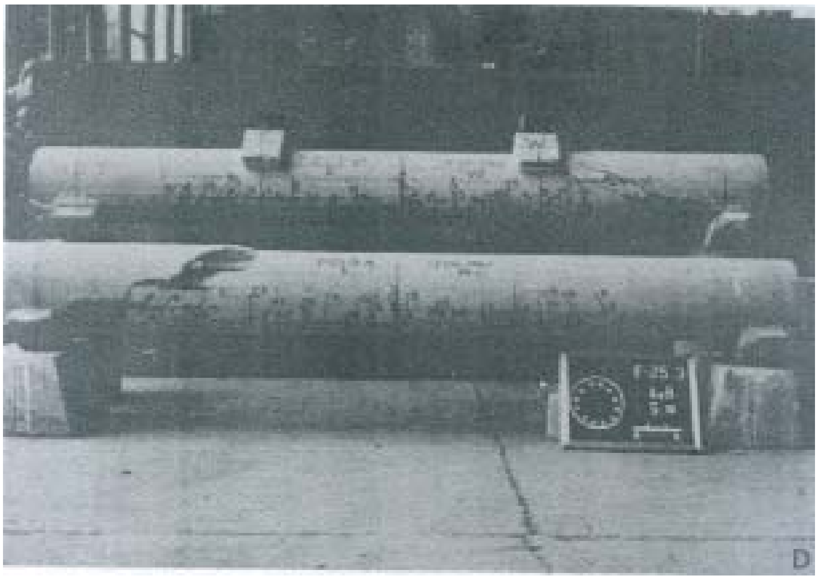


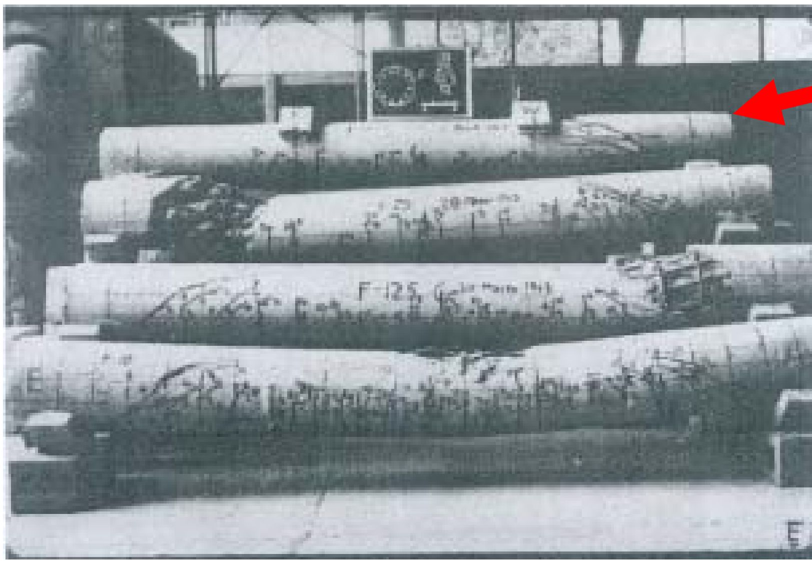
b) Zapatas de apoyo y carga
Details of load and support shoes

FIG. 2 CARACTERISTICAS GENERALES DE LOS ESPECIMENES

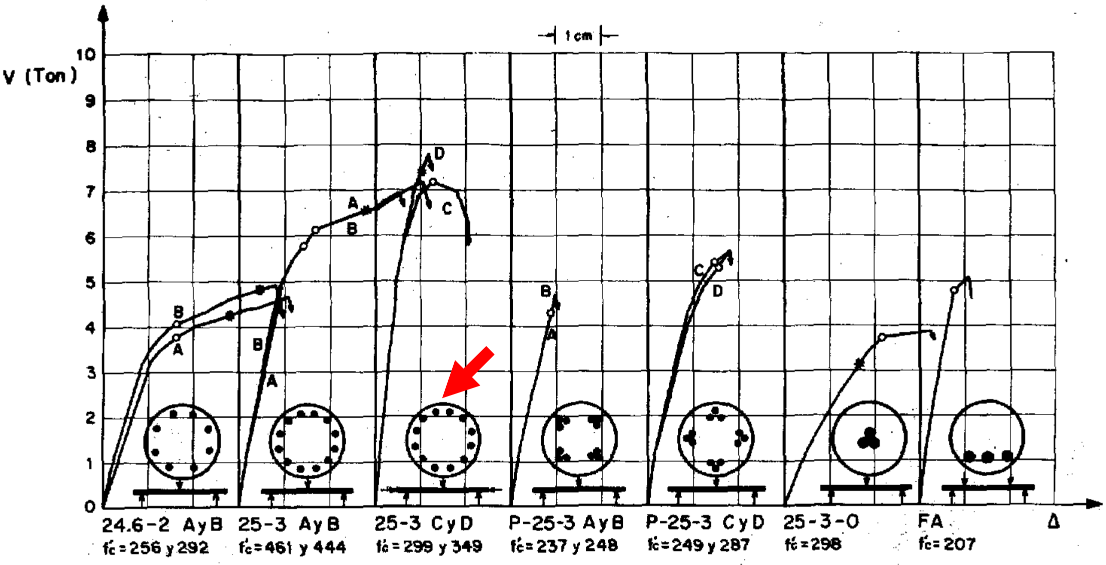
FIG. 2 GENERAL DETAILS OF SPECIMENS

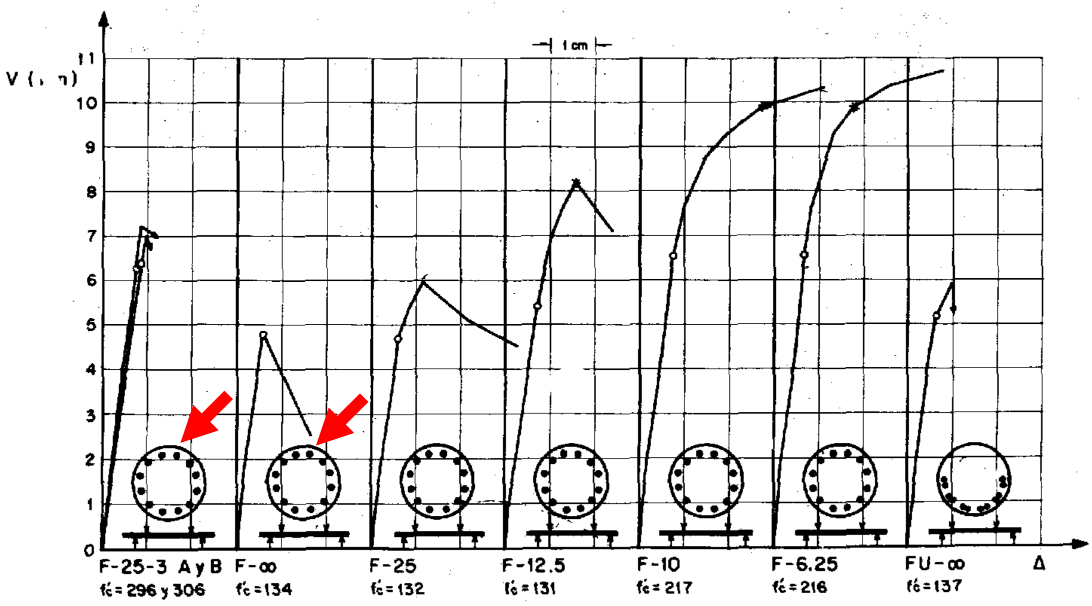






F-infinity

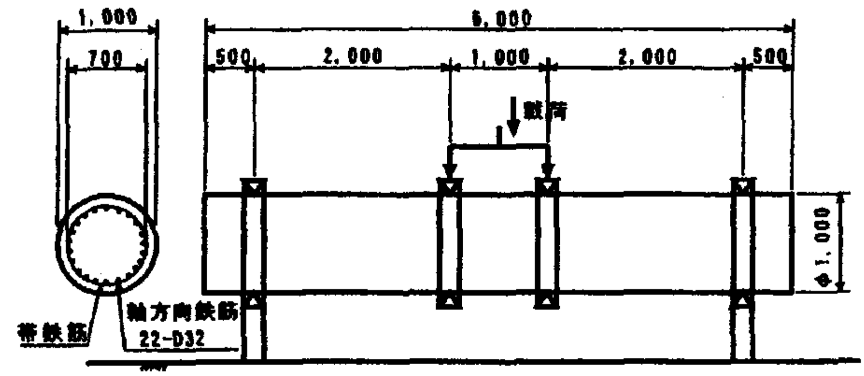

1 cm
V (Ton)
24.6-2 A y B
f'c = 256 y 292
25-3 A y B
f'c = 461 y 444
25-3 C y D
f'c = 299 y 349
P-25-3 A y B
f'c = 237 y 248
P-25-3 C y D
f'c = 249 y 287
25-3-0
f'c = 298
FA
f'c = 207
Δ

