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IV

Experimental Research on Ductility of Reinforced Concrete Short Columns under Cyclic Lateral Loads

Recherche expérimentale sur la ductilité de colonnes courtes en béton armé soumises à des charges latérales répétées

Experimentelle Untersuchungen über das Verformungsvermögen von kurzen Stahlbetonstützen unter wiederholten Querbelastungen

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1. INTRODUCTION

In such a country as Japan where severe earthquakes occur so often, structural safety of buildings is dominated distinguishly by earthquake. It is gradually becoming apparent by many strong motion earthquake observations, response analyses and earthquake damage observations that the influence of earthquake to structures is fairly superior to the simulated earthquake load which is usually regulated as design seismic force coefficient in many countries. Actually, at the Tokachi-Oki earthquake, 1968, several buildings, whose yield shear coefficients are more than 0.5, were destroyed.

It is generally impossible and uneconomical to make all buildings so strong as to resist severe shocks only by their strength. However, it also became apparent by many response analyses that the buildings with adequate strength and full ductility can survive even at a severe shock.

As it is said that the reinforced concrete buildings with short length columns are considered to fail in brittle way, synthetic experimental research was started with the objective, how to make the short columns ductile. This research is sponsored by Ministry of Construction of Japan and a committee was organized for the execution of this big project. This committee was consisted from not only official researchers but also ones belonging to technical branches of general constructions companies. The members are refered to §7 ACKNOWLEDGEMENT.

About the results on 125 specimens, failure mode and factors which affect ductility are discussed in this report.

2. EPITOME OF EXPERIMENT

2.1 Master Plan for Experiment

Mean unit axial stresses of the columns at the first story due to permanent load in usual reinforced concrete low stories buildings in Japan, are about 40 kg/cm^2. Assuming 0.4 as yield shear coefficient, mean unit shear stresses at a severe shock are generally less than 20 kg/cm^2. The web reinforcements required for such a stress condition as above will not become excessively much and it will be not so difficult to execute them well.

Accordingly, combinations of tensile reinforcement ratio and shear span ratio were selected at first so that their maximum flexural capacities do not become excessively large under constant axial stresses 40 kg/cm^2. Next such reinforcing details as quantity, spacing and shape of web reinforcement and arrangement of axial reinforcement were selected.

2.2 Outline of Tested Series and Specimens

1) Variable Factors

Based on the past experimental research results, the committee selected ten factors affecting ductility. They are shown in Table 1 as from F1 to F10. Further, after investigations for these factors of many existing reinforced concrete buildings, standard values and characteristics for several variables were decided as shown in Table 1.

Out of them, how to decide the standard size of cross section related to scale effect, web reinforcement ratio, loading cycling and loading apparatus are described in the followings.

2) Scale Effect

There seems not to be the reliably available data concerning scale effect. And further, due to the limited budget and facilities, it is practically impossible to carry out all the tests in full scale. So, the standard size of cross section was decided to be 25 cm square and one series were carried out using 50 cm square cross section to discuss scale effect.

3) Web Reinforcement Ratio

The method to calculate reasonable web reinforcement ratio for ductile column has not been established up to date. So, it was decided to use tentatively the Arakawa's minimum equation (Cf. § 4.2, eq. (1)) to set the standard web reinforcement ratio. This equation is the experimental one showing the minimum shear strength of reinforced concrete members without axial force and was used as a back datum to decide the regulations of A.I.J. standard in Ref. 1) for shear reinforcement. One of the standard value of web reinforcement ratio was set as P_{w1} which can be obtained by equating the flexural capacity $_{C}Q_{FU}$ (Cf. § 4.2, eq. (2)) to the Arakawa's minimum equation. This aimed to raise the shear strength of the column up to the flexural strength, because the flexural yielding shows good ductility. And, the half of P_{w1} was adopted as another standard value as the example of poor shear strength.

TABLE 1 LIST OF FACTORS COMMON TO SPECIMEN

	COMMON FACTORS	NOTE
F1	Section: b x D = 25 x 25 cm, d_t=d_c=3.5 cm	SE Series: b x D = 50 x 50 cm, d_t=d_c=7.0 cm
F2	Concrete: Normal concrete, F_c=210 kg/cm^2 (Design)	FC Series: F_c = 350 kg/cm^2 (Design)
F3	Shape of web reinforcement: Rectangular hoop with a standard hook at each end	PILOT Series: Welded rectangular hoops used in the half of specimens
F4	Web bar: SR24 Round bar	Specified yield point of 2400 kg/cm^2
F5	Axial bar: SD35 Deformed bar	Specified yield point of 3500 kg/cm^2
F6	Tensile reinforcement ratio: p_t=0.34% (3-D10), 0.61% (3-D13), 0.95% (3-D16)	PILOT Series: p_t= 0.41% (2-D13), 1.24% (2-D22) AF Series: p_t=1.38% (3-D19)
F7	Shear span ratio M/QD: M/QD = 1 and 2	PILOT and LS Series: M/QD = 1.5 and 3.0
F8	Axial stress: N/bD = 210/4 and 210/8 kg/cm^2	30 kg/cm^2 (PILOT S.), 0 and -210/10 kg/cm^2 (AF S.)
F9	Web reinforcement ratio: by Arakawa's min. Equation	PILOT Series: by A.I.J. Equation
F10	Loading method: Restrained column type, reversal loading of multi cycles	WAKABAYASHI-type (PILOT S.), Inverse-Symmetrical-type (LM1, SE S.), B.R.I.-type (other series)

4) Loading Excursion

Until a certain rational dynamic method is established for estimating seismic properties of structural members, it seems difficult to discuss them by some special vibration test results. Accordingly we must adopt a certain rational, static, cyclic loading method. In this research, such a multi-cyclic alternate loading method as shown in Fig. 1 was adopted as standard loading excursion. This mainly depends on the following reasons.

a) Ductility factors μ obtained through response analyses of not so highrized buildings at severe shocks are about 3 or 4 at most, if their strength and ductility are not excessively small.

b) The number of acceleration responses corresponding to more than 80% of the maximum value has been reported to be about 10 times in some cases.

c) Structural behavior of members under cyclic loading at constant deflection tends to become apparent within 10 cycles.

5) Loading System

In most of past experiment, simple beam systems are used world-widely as the loading system for structural members. However, this method is not so good to discuss seismic behavior of reinforced concrete column, because of the following reasons.

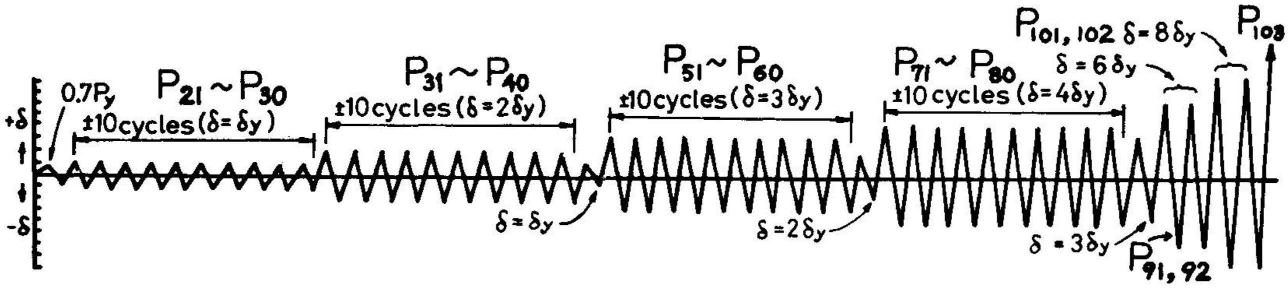
a) When discussing shear and bond problems of members, restrained column type is more similar to the real condition of columns in actual buildings during an earthquake than simple beam is.

b) It is more similar to the real condition that both sections at column ends keep pararel to each other than that they have some inclinations.

c) Discussing behaviors under large deflection, it is preferable that the influence of additional moment due to eccentric axial load can be easily estimated.

d) It is preferable that many cycles of load reversals can be easily carried out and that developing of cracks can be easily observed and recorded.

So, such a new loading apparatus as shown in Fig. 2 was developed and was used to test many series. Still more, in order to discuss the influence of loading systems, two series were carried out by continuous beam system.



P_{mn}: Load at test equals shear force Q of specimen, (mn; loading number).

P_y: Load when tensile reinforcement yields at test, ($P_y = P_{21}$).

δ_y : Measured horizontal displacement at yielding.

FIG. 1 LOADING PROCEDURE USED IN ALL SPECIMENS

2.3 Constitution of Typical Series and Specimens

On the basis of the previous discussions, it was decided that a typical series is constituted from 15 specimens taking 3 kinds of tensile reinforcement ratio p_t, 2 kinds of shear span ratio M/QD, 2 kinds of axial compression stress σ_o and 2 kinds of web reinforcement ratio p_w as variable factors. A list of specimens belonging to a typical series is shown in Table 2. In other series, each common factors, for example $c\sigma_B$ or size of section etc., was changed one by one. Fig. 3 shows an example of specimen details.

Ten series, including 165 specimens in total, as shown in Table 3 had been tested already and the results on the upper seven series in the table are discussed in this report. In Fig. 4, frequency distributions of σ_o, M/QD, p_t and p_w of the 125 specimens are shown with their failure modes and classified ductilities which are described as follows.