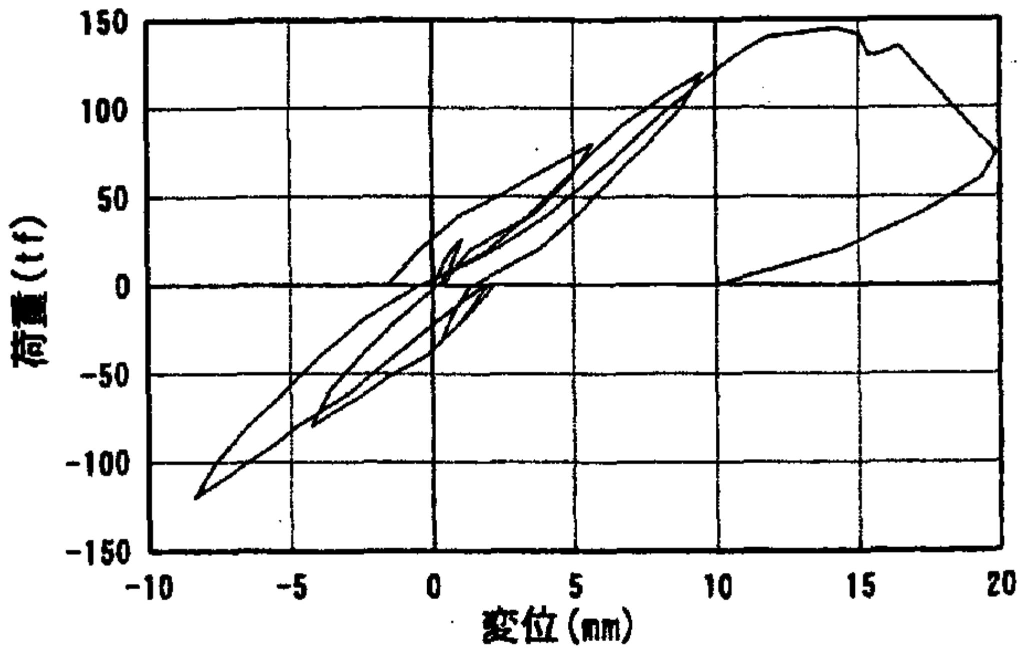

1 cm
F-25-3 A y B
f'c = 296 y 306
F-∞
f'c = 134
F-25
f'c = 132
F-12.5
f'c = 131
F-10
f'c = 217
F-6.25
f'c = 216
FU-∞
f'c = 137
Δ

Kimura et al. 1998

木村嘉富，大越盛幸，阪野彰，福井次郎 (1998)，場所打ち杭のせん断耐力評価法に関する載荷実験，土木学会第 53 回年次学術講演会, III-B42, pp.84-85.

Specimen	Concrete Strength fc' N/mm^2	Axial Load N KN	Axial Level N/Agfc'	Geometry			Longitudinal Reinf.			Shear Strenth Vc kN
				Shear Span a mm	Dia. D mm	a/D	Reinf. Ratio ρ %	Yield fy N/mm^2	Bar Dia φ mm	
Kimura 1988, No. 1	27.30	0	0	2000	1000	2.0	2.22	371	22-D32	711



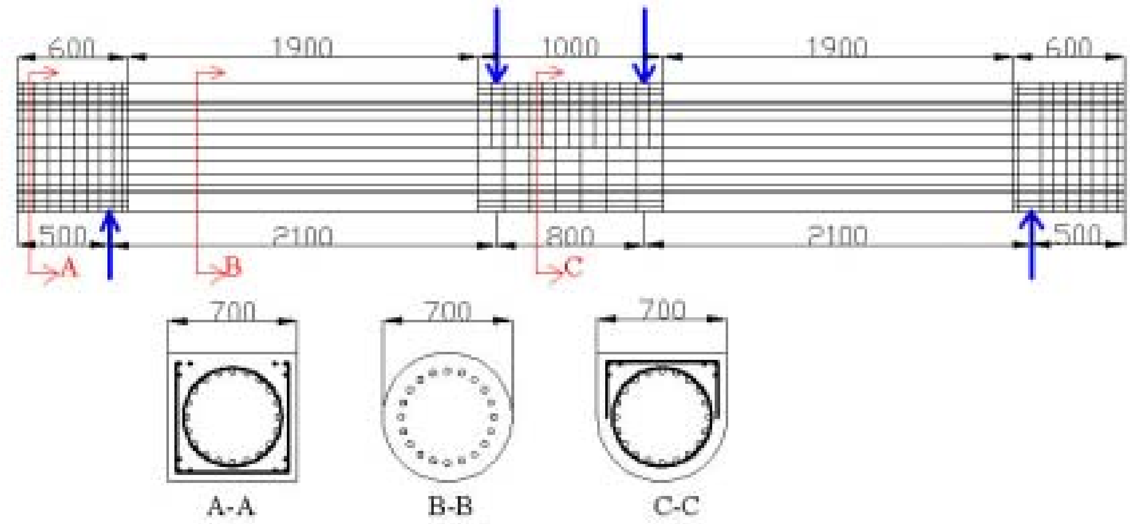


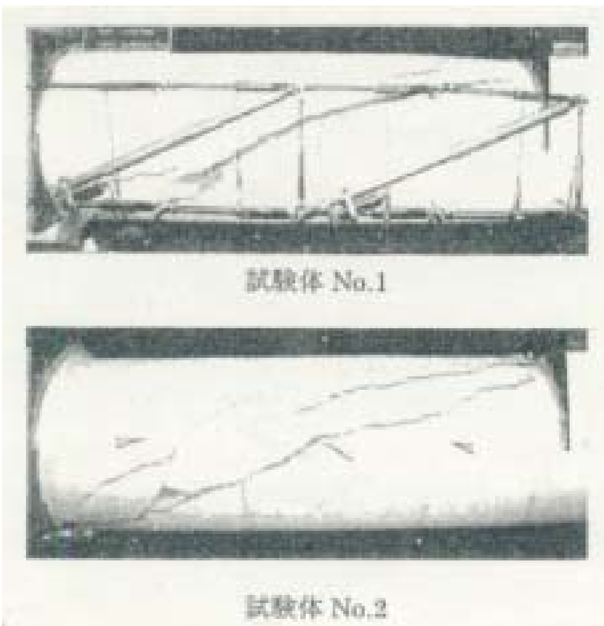
図－２　荷重～変位曲線（ケース１）

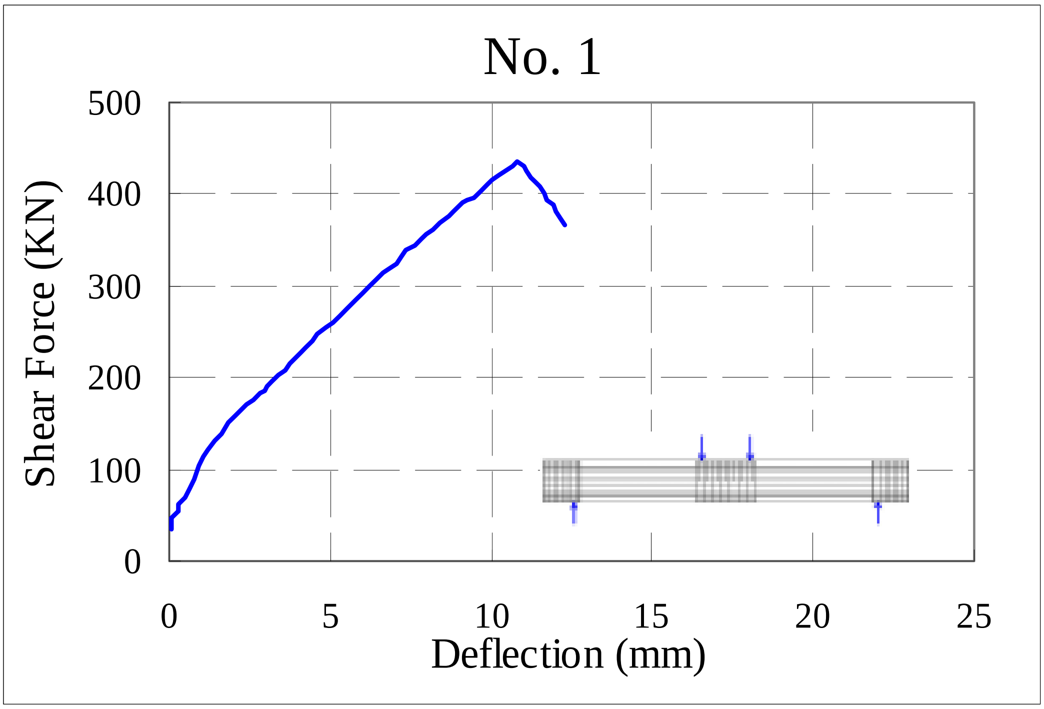
Koroiwa and Okamoto (1999)

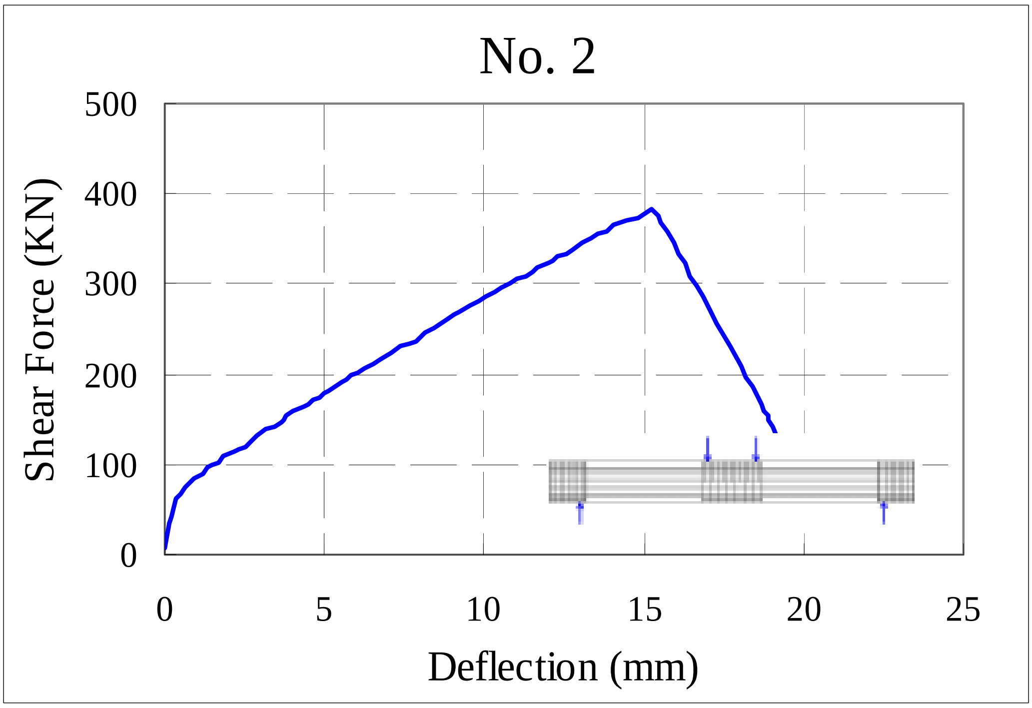
黒岩俊之，岡本大（1999），軸方向鉄筋比の異なるせん断補強筋のない円形部材の載荷実験，土木学会第 54 回年次学術講演会, pp.596-597.

Specimen	Concrete Strength	Axial Load	Axial Level	Geometry			Longitudinal Reinf.			Shear Strenth
	fc' N/mm^2	N KN	N/Agfc'	Shear Span a	Dia. D mm	a/D	Reinf. Ratio ρ %	Yield fy N/mm^2	Bar Dia ϕ mm	Vc kN
Kuroiwa 1999,No. 1	26.3	0	0	1900	700	2.71	4.13	545	20-D32(SD490)	437
Kuroiwa 1999,No. 2	27.0	0	0	1900	700	2.71	2.16	1004	20-D23(PC Bar)	383






No. 1
500
400
300
200
100
0
Shear Force (KN)
0
5
10
15
20
25
Deflection (mm)


No. 2
500
400
300
200
100
0
Shear Force (KN)
0
5
10
15
20
25
Deflection (mm)

APPENDIX B

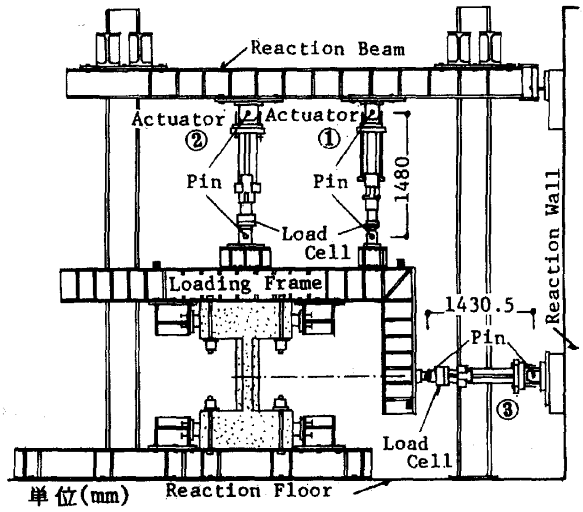
A SUMMARY OF EXPERIMENTAL RESULTS OF OCTAGONAL RC COLUMNS

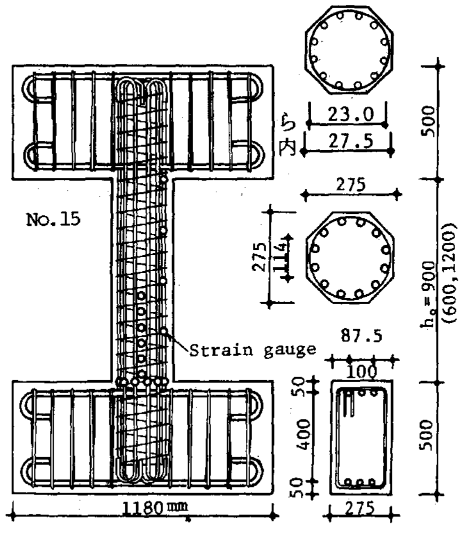
<u>Arakawa et al. 1987</u>

Arakawa, T. et al. (1987), Ultimate shear strength of spirally-confined concrete columns, *Proceedings of JCI*, Vol.9, No.2, pp.299-304.

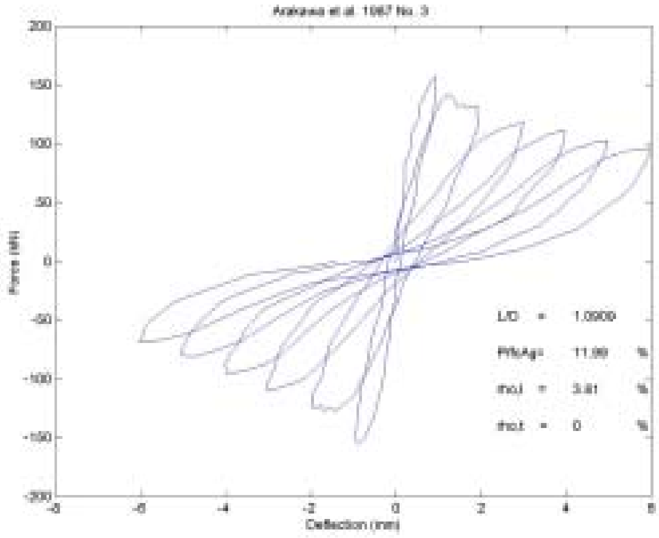
Arakawa, T. et al. (1988), Shear resisting behavior of reinforced concrete columns with spiral hoops, *Proceedings of JCI*, Vol.10, No.3, pp.577-582.