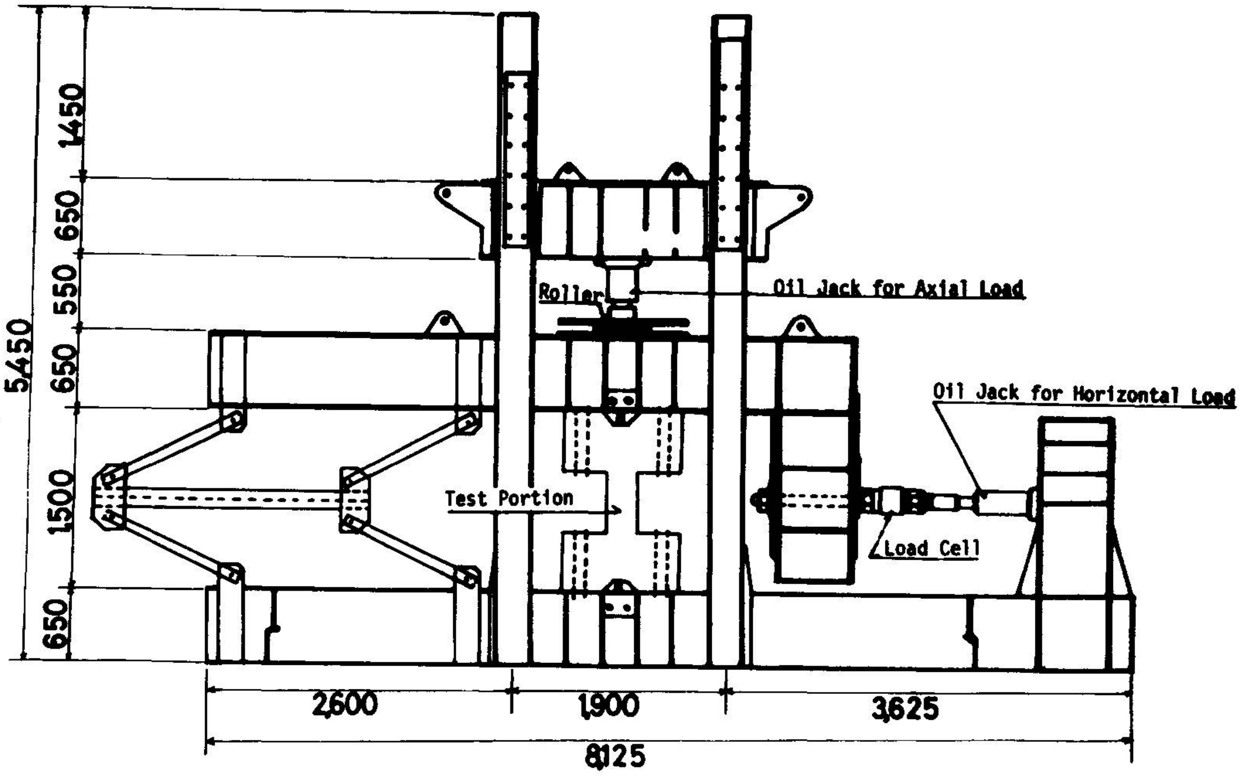


FIG. 2 B.R.I.-TYPE LOADING APARATUS

TABLE 2 LIST OF SPECIMENS BELONG TO A TYPICAL SERIES

Mark	Tensile Reinforcement	Tensile Reinforcement Ratio	Shear Span Ratio	Web Reinforcement	Web Reinforcement Ratio	Axial Unit Stress
	Number-Size	$p_t = a_t/bD$ (%)	$\frac{M}{QD}$	Number-Size-Spacing (mm)	$p_w = a_w/bs$ (%)	$\sigma_o = N/bD$ (kg/cm^2)
1B	3-D10	0.34	1	16- 9 - 33.3	1.53	210/4
2A	3-D10	0.34	2	15- 9 - 71.5	0.71	210/4
2B	3-D10	0.34	2	17- 6 - 62.5	0.36	210/4
3A	3-D10	0.34	1	10- 9 - 55.5	0.92	210/8
3B	3-D10	0.34	1	11- 6 - 50.0	0.45	210/8
4A	3-D10	0.34	2	10- 4 - 55.5	0.18	210/8
4B	3-D10	0.34	2	10- 4 - 111.1	0.09	210/8
5A	3-D13	0.61	1	12-13 - 45.5	2.33	210/8
5B	3-D13	0.61	1	12- 9 - 45.5	1.12	210/8
6A	3-D13	0.61	2	24- 6 - 43.5	0.51	210/8
6B	3-D13	0.61	2	28- 4 - 37.0	0.27	210/8
7A	3-D16	0.95	2	24- 4 - 43.5	2.44	210/4
7B	3-D16	0.95	2	25- 9 - 41.7	1.22	210/4
8A	3-D16	0.95	2	26- 9 - 40.0	1.27	210/8
8B	3-D16	0.95	2	28- 6 - 37.0	0.61	210/8

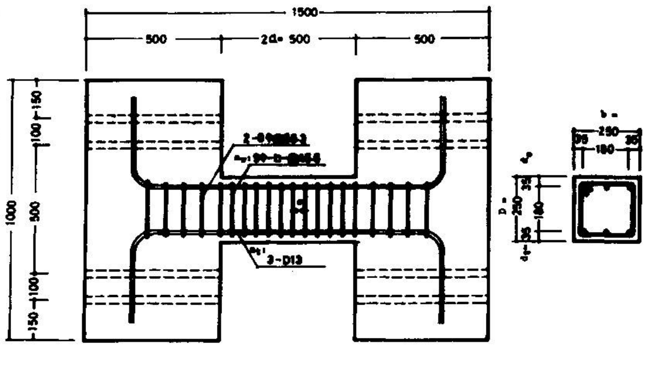


FIG. 3 AN EXAMPLE OF SPECIMEN DETAILS (Unit in mm)

TABLE 3 LIST OF TEST SERIES

Name of Series	Section of Specimen (cm x cm)	Shear Span Ratio (M/QD)	Loading *1 System	Institution *2 in Charge	Year	Main Objective	Number of Specimens
Pilot	25 x 25	1.5, 3.0	W - Type	B.R.I.	1971	Welded Hoop	36
LM1	25 x 25	1.0, 2.0	R,B - Type	T.I. of Takenaka Komuten	1972	Loading System Scale Effect	15
LM2	25 x 25	1.0, 2.0	BRI - Type	Tokyo Institue of Technology	1972	Loading System	15
SE	50 x 50	1.0, 2.0	R,B - Type	Meiji Univ. and B.R.I.	1972	Scale Effect	15
FC	25 x 25	1.0, 2.0	BRI - Type	T.I. of Taisei Const. Co., Ltd.	1972	Concrete Strength	14
WS	25 x 25	1.0, 2.0	BRI - Type	T.I. of Obayashi-Gumi Co., Ltd.	1973	Welded Hoop of Deformed Bar	15
AF	25 x 25	1.0, 2.0	BRI - Type	T.I. of Fujita Kogyo	1973	Axial Force	15
CW	25 x 25	1.0, 2.0	BRI - Type	Tokyo Metropolitan Univ.	1973	Columns with Side Wall	10
WS2	25 x 25	1.0, 2.0	BRI - Type	T.I. of Kajima Const. Co., Ltd.	1973	Spiral Hoop	15
LS	25 x 25	1.5, 3.0	BRI - Type	T.D.C. of Toda Const. Co., Ltd.	1973	Shear Span Ratio	15

*1 W - Type : Loading System devised by Dr. Wakabayashi
R,B - Type : Restrained Beam Type
BRI - Type : Loading System newly developed in B.R.I.

*2 T.I. : Technical Institute
T.D.C. : Technical Development Center

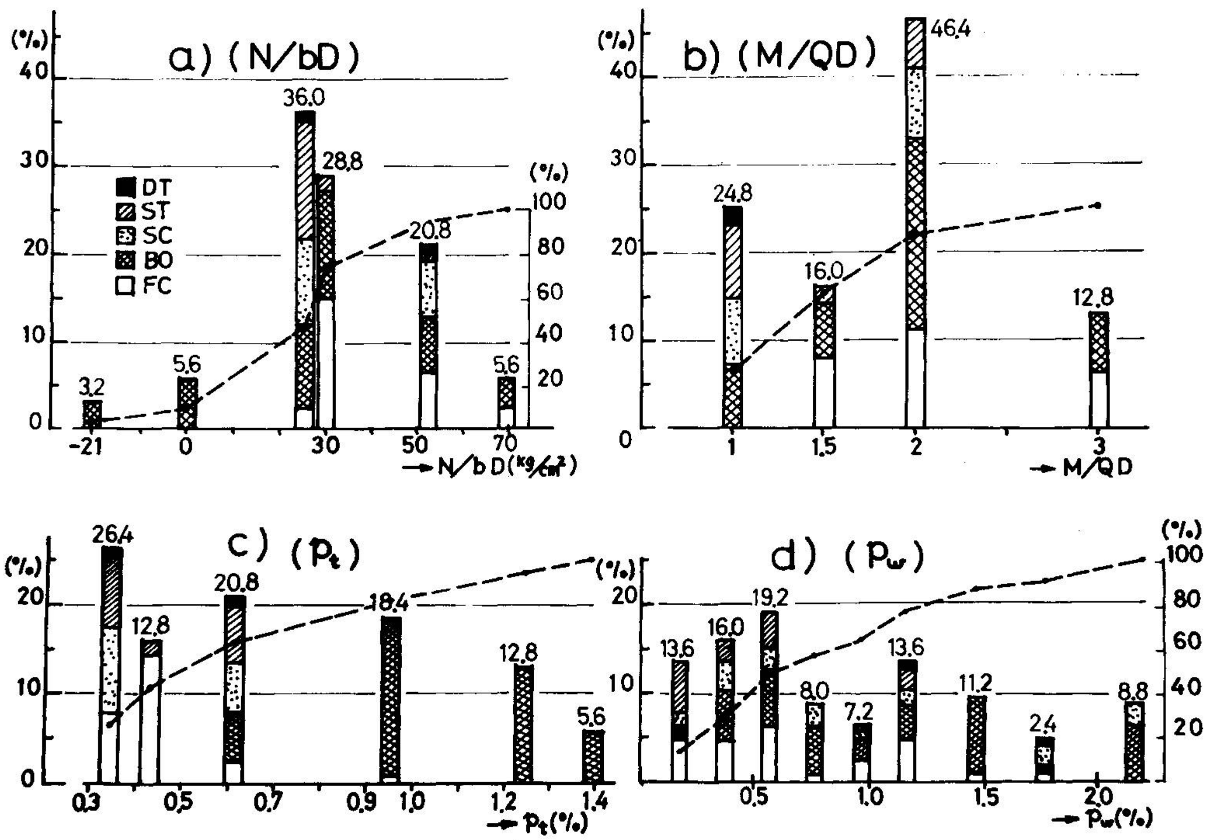


FIG. 4 FREQUENCY DISTRIBUTION OF σ_o, M/QD, p_t and p_w

3. OUTLINE OF TEST RESULTS

3.1 Failure Mode of Tested Specimens

Followings are the typical failure modes observed in the test.