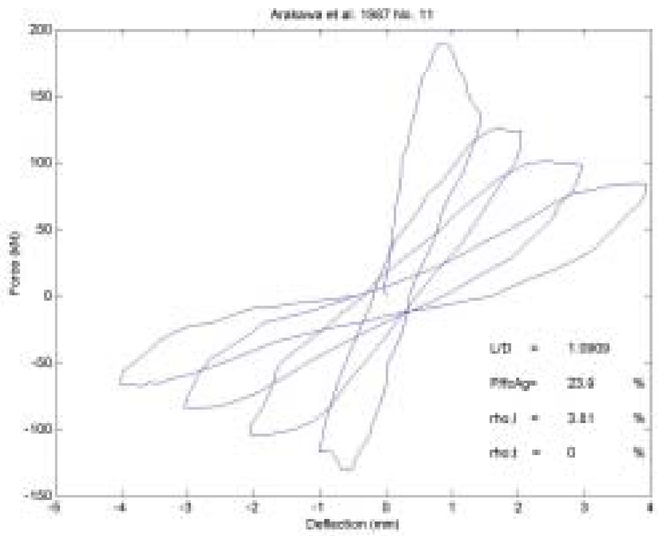
Specimen	Concrete Strnegth	Axial Load	Axial Level	Geometry			Longitudinal Reinf.			Shear Strenth
				Shear Span	Dia.		Reinf. Ratio	Yield	Bar Dia	
	fc'	N	N/Agfc'	a	D	a/D	ρ	fy	φ	Vc
	N/mm^2	KN		mm	mm		%	N/mm^2	mm	kN
Arakawa 1987, No. 3	28.6	215	0.12	300	275	1.09	3.85	366	12-D16	158
Arakawa 1987, No. 11	28.7	430	0.24	300	275	1.09	3.85	366	12-D16	188
Arakawa 1987, No. 18	31.1	215	0.11	450	275	1.64	3.85	363	12-D16	132

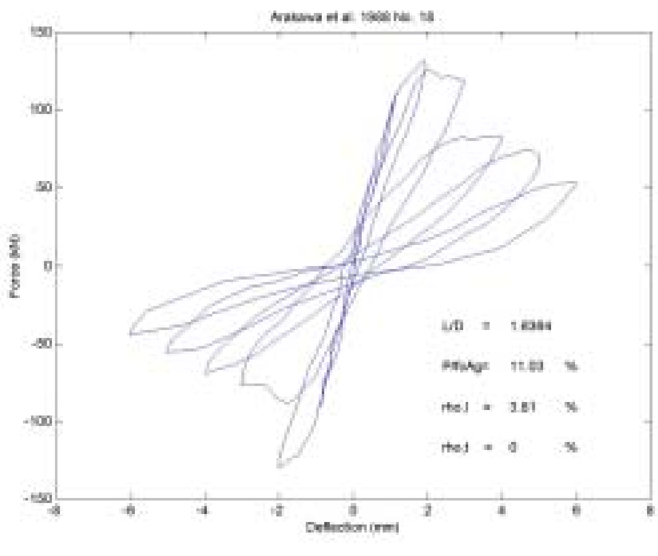




図ー1　試験体の詳細


Arakawa et al. 1967 No. 3
Force (kN)
Deflection (mm)
L/D = 1.0909
P/fcAg= 11.99 %
rho.l = 3.81 %
rho.t = 0 %


Arakawa et al. 1967 No. 11
Force (kN)
Deflection (mm)
L/D = 1.0909
P/fcAg= 23.9 %
rho.l = 3.81 %
rho.t = 0 %


Arakawa et al. 1969 No. 18
Force (kN)
Deflection (mm)
L/D = 1.6364
P/fcAg= 11.03 %
rho.l = 3.81 %
rho.t = 0 %

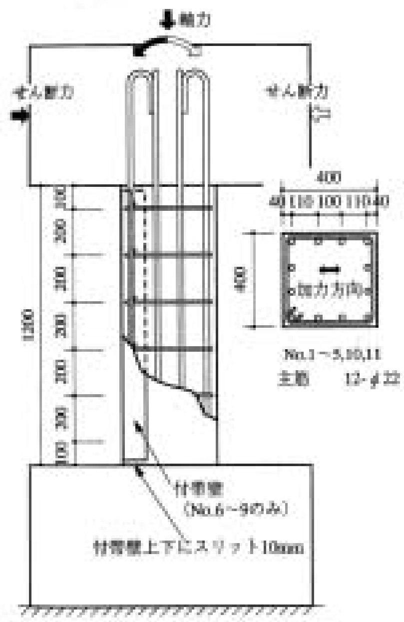
APPENDIX C

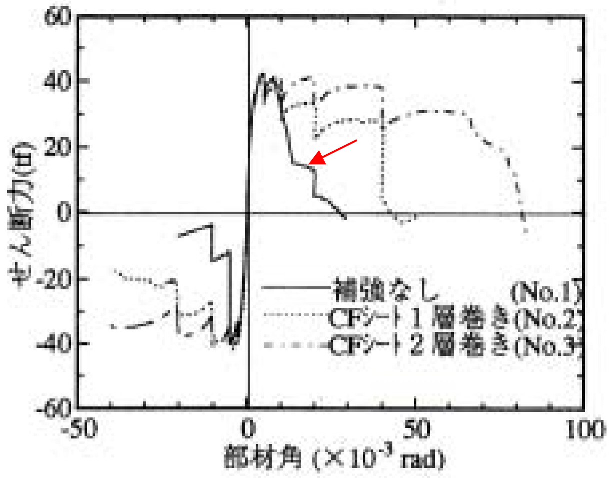
A SUMMARY OF EXPERIMENTAL RESULTS OF OCTAGONAL RC COLUMNS

<u>Jinno et al. 1996</u>

Jinno et al. (1996), Strengthening of RC structure designed according to the former standards (Part II Strengthening of columns with steel jacket and carbon fiber sheet), *Summary of Technical Papers of Annual Meeting, AIJ*, pp. 335-336

Specimen	Concrete Strnegth	Axial Load	Axial Level	Geometry					Longitudinal Reinf.			Shear Strenth
				Width	Heigh	Shear Span			Reinf. Ratio	Yield	Bar Dia	
	fc'	N	N/Agfc'	b	H	a	d	a/d	ρ	fy	φ	Vc
	N/mm^2	KN		mm	mm	mm	mm		%	N/mm^2	mm	kN
Jinno 1996, No. 1	22.4	980	0.27	400	400	600	360	1.7	2.85	314	12-D22	417





<u>Kato et al. 1996</u>

Kato et al. (1996), A study on seismic retrofit of existing RC columns with carbon fiber sheets, Part 3 Results of shear reinforcing tests, and Part 4 Shear strength of column, *Summary of Technical Papers of Annual Meeting, AIJ*, pp. 157-160

Specimen	Concrete Strnegth	Axial Load	Axial Level	Geometry					Longitudinal Reinf.			Shear Strenth
				Width	Heigh	Shear Span			Reinf. Ratio	Yield	Bar Dia	
	fc'	N	N/Agfc'	b	H	a	d	a/d	ρ	fy	φ	Vc
	N/mm^2	KN		mm	mm	mm	mm		%	N/mm^2	mm	kN
Kato 1996, A1-1	28.5	513	0.20	300	300	450	260	1.7	3.82	611	12-D19	210
Kato 1996, A2-1	24.5	441	0.20	300	300	300	260	1.2	3.82	611	12-D19	217
Kato 1996, A3-1	24.3	437	0.20	300	300	600	260	2.3	3.82	611	12-D19	160

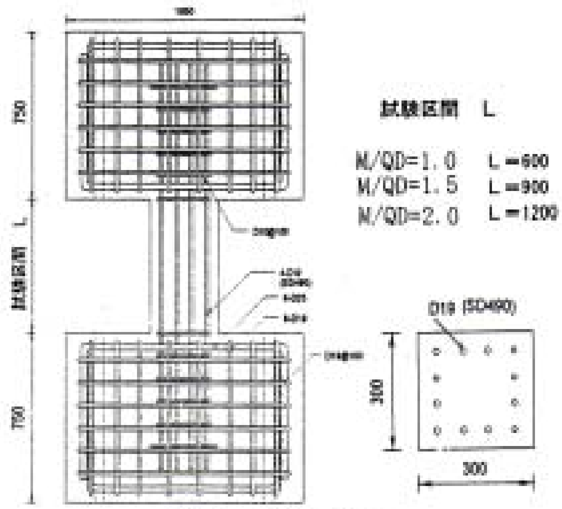
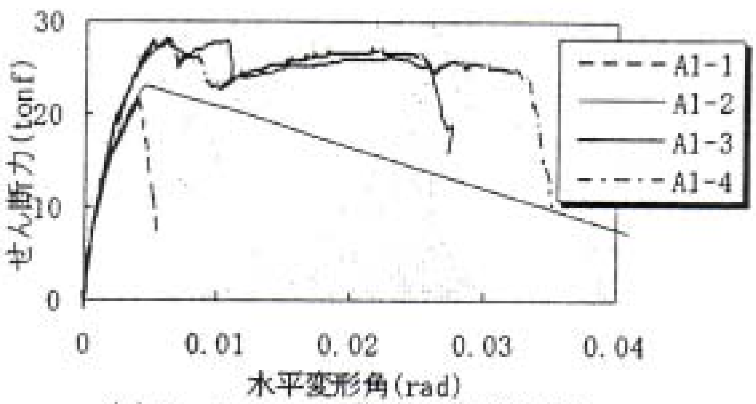
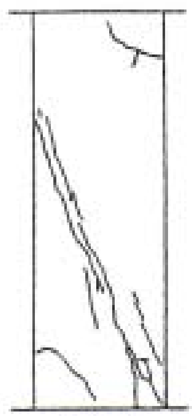


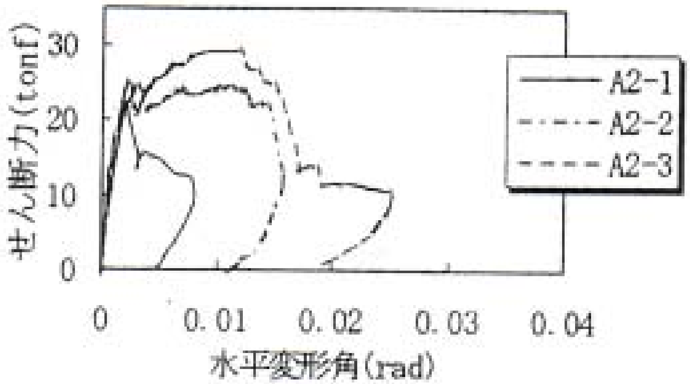
図2 試験体形状



(a)A1-1～A1-4 荷重－変形関係

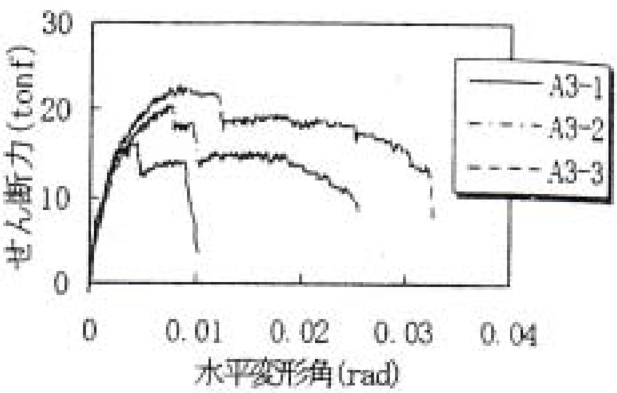


A1-1



(b)A-2 荷重一変位関係


A2－1



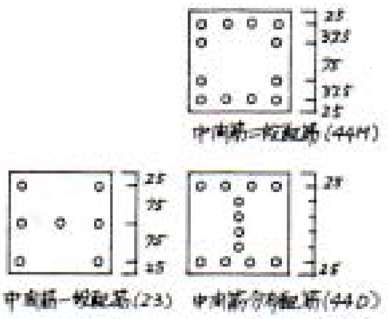
(c)A-3 荷重一変位関係


A3－1

Tadehara 1985

Tadehara, S. (1985), Experimental study on shear strength of concrete members with multi-layered reinforcement-Influence of volume of top and bottom reinforcement, *Summary of Technical Papers of Annual Meeting, AIJ*, pp.523-524

Specimen	Concrete Strnegth	Axial Load	Axial Level	Geometry					Longitudinal Reinf.			Shear Strenth
				Width	Heigh	Shear Span			Reinf. Ratio	Yield	Bar Dia	
	fc'	N	N/Agfc'	b	H	a	d	a/d	ρ	fy	φ	Vc
	N/mm^2	KN		mm	mm	mm	mm		%	N/mm^2	mm	kN
Tadehara 1985, 22	22.7	0	0	200	200	200	175	1.14	2.98	361	6-D16	88
Tadehara 1985, 32	22.9	0	0	200	200	200	175	1.14	3.97	361	8-D16	83
Tadehara 1985, 42	22.9	0	0	200	200	200	175	1.14	4.96	361	10-D16	92
Tadehara 1985, 42M	22.9	0	0	200	200	200	175	1.14	4.96	361	10-D16	94
Tadehara 1985, 44M	23.0	0	0	200	200	200	175	1.14	5.96	361	12-D16	97
Tadehara 1985, 44D	23.0	0	0	200	200	200	175	1.14	5.96	361	12-D16	93



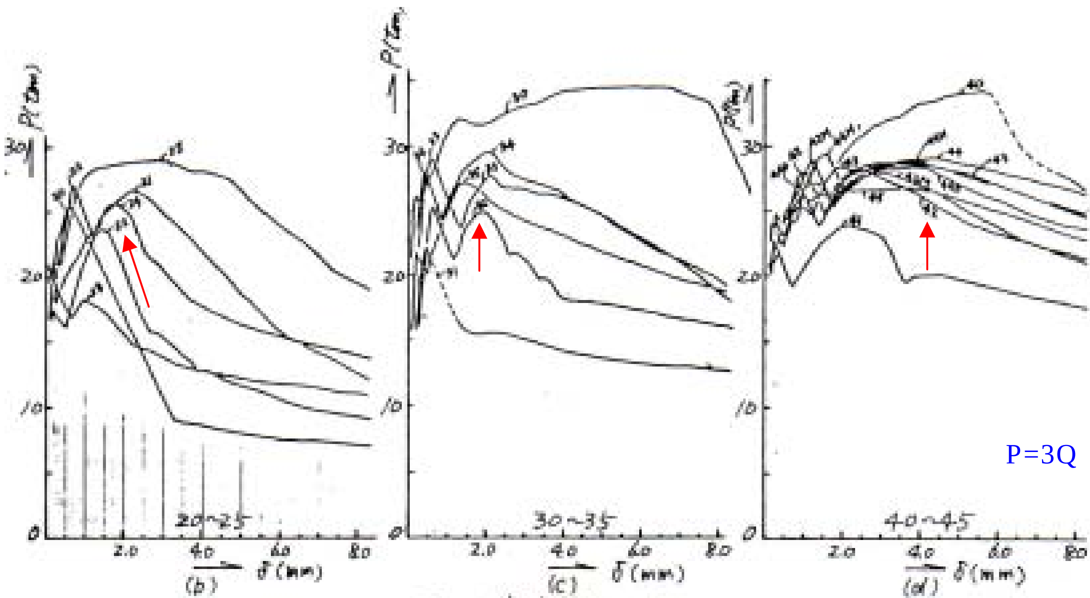


図1. 荷重変位曲線

Kokusho et al. (1978)

黒正清治，林静雄，雅己能森，小川幸雄 (1978)，円形断面を有する鉄筋コンクリート柱のせん断性状に関する実験，日本建築学会大会学術講演梗概集（北海道），pp.1737-1738.