1) Compression failure of concrete after flexural tensile yielding with or without compression steel buckling. - F (flexural yielding), F•C (after flexural yielding, failed by crushing of concrete), F•C•Bu (after flexural yielding and crushing, failed by buckling of compression bar).
2) Shear failure before or after flexural yielding. - S•DT (after shear cracking, failed by diagonal tension), S•ST (after shear cracking, failed by shear tension), F•ST (after flexural yielding, failed by shear tension), S•SC (after shear cracking, failed by shear compression), F•SC (after flexural yielding, failed by shear compression).
3) Bond split failure before or after flexural yielding. - B•BO (after bond cracking, failed by bond split), F•BO (after flexural yielding, failed by bond split).

3.2 Classified Ductility

In this report, test results were classified by the ductilities defined in Table 4, based on their critical deflections δ and their ductility factors δ/δ_y, (δ_y shows yeilding deflection).

3.3 Relationship between Failure Modes and Classified Ductilities

Test results of 125 specimens were classified by their failure modes and classified ductilities, and their relationships and their frequencies are shown in Fig. 5. As observed in these figures, the most ductile failure mode is F•C type and next one is S•C type. Many specimens which were failed in ST type and BO type showed poor ductilities. Accordingly, it becomes one of the important subjects how to prevent such brittle failure modes as SC type, ST type and B type.

TABLE 4 CLASSIFIED DUCTILITY

DUCTILITY	CHARACTERISTICS
A	Very ductile columns which failed by shear or by buckling of compression bars at horizontal large deflection. ($P_{21} \sim P_{91} \geqq 0.8{}_cQ_{FU}$, ${}_cQ_{FU}$: Flexural strength obtained by A.I.J. formula)
B	Ductile columns, whose deterioration of shear capacity were small untill $\mu = 4$, ($\mu = \delta / \delta_y$), but which failed by shear or by bond or by buckling of bar before $\mu = 6$. ($P_{21} \sim P_{71} \geqq 0.75{}_cQ_{FU}$)
C	Columns yielded by flexure at first, but deteriorated remarkably due to shear or bond failure or buckling of bars before they reached to large deflection. ($P_{21} \sim P_{31} \geqq 0.75{}_cQ_{FU}$)
D	Columns failed by shear or by bond before flexural yielding. (Others than A, B and C)

Also, load-deflection relationships, cracking patterns and shear force capacity deteriorations of the typical specimens are shown in Fig. 6, 7 and 8 respectively.

4. DISCUSSION ON TEST RESULTS

4.1 Axial Reinforcement Buckling

Out of 32 specimens which failed in F·C typed mode, bucklings of axial compression reinforcements were observed in 24 specimens. In 10 specimens, buckling occurred at the ductility factors $\mu = 2 \sim 4$ and caused the strength deterioration. Discussing these results, are as follows.

1) Lengths of buckling ℓ_k were around 1 ~ 2 times spacings of web reinforcement s. But in case of spiral hoops and welded square hoops, ℓ_k was nearly equal to s.
2) The relationship between slenderness ratio λ calculated assuming ℓ_k equals s, and the rotation angle ($R = \delta/2a$) of the member at buckling R_{BU} was investigated about all specimens. By this result, when λ is more than 34, R_{BU} becomes between 1/100 and 1/50, but when λ is less than 34, buckling did not occur until fairly large deflection.
3) The above results are expressed $s \leqq 8\phi$ where ϕ shows the diameter of compression reinforcement. Namely, it is preferable to space the web reinforcements at column ends less than 8 times diameter of axial reinforcement.

4.2 Shear Failure Mode

41 specimens failed by shear before or after flexure yielding. They can be divided to 10 specimens failed by diagonal tension, 12 by shear tension and 19 by shear compression.

1) General Discussion by means of Arakawa's Formula

As there is no quantitative equation concerning the relationship between ductility of members and web reinforcement ratio, general discussion was at first done by Arakawa's formula which was used as the ground to calculate web reinforcement of our specimens.

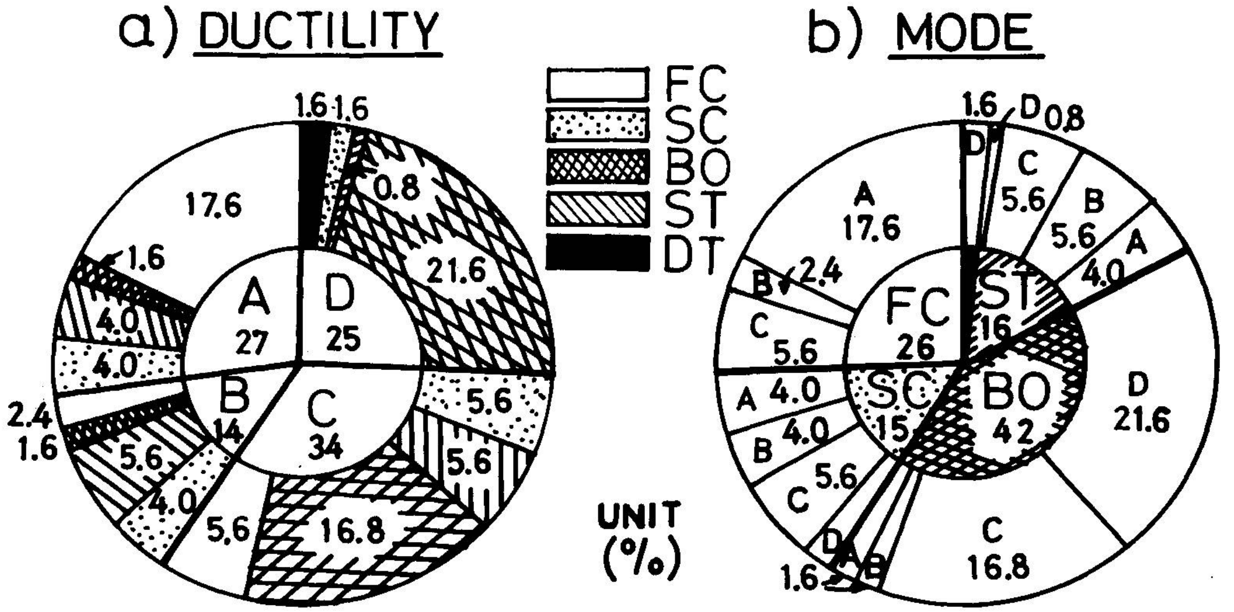
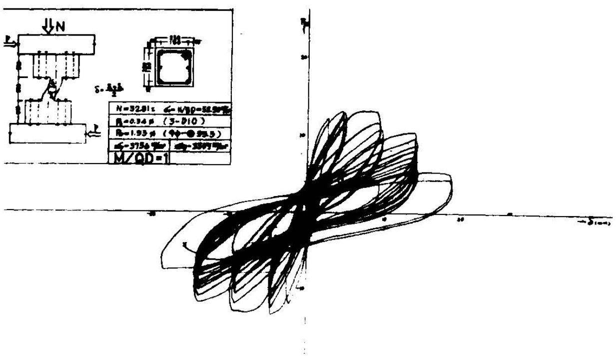
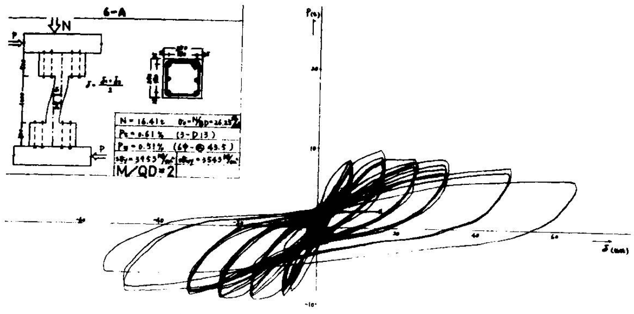


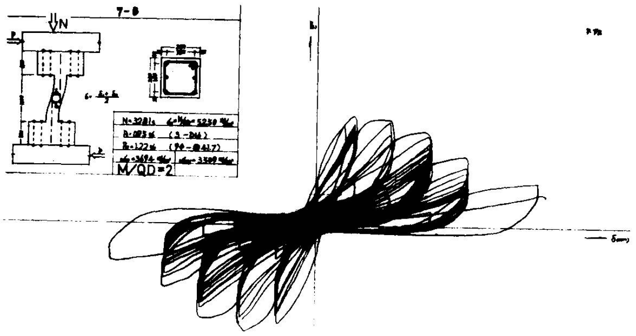
FIG. 5



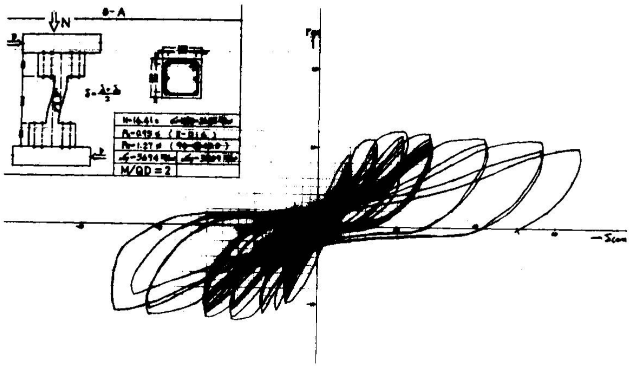
a) LM2-1B (S•DT, Ductility D)



b) LM2-6A (F•SC, Ductility A)



c) LM2-7B (F•BO, Ductility D)



d) LM2-8A (FC, Ductility B)