Specimen	Concrete Strnegth fc' N/mm^2	Axial Load N KN	Axial Level N/Agfc'	Geometry Width b mm	Geometry Heigh H mm	Geometry Shear Span a mm	Geometry d mm	Geometry a/d	Longitudinal Reinf. Reinf. Ratio ρ %	Longitudinal Reinf. Yield fy N/mm^2	Longitudinal Reinf. Bar Dia ϕ mm	Shear Strenth Vc kN
Kokusho et al. 1978, R-10-0	25.3	0	0	220	220	200	197.6	1.01	2.09	340	8-D13	122
Kokusho et al. 1978, R-15-0	25.2	0	0	220	220	300	197.6	1.52	2.09	340	8-D13	74
Kokusho et al. 1978, R-20-0	25.3	0	0	220	220	400	197.6	2.02	2.09	340	8-D13	54
Kokusho et al. 1978, R-30-0	25.3	0	0	220	220	600	197.6	3.04	2.09	340	8-D13	49

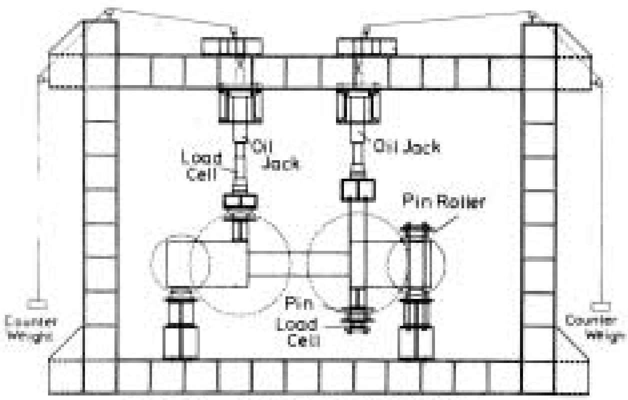
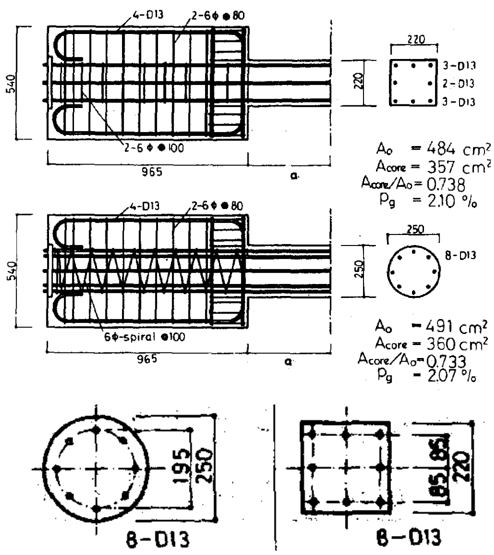


Fig. 2 Loading Arrangement



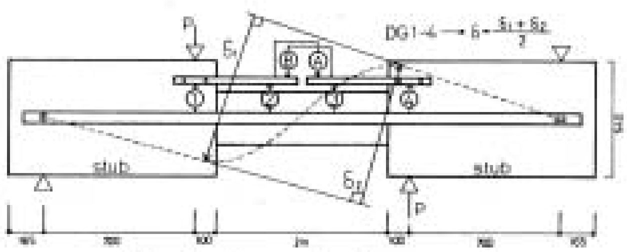
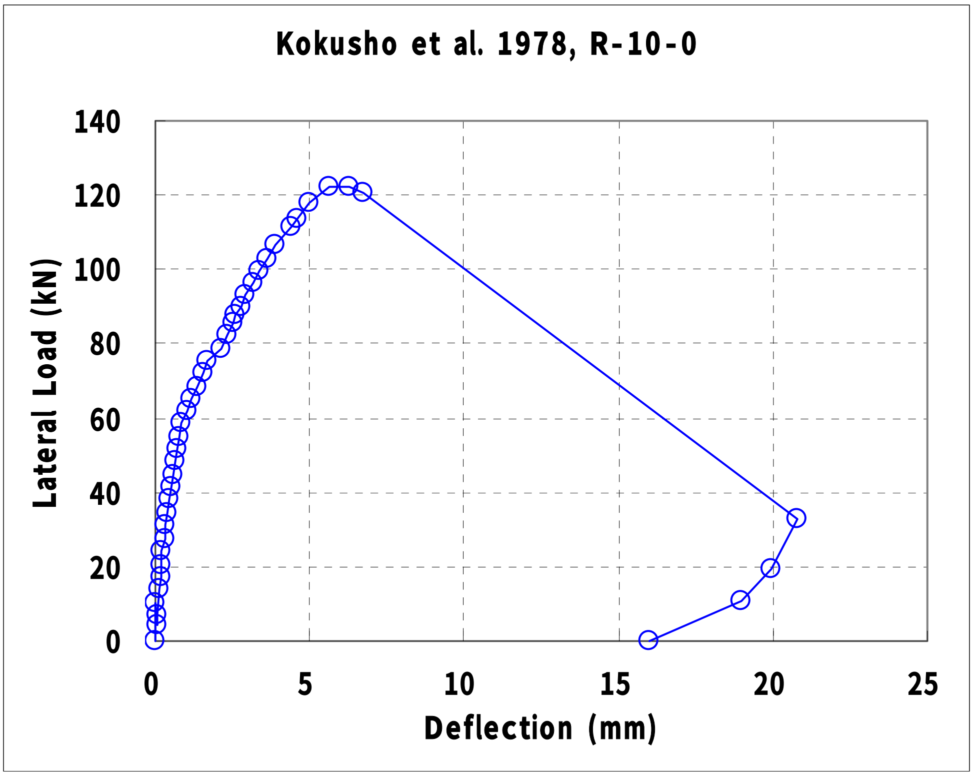
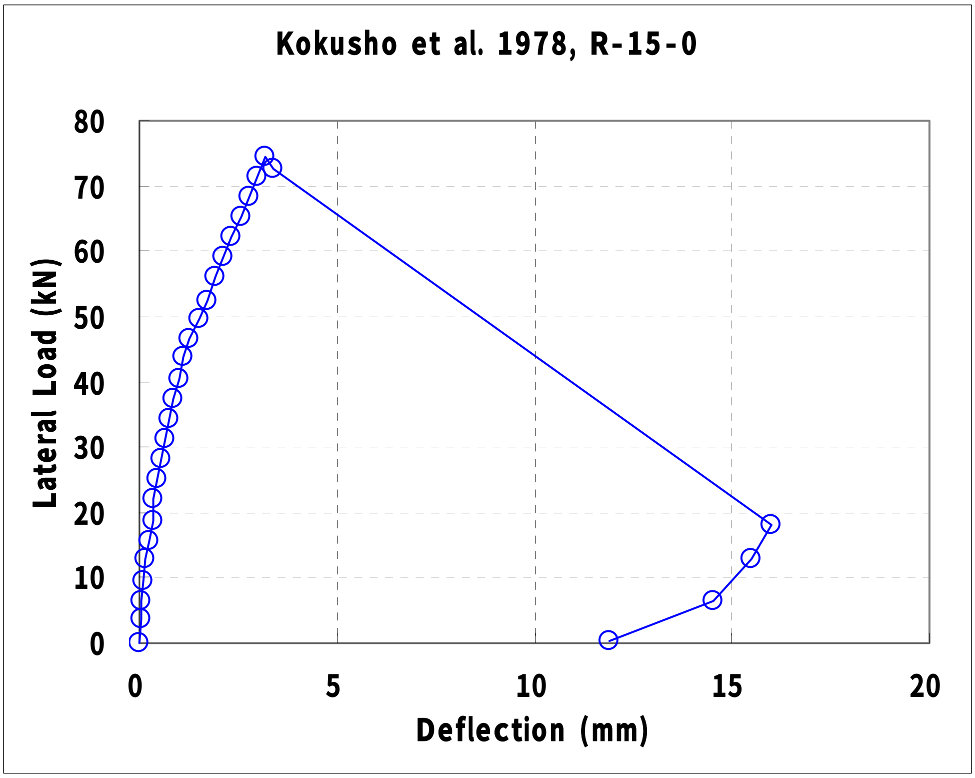
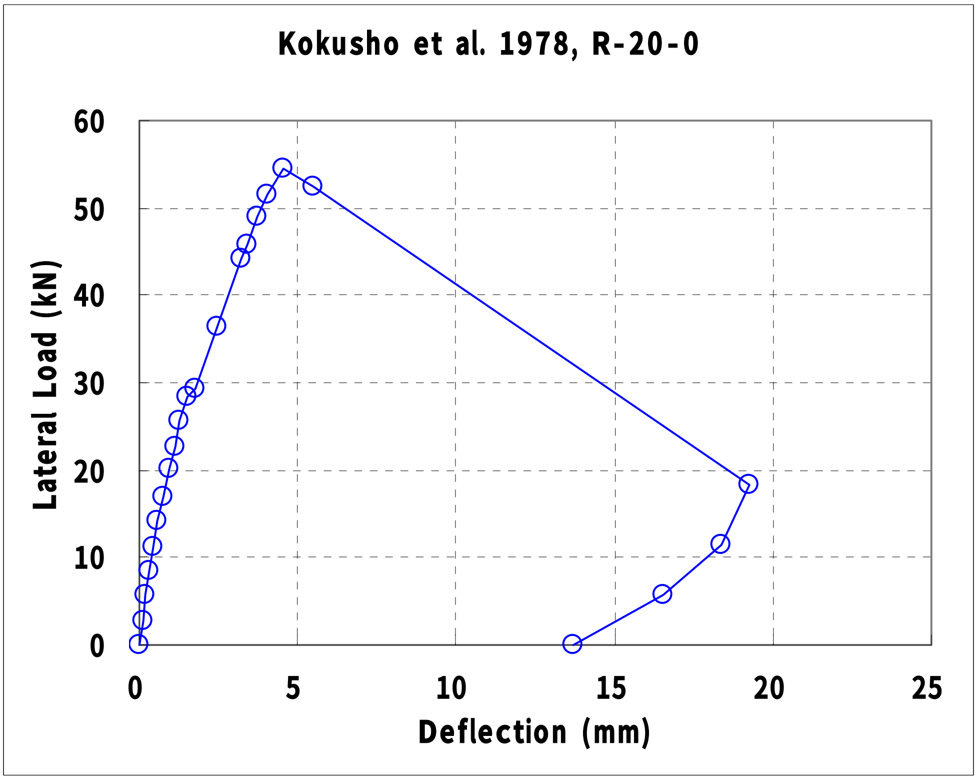
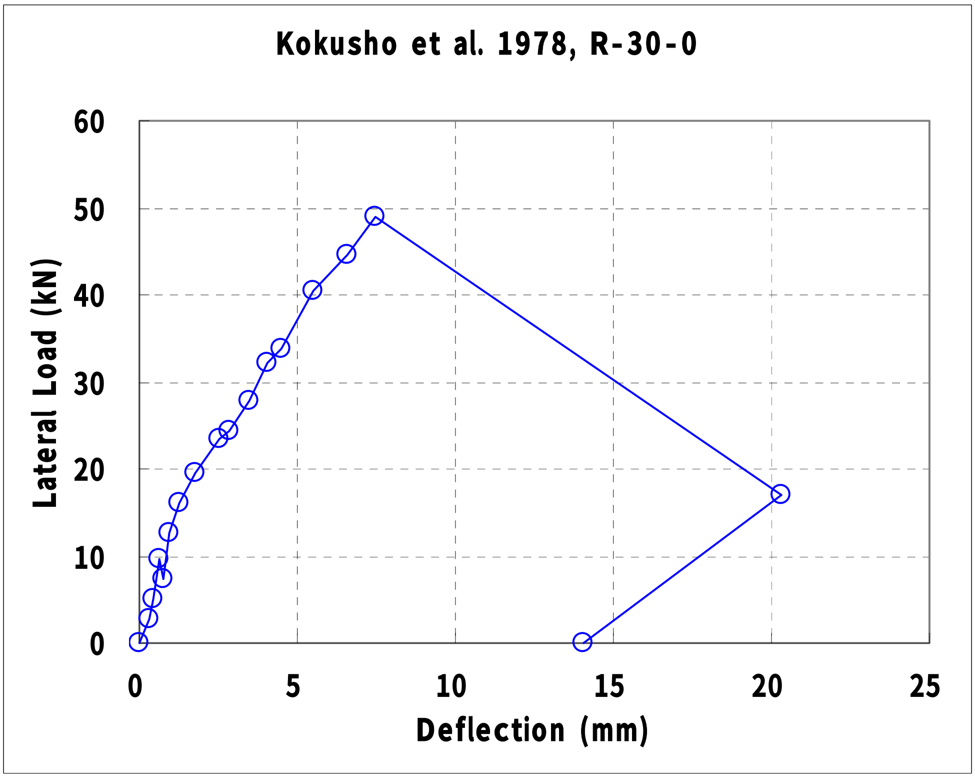


Fig. 3 Measuring Arrangement


Kokusho et al. 1978, R-10-0
Lateral Load (kN)
140
120
100
80
60
40
20
0
0
5
10
15
20
25
Deflection (mm)


Kokusho et al. 1978, R-15-0
Lateral Load (kN)
80
70
60
50
40
30
20
10
0
0
5
10
15
20
Deflection (mm)


Kokusho et al. 1978, R-20-0
Lateral Load (kN)
60
50
40
30
20
10
0
0
5
10
15
20
25
Deflection (mm)


Kokusho et al. 1978, R-30-0
Lateral Load (kN)
60
50
40
30
20
10
0
0
5
10
15
20
25
Deflection (mm)

<u>Yamamoto 1972</u>

山本幹夫，小島雅樹，荒川卓（1972），鉄筋コンクリート柱のせん断抵抗に及ぼす補強筋の結果，日本建築学会大会学術講演梗概集（九州），pp.1085-1086.

Specimen	Concrete Strnegth fc' N/mm^2	Axial Load N KN	Axial Level N/Agfc'	Geometry Width b mm	Heigh H mm	Shear Span a mm	d mm	a/d	Longitudinal Reinf. Reinf. Ratio ρ %	Yield fy N/mm^2	Bar Dia ϕ mm	Shear Strenth Vc kN
Yamamoto 1972, IS25-0-20	23.8	126	0.08	252	255	450	222	2.0	4.26	445 & 468	4D19+8D16	107
Yamamoto 1972, IS25-0-20	23.8	252	0.16	252	255	450	222	2.0	4.26	446 & 468	4D19+8D16	126

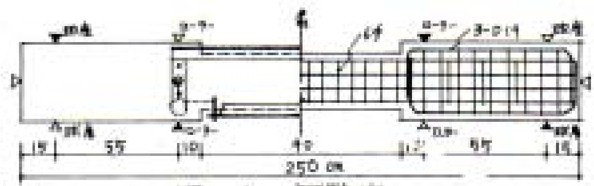
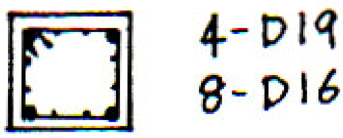


図-1 試験体

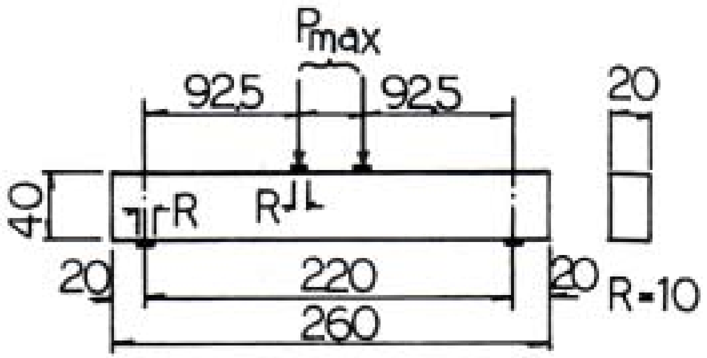
表-1 試験体の種類



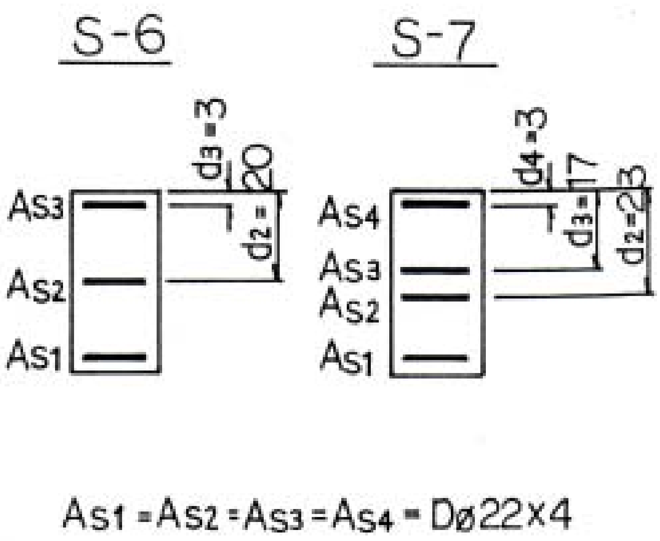
Ishibashi 1985

石橋忠良，斉藤恵一，寺田年夫 (1985b)，RC はりの腹部に配置された軸方向鉄筋のせん断力に及ぼす影響について，土木学会第 40 回年次講演概要集，pp.321-322.