FIG. 6 LOAD - DEFLECTION CURVES

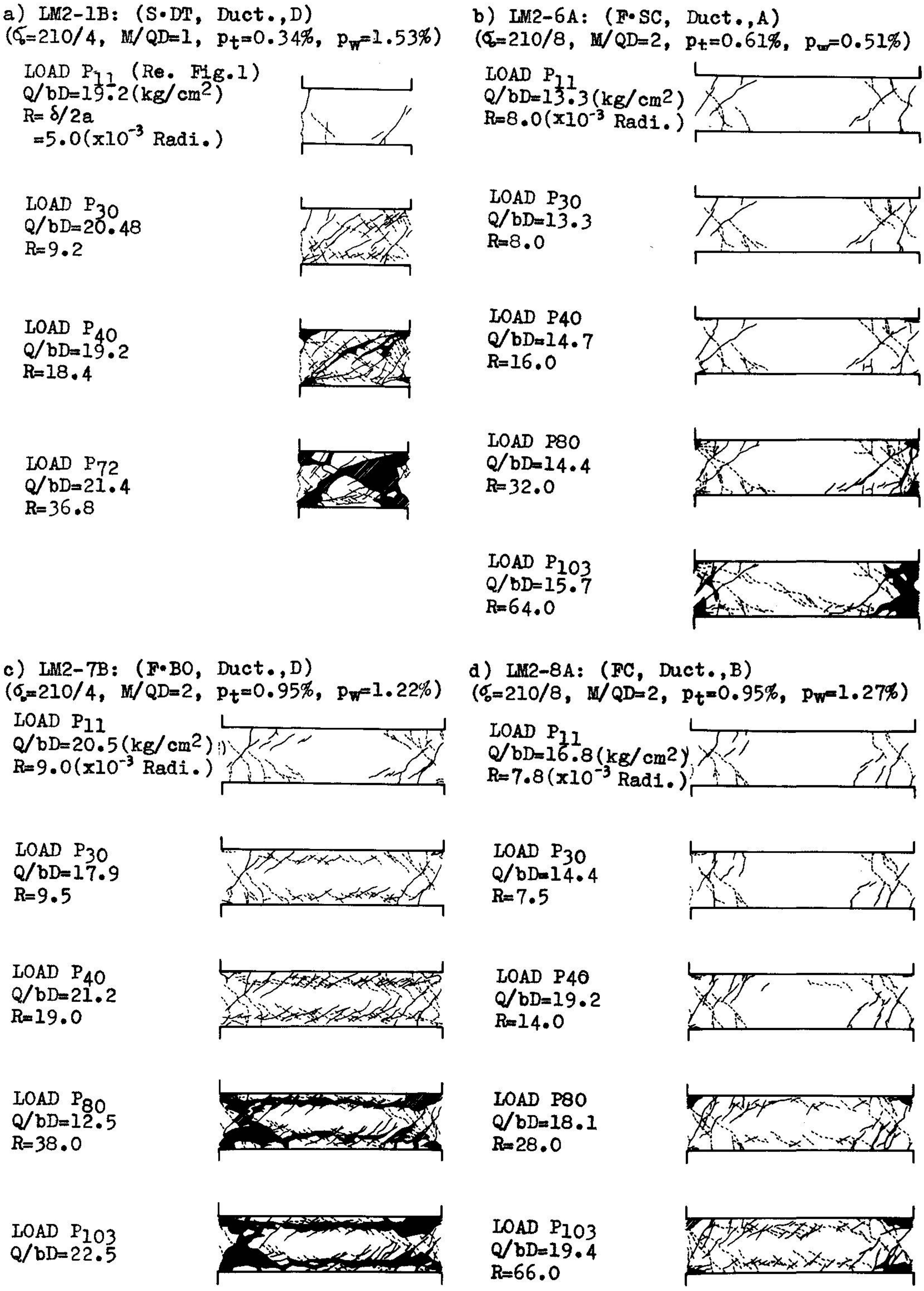


FIG. 7 CRACKING PATTERNS

Namely, we investigated the relationship between the classified ductility and ratio of calculated shear strength $_cQ_{ARA}$, to calculated flexure strength when the column yield at both ends $_cQ_{FU}$. The formulae used to calculate $_cQ_{ARA}$ and $_cQ_{FU}$ are shown in the followings. The notations used in the formulae are refered to §8.

$$_cQ_{ARA} = \frac{0.0754\ P_t^{0.23}\ K_U(_c\sigma_B + 180)}{M/Q_d + 0.12} + 2.7\sqrt{p_w \cdot {_s\sigma_{wy}}} \times \frac{7}{8}\ bd \qquad (1)$$

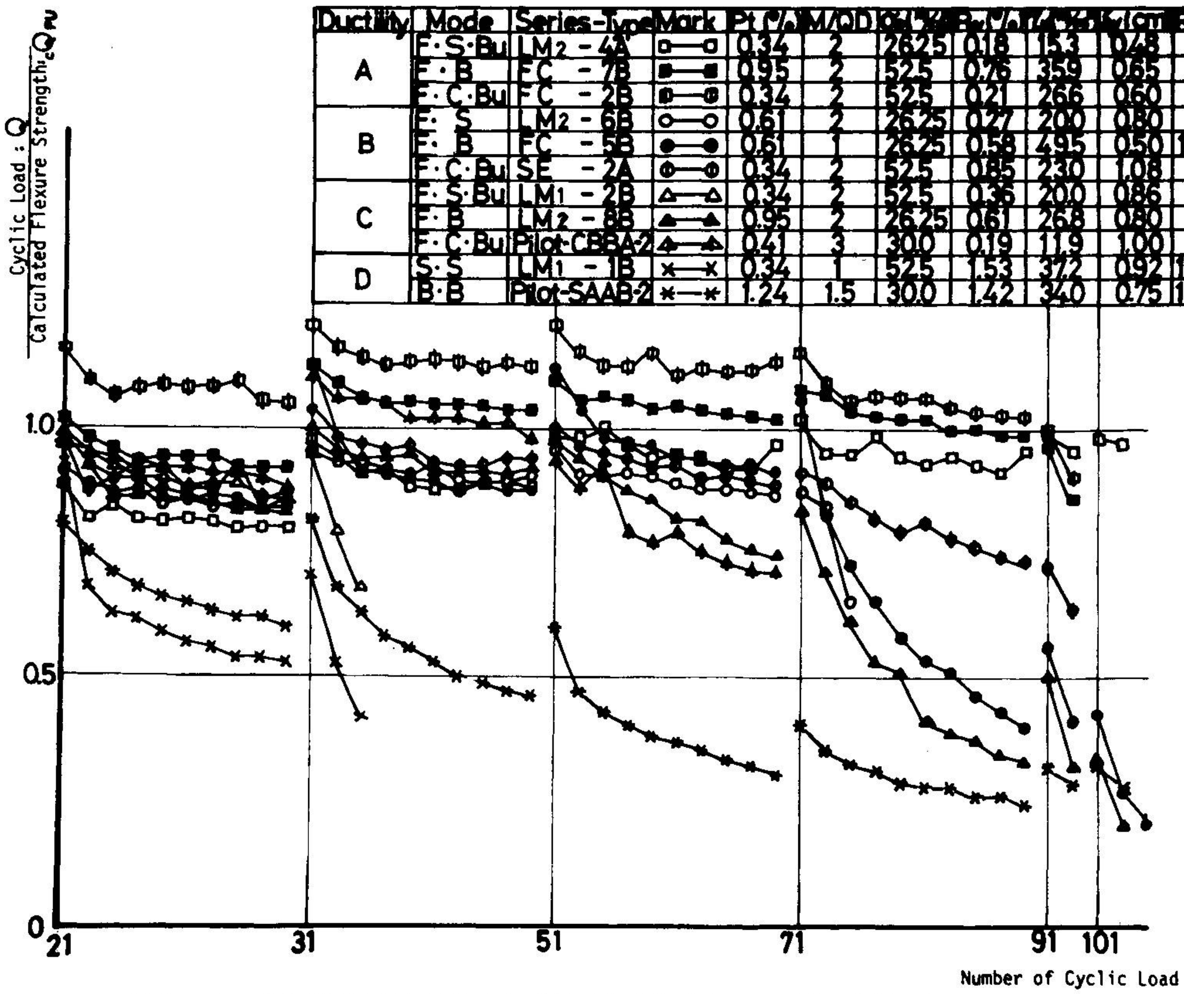
here K_U = 0.80 (d = 21.5 cm), 0.72 (d = 43 cm); scale factor

$$_cQ_{FU} = a_t \cdot {_s\sigma_y} \cdot g + 0.5ND\left(1 - \frac{N}{bD \cdot {_c\sigma_B}}\right) / a \qquad (2)$$

The results except 52 specimens which failed by bond-split are shown in Fig. 9. As observed, when the ratio $_cQ_{ARA}/_cQ_{BU}$ is larger than unit, the classified ductility is A or B except several cases, and when the ratio is less than unit, the ductility is almost C or D. However, as Qmax/bd becomes larger, the ductility tended to become poor.

2) Diagonal Tension Failure

At first, initial diagonal tension cracking load Q_{DTC}, was investigated. The theoretical value $_cQ_{DTC}$ was obtained to put the tensile principal stress, which was calculated on the center of the section neglecting the appearance of



Ductility	Mode	Series-Type	Mark	Pt (%)	M/QD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
A	F·S·Bu	LM2 - 4A	□—□	0.34	2	26.25	0.18	15.3	0.48	4.8
	F·B	FC - 7B	■—■	0.95	2	52.5	0.76	35.9	0.65	6.5
	F·C·Bu	FC - 2B		0.34	2	52.5	0.21	26.6	0.60	6.0
B	F·S	LM2 - 6B	○—○	0.61	2	26.25	0.27	20.0	0.80	8.0
	F·B	FC - 5B	●—●	0.61	1	26.25	0.58	49.5	0.50	10.0
	F·C·Bu	SE - 2A	Φ—Φ	0.34	2	52.5	0.85	23.0	1.08	5.4
C	F·S·Bu	LM1 - 2B	△—△	0.34	2	52.5	0.36	20.0	0.86	8.6
	F·B	LM2 - 8B	▲—▲	0.95	2	26.25	0.61	26.8	0.80	8.0
	F·C·Bu	Pilot-CBBA-2		0.41	3	30.0	0.19	11.9	1.00	6.67
D	S·S	LM1 - 1B	×—×	0.34	1	52.5	1.53	37.2	0.92	18.4
	B·B	Pilot-SAAB-2	*—*	1.24	1.5	30.0	1.42	34.0	0.75	10.0

FIG. 8 DETERIORATION OF SHEAR CAPACITY BY CYCLIC LOADING IN TYPICAL SPECIMENS

other cracks and the influence of axial reinforcements, equals to the tensile strength of concrete ${}_{c}\sigma_{t}$ which was assumed equal to $1.8\sqrt{{}_{C}\sigma_{B}}$. Here, ${}_{C}\sigma_{B}$ shows the compressive strength of concrete. The result is as follow.

$$ {}_{C}Q_{DTC} = \frac{{}_{c}\sigma_{t}}{1.5}\, bD\sqrt{1 + \frac{\sigma_{o}}{{}_{c}\sigma_{t}}} \tag{3}$$

The ratios of test results ${}_{T}Q_{DTC}$ to calculated values concerning 23 specimens in which diagonal tension cracks appeared, were deviated from 0.66 to 1.13 and the average was 0.87.

Next, the critical web reinforcement p_{WDTC} was calculated assuming that web reinforcements carry the diagonal tension lost by the appearance of this crack. The result is as follow.

$$ p_{WDTC} = \frac{{}_{c}\sigma_{t}}{\cos\theta \cdot {}_{s}\sigma_{wy}} \tag{4}$$

here $$\theta = \frac{1}{2}\tan^{-1}\left\{2\sqrt{\frac{{}_{c}\sigma_{t}}{\sigma_{o}} + \left(\frac{{}_{c}\sigma_{t}}{\sigma_{o}}\right)^{2}}\right\} \tag{4.1}$$

Taking the values of p_w/p_{WDTC} and ${}_{T}Q_{max}/{}_{C}Q_{DTC}$ as the ordinate and the abscisa respectively, test results except bond-split failured specimens were plotted in Fig. 10 with their failure mode. Here, ${}_{T}Q_{mas}$ and p_w mean the tested maximum shear strength and web reinforcement ratio, respectively.

It is observed in this figure that the specimens, in which ${}_{T}Q_{max}$ are more than about 0.8 ${}_{C}Q_{DTC}$ and p_w are less than p_{wDTC}, show the diagonal tension failure mode.

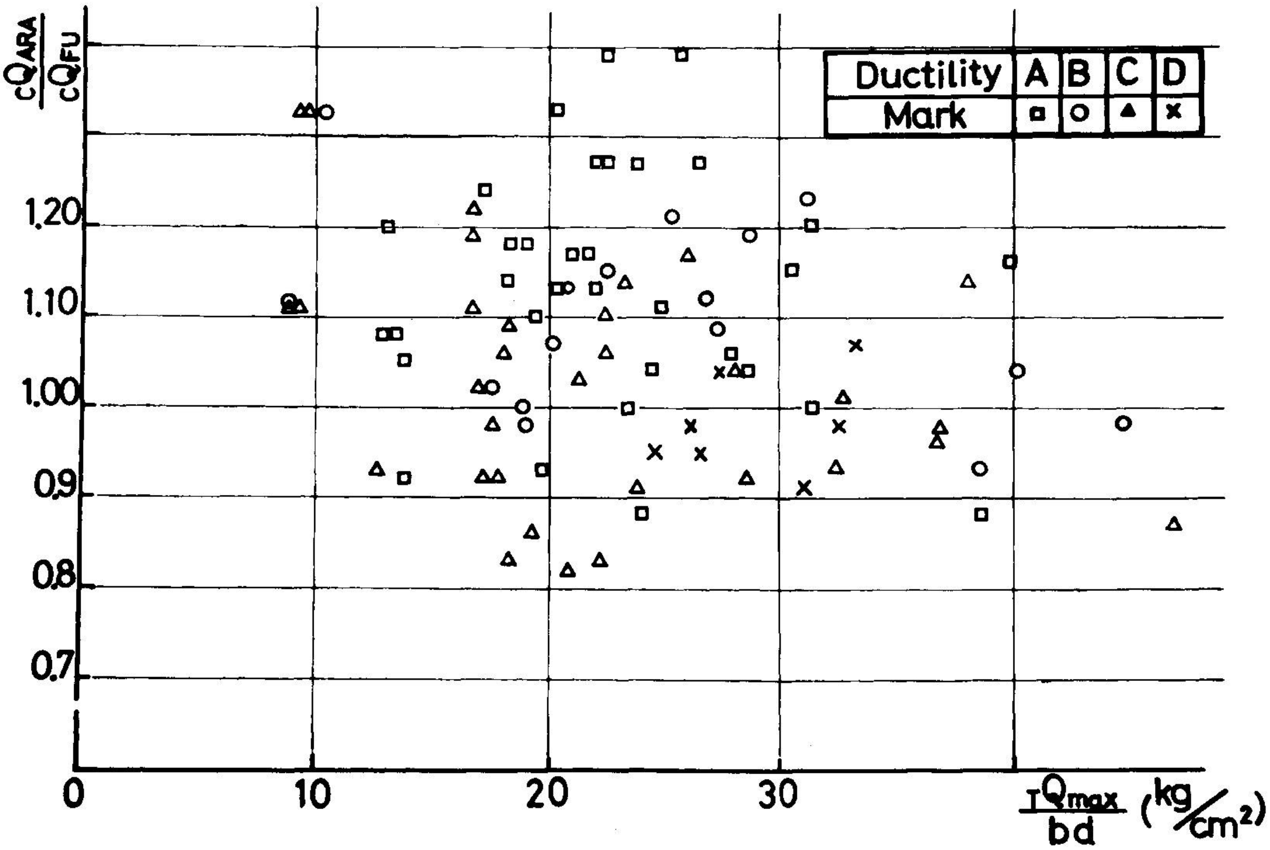


FIG. 9

3) Shear Tension Failure

At first, initial shear tension cracking load Q_{STC} was investigated. The theoretical value $_cQ_{STC}$ was calculated by the following formula which was derived semi-theoretically with some assumptions.

$$_cQ_{STC} = 0.589\, K_0 \cdot {_c\sigma_t} \cdot bD \tag{5.1}$$

When $a_1 = M/Qd < 0.75 + 0.283\, K_0$

$$_cQ_{STC} = \left\{ 0.78 - 1.04a_1 + \sqrt{(0.78 - 1.04a_1)^2 + 0.694K_0^2} \right\} {_c\sigma_t} \cdot bD \tag{5.2}$$

When $a_1 \geqq 0.75 + 0.283\, K_0$

here $K_0 = \sqrt{\dfrac{_c\sigma_t + \sigma_0}{_c\sigma_t}}$

The ratios of $_TQ_{STC}$ to $_cQ_{STC}$, concerning 119 specimens in which shear tension cracks appeared, were deviated from 0.62 to 1.59 and the average was 0.91.

Next, the following equation of critical web reinforcement ratio p_{WST} which will be required for the specimens to reach flexure strength after shear tension cracking was derived with some assumptions.

$$p_{WST} = \frac{1.6\, {_jx_1} \cdot p_t \cdot {_s\sigma_y}}{a_1^{\,1.25} \cdot {_s\sigma_{wy}}} \tag{6}$$