Specimen	Concrete Strnegth	Axial Load	Axial Level	Geometry					Longitudinal Reinf.			Shear Strenth
				Width	Heigh	Shear Span			Reinf. Ratio	Yield	Bar Dia	
	fc'	N	N/Agfc'	b	H	a	d	a/d	ρ	fy	ϕ	Vc
	N/mm^2	KN		mm	mm	mm	mm		%	N/mm^2	mm	kN
Ishibashi 1985, S6	34.1	0	0	200	400	925	370	2.50	5.81	-	12-D22	134
Ishibashi 1985, S7	40.6	0	0	200	400	925	370	2.50	7.74	-	16-D22	145



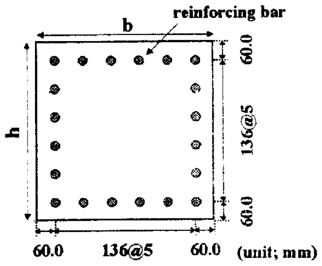
図－1 供試体形状および載荷方法 (cm)

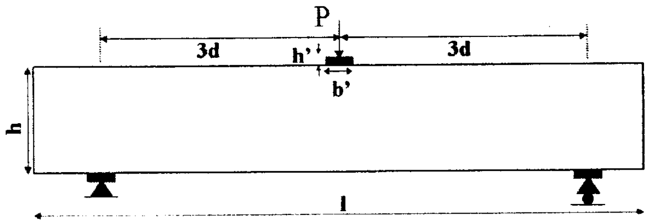


Tsuchiya 2001

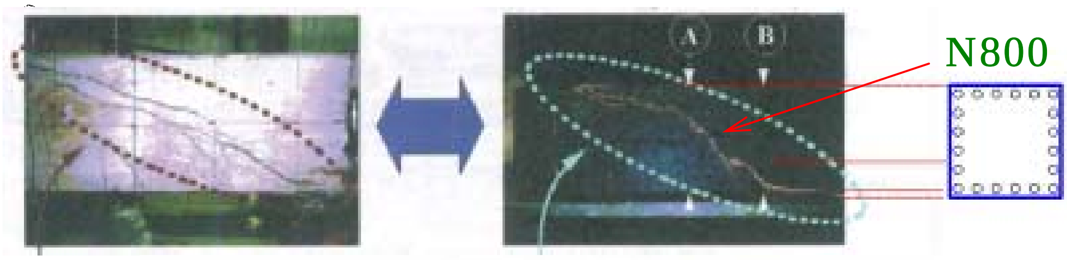
Tsuchiya, S., Mishima, T., and Maekawa, K. (2001), Shear failure and numerical performance evaluation of RC beam members with high strength materials, *Journal of Materials, Concrete Structures and Pavements, JSCE*, Vol.697/V-54, pp.65-84.

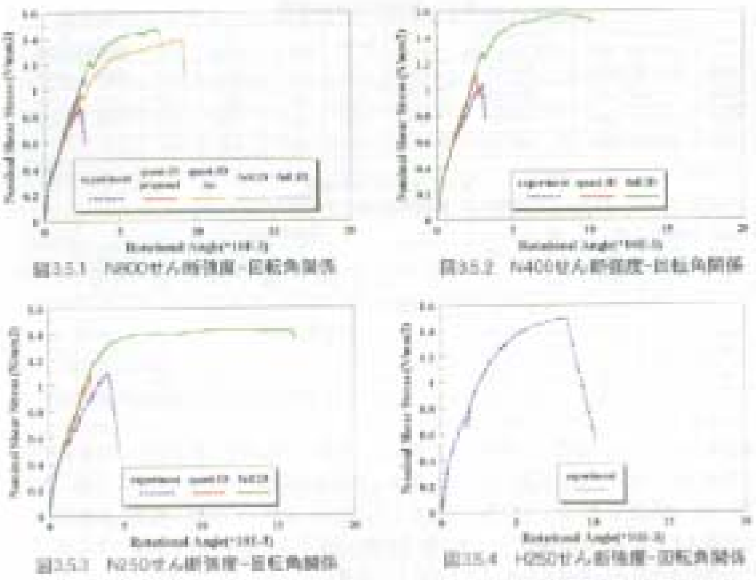
Specimen	Concrete Strnegth	Axial Load	Axial Level	Geometry			Longitudinal Reinf.			Shear Strenth
	fc'	N	N/Agfc'	Shear Span a	d	a/d	Reinf. Ratio ρ	Yield fy	Bar Dia ϕ	Vc
	N/mm^2	KN		mm	mm		%	N/mm^2	mm	kN
Tsuchiya 2001, N800	29.7	0	0	2220	740	3.00	2.48	351	20-D32	519
Tsuchiya 2001, N400	29.7	0	0	1110	370	3.00	2.48	353	20-D16	153
Tsuchiya 2001, N250	29.7	0	0	693	231	3.00	2.28	346	20-D10	64
Tsuchiya 2001, H250	58.7	0	0	693	231	3.00	2.28	346	20-D10	87





数字は N800 のもの(以下 1/2, 5/16 倍する)

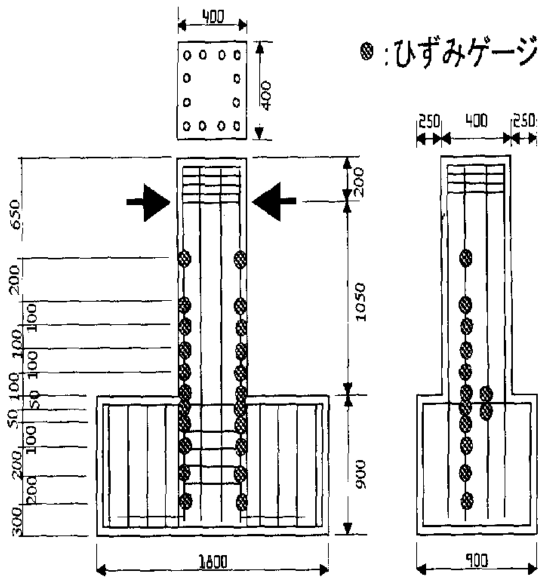




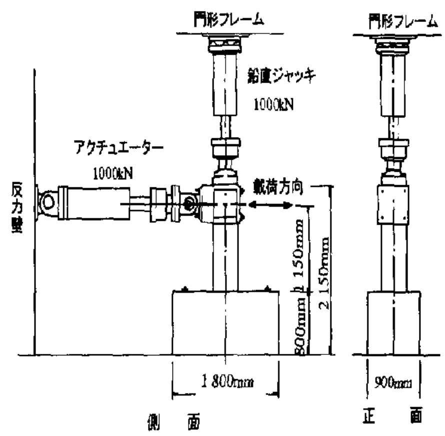
Ishibashi 2001

Ishibashi, T., Nakayama, Y., and Tsuyoshi, T. (2001), Failure mode of reinforced concrete column without hoop reinforcement, *Journal of Materials, Concrete Structures and Pavements, JSCE,* No.676/V-51, pp.13-18.

Specimen	Concrete Strnegth	Axial Load	Axial Level	Geometry			Longitudinal Reinf.			Shear Strenth
				Shear Span			Reinf. Ratio	Yield	Bar Dia	
	fc'	N	N/Agfc'	a	d	a/H	ρ	fy	ϕ	Vc
	N/mm^2	KN		mm	mm		%	N/mm^2	mm	kN
Ishibashi 2001, I-1	27.2	157	0.036	1150	360	3.19	3.58	374.8	16-D19	216
Ishibashi 2001, I-6	23.2	157	0.042	1150	360	3.19	3.58	370.1	16-D19	212



図－1 試験体の配筋例



図－2 載荷装置

写真－1 試験体 I－1