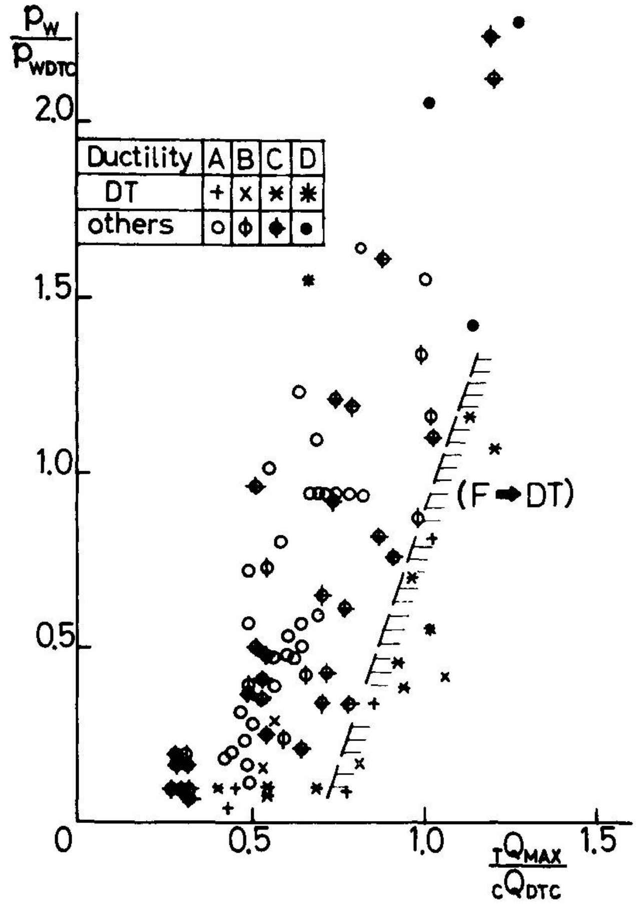


FIG. 10

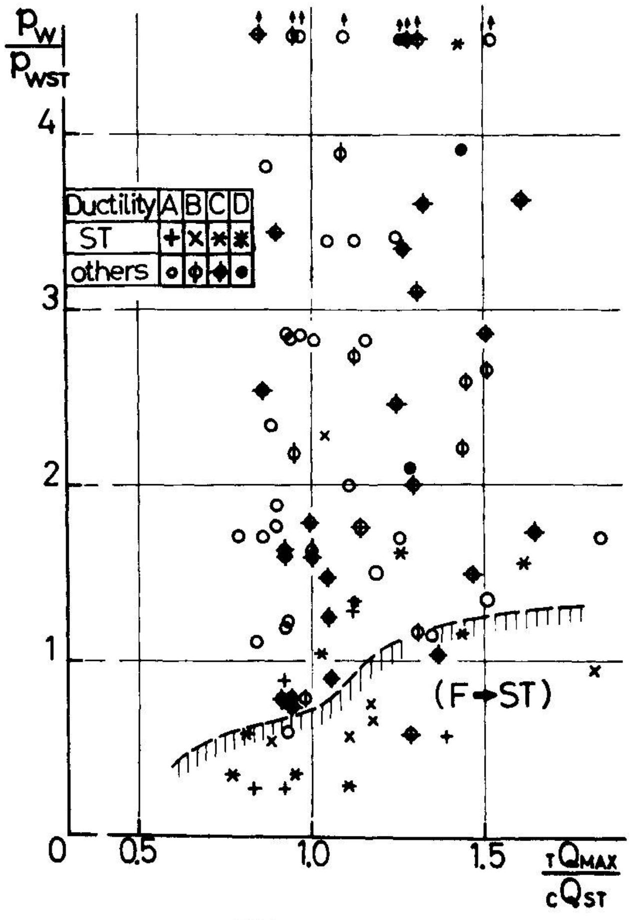


FIG. 11

here $$j_{x1} = \frac{d - x_n}{D} = d_1 - \frac{\sigma_o + p_t \cdot {}_s\sigma_y \cdot a_1^{-0.75}}{0.85{}_c\sigma_B} \tag{6.1}$$

Taking the values of p_w/p_{wST} and ${}_TQmax/{}_cQ_{STC}$ as the coordinates, test results except bond-split failured specimens were plotted in Fig. 11 with their failure mode. As shown, there are many specimens whose values of ${}_TQmax$ are more than ${}_cQ_{STC}$, however the specimens with ${}_TQmax$ more than about 0.8 ${}_cQ_{STC}$ but p_w less than p_{wST} show shear-tension failure mode.

4) Shear Compression Failure

As shear compression failure mode is caused essentially by collapse of compression concrete, it is easily presumed that there will be some conditions where adequate ductility cannot be expected even with much web reinforcements. In this report, we could not succeed to show the conditions definitely. However, we could find out some tendencies as follows.

First, it is necessary to get good ductility that axial stress is not so high and influence of shear stress is not so large. Accordingly, the distance from neutral axis to compression fiber Xn was roughly calculated neglecting compression reinforcement as shown in the following and the ratio of tensile principal stress to concrete tensile strength $\sigma_1/{}_c\sigma_t$ was calculated assuming that all of shear force is carried only by compression concrete at the critical section.

$$Xn1 = \frac{Xn}{D} = \frac{N + a_t \cdot {}_s\sigma_t}{0.85 \cdot {}_c\sigma_B \cdot bD} \tag{7.1}$$

$$\sigma_1 = -0.425\ {}_c\sigma_B + \sqrt{(0.425\ {}_c\sigma_B)^2 + \tau^2} \tag{7.2}$$

here $$\tau = \frac{{}_TQmax}{bXn} \tag{7.3}$$

Further, the critical web reinforcement ratio p_{wz} was calculated by the so-called truss-analogy which is used as one of the shear strength theories for beam.

$$p_{wz} = \frac{{}_TQmax}{b \cdot j \cdot {}_s\sigma_{wy}} \tag{7.4}$$

here $j = 0.875\ d$

Taking the values of $\sigma_1/{}_c\sigma_t$ and Xn1 as the coordinate, test results except 52 bond-split failured specimens were plotted in Fig. 12. As observed, many of the specimens satisfying the following limitation show good ductility regardless that p_w is more than p_{wz} or not.

$$\frac{\sigma_1}{{}_c\sigma_t} + 6\frac{Xn}{D} \leq 3 \tag{8}$$

Basing on this result, the relations of the calculated values corresponding to the left side of eq. (8) and p_w/p_{wz} were plotted in Fig. 13. The zones which show the shear-tension failure mode, the shear-compression failure mode

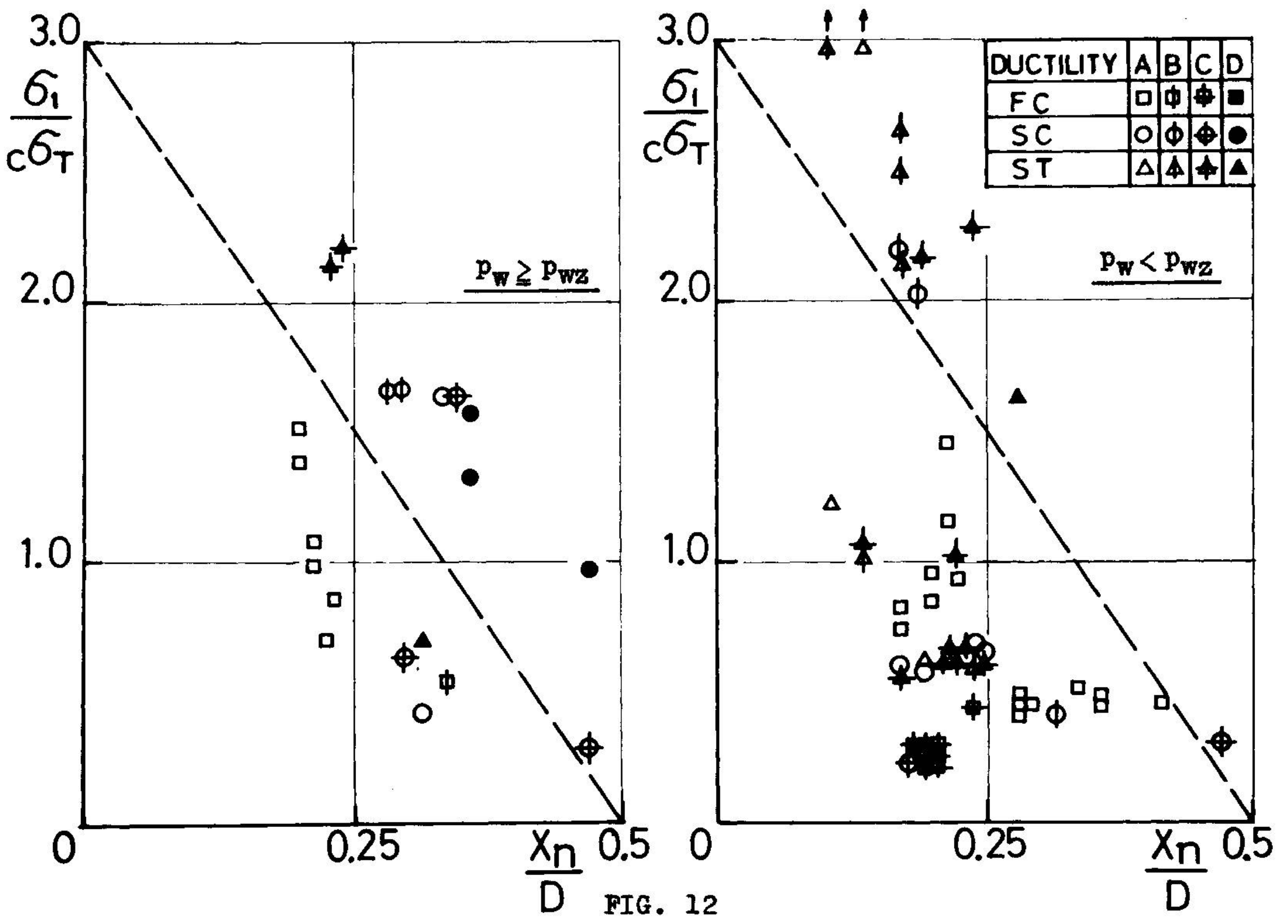

3.0
$\frac{\sigma_1}{{}_c\sigma_T}$
2.0
1.0
0
0.25
0.5
$\frac{X_n}{D}$
$p_w \geq p_{wz}$
$p_w < p_{wz}$
DUCTILITY A B C D
FC
SC
ST

FIG. 12

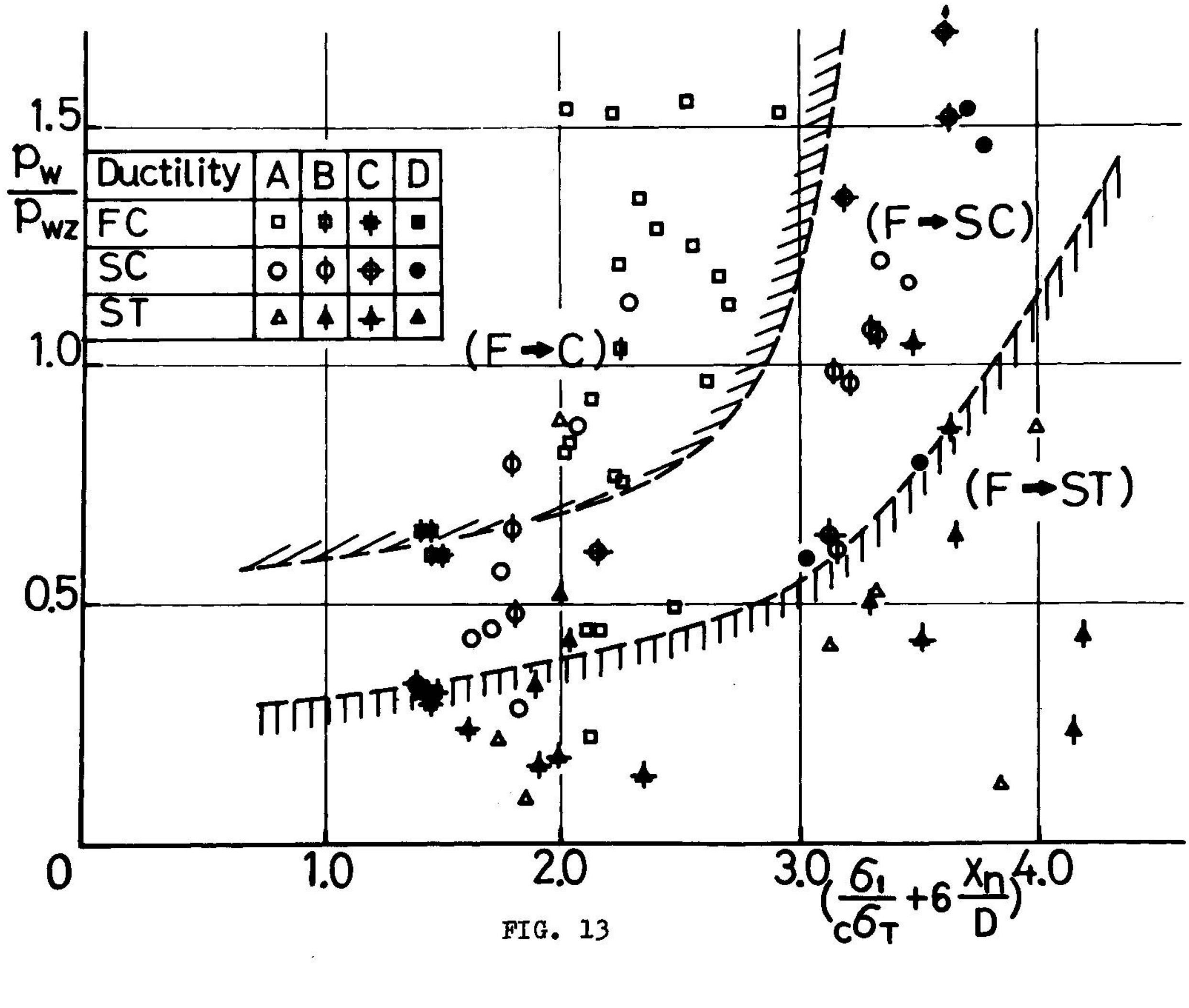

1.5
$\frac{p_w}{p_{wz}}$
1.0
0.5
0
1.0
2.0
3.0
4.0
$\left(\frac{\sigma_1}{{}_c\sigma_T}+6\frac{X_n}{D}\right)$
Ductility A B C D
FC
SC
ST
(F → C)
(F → SC)
(F → ST)

FIG. 13

and flexure-compression failure mode can be understood relatively apparently from this figure.

Accordingly, if such a limitation as shown in eq. (8) can be refined more theoretically and rigorously by accumulating test results, it will be possible to decide the limitation for effective web reinforcement ratio related to the combination of pt, σ_o and M/QD.

4.3 Bond-Split Failure

The distinction of this experimental result is that many specimens failed in the bond-split failure mode. 52 specimens showed this mode and this mode was observed in the specimens with high tensile reinforcement ratio p_t. Especially, 46 specimens showed this failure mode, out of 47 specimens with p_t more than 0.95%.

In the specimens failed in this mode, a small inclined crackings appeared initially at the position of tensile reinforcements apart by the effective depth d from the ends of the column. As the number of cyclic loadings increased or as horizontal deflection enlarged, many similar cracks progressed along the tensile reinforcements toward the center of the specimen. And, gradually, concrete cover exfoliated and shear capacity of the speciman deteriorated. Thus, the bond failures in this test are different from usual bond slip in massive concrete and this initial small inclined crack-called bond-split crack here after--is considered as a trigger for this failure mode.

So, after calculating the principal tensile stress caused by bending moment, shear force and bond stress at the point on tensile reinforcement apart by d from the end of the column, the shear force corresponding to this initial crack ${}_cQ_{BO}$ was obtained by putting the principal stress equals to ${}_c\sigma_t$.

The result is as follows.

$$ {}_cQ_{BO} = \frac{-{}_c\sigma_t \cdot B + \sqrt{{}_c\sigma_t^2 \cdot B^2 + 4C^2\,({}_c\sigma_t^2 + {}_c\sigma_t \cdot \sigma_0)}}{2C^2} \quad \text{(kg)} \qquad (9) $$

here
$$ B = \frac{(M/Q - d)\cdot(D/2 - d_t)}{I_e}, \quad {}_c\sigma_t = 1.8\sqrt{{}_c\sigma_B} $$

$$ C = \frac{1}{1.75 \cdot n' \cdot b' \cdot d} + \frac{6 \cdot I \cdot d \cdot d_t}{I_e \cdot b \cdot D^3} $$

I, Ie: Moment of inertia without and with the effect of axial reinforcements, respectively

b': smaller one of $(2\sqrt{2}\,dt - \phi_o)$ or $\dfrac{b - \Sigma\phi_o}{n'}$

n': number of tensile reinforcements

ϕ_o, $\Sigma\phi_o$: Diameter of a tensile reinforcement and the summation, respectively

Comparison of the calculated results ${}_cQ_{BO}$ with the test results ${}_TQ_{BO}$ are shown in Fig. 14. As shown, ${}_TQ_{BO}$ becomes generally more than ${}_cQ_{BO}$ as M/QD and pt decrease, and as σ_o increase. However, influence of web reinforcement ratio can not be observed.

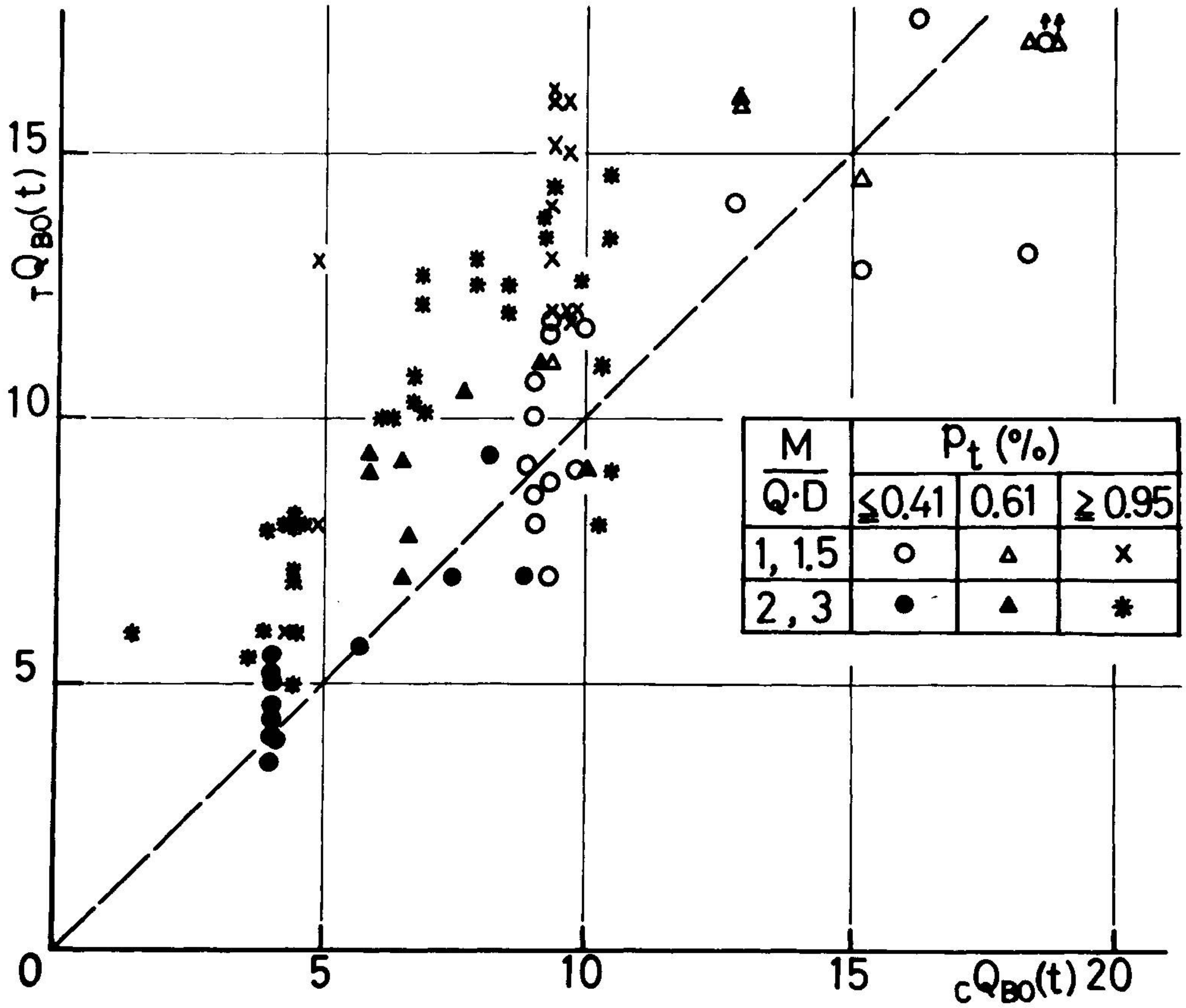


FIG. 14

Next taking the values of $_TQmax/_CQ_{BO}$ and $_TFmax/Fal$ as the coordinates, all of the test results were plotted in Fig. 15 with their failure mode and classified ductilities. Here, $_TQmax$ means the maximum shearing force at the test, and $_TFmax$ and Fal mean the maximum test results of mean bond stress and the allowable bond stress in the A.I.J. Standard(in Ref. 1) respectively. As shown, the value of $_TQmax/_CQ_{BO}$ has more influence to the bond-split failure than $_TFmax/Fal$ has.

Fig. 16 shows the relationship between the value of p_w/p_{wz} and the classified ductilities of the specimens failed in bond-split failure. As shown, these ductilities could not be improved so much even by increasing web reinforcement. However, in such specimens with $_C\sigma_B$ more than 270 kg/cm^2, when $_CQ_{FU}$ is less than 1.4 times $_CQ_{BO}$ and p_w nearly equal to p_{wz}, good ductilities could be obtained. Further, it is presumed from their cracking patterns that the confinement of the concrete within the core with spiral hoops etc. will be effective.

5. CONCLUSION

Flexure-shear tests under constant axial compression and multi-cyclic lateral forces were carried out on 125 short column specimens and following items on ductility were obtained.

1) As the factors which control ductility of columns, buckling of compression reinforcements, shear failure and bond failure are considered important.

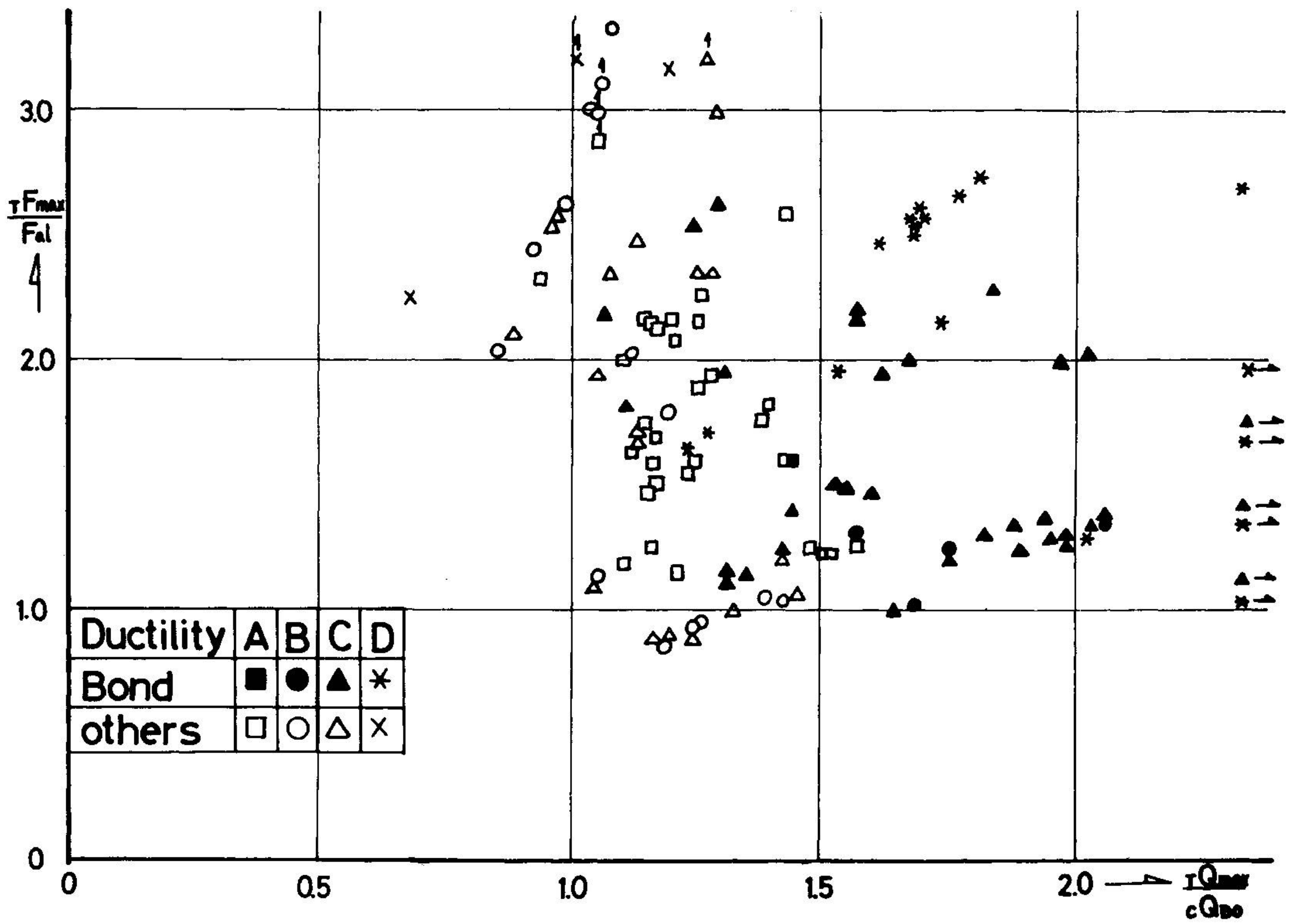


FIG. 15

2) To prevent compression reinforcements buckling until large deflection of column, it will be effective to keep spacing of web reinforcements less than 8 times diameter of the axial reinforcement.

3) When the flexure capacity ${}_CQ_{FU}$ shown by eq. (2) is more than about 0.8 times diagonal tension cracking load ${}_CQ_{DTC}$ shown by eq. (3), it is effective for preventing members from diagonal tension failure to put the web reinforcements much more than that calculated by eq. (4).

4) When the flexure capacity ${}_CQ_{FU}$ is more than about 0.8 times shear tension cracking load ${}_CQ_{STC}$ calculated by eq. (5), it is effective to put more web reinforcements than that shown in eq. (6).

5) There are certain conditions for the combination of pt, σ_o and M/QD where shear compression failure can not be avoid even with much web reinforcements. This conditions could not be made so apparent, but eq. (8) is considered to be one approach. When the combination of pt, σ_o and M/QD of specimens satisfy eq. (8) and when their p_w are more than p_{wz} shown in eq. (7.4), good ductilities could be recognized. However, there is a possibility that this value p_{wz} may be decreased still more.

6) Discussion by bond stress is not effective to prevent bond-split failure of the column. It seems effective to keep the flexure capacity ${}_CQ_{FU}$ within about 1.4 times of the initial bond-split cracking load ${}_CQ_{BO}$ shown by eq. (8). It is not effective to increase rectangular-typed hoops but it will be effective to put spiral hoop closely.

6. ADDITIONAL REMARKS

Analitical works about the following items are scheduled by the committee (Cf. § 7), and authors wish to present these results and the experimental results about the columns with side walls in the near future.

1) Influence of scale effect, loading excursion etc. to ductility of structural members.
2) Quantification of the rational web reinforcement ratio to make structural members ductile.
3) Persuit of structural device for preventing the split-bond failure.
4) Estimation of hysteresis damping of test results.
5) Method for estimating the seismic properties of the members from the results due to certain static cyclic loadings.

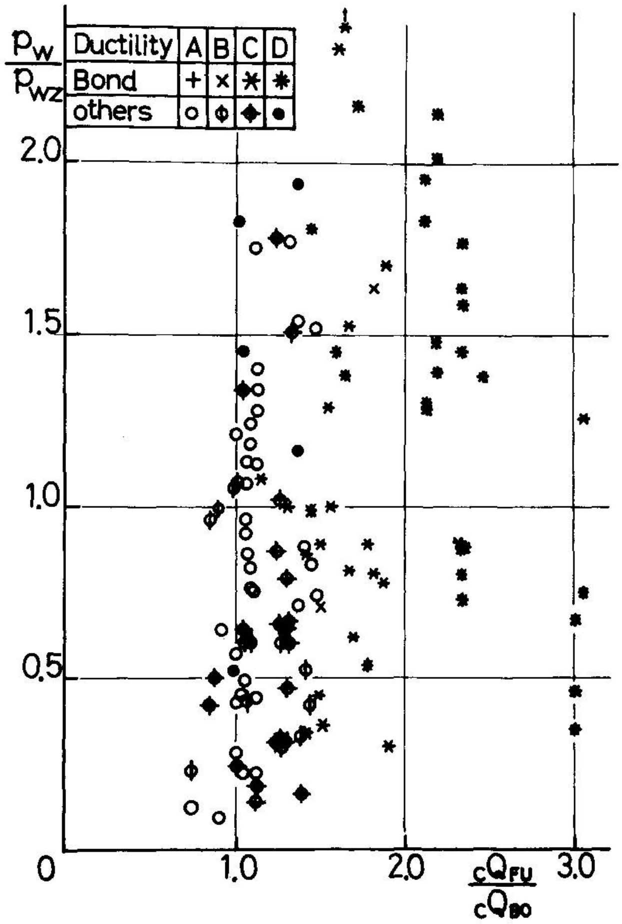


FIG. 16

7. ACKNOWLEDGEMENT

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8. NOTATION

b, D	;	width and overall depth of speciman respectively (cm)
d_t (or d_c)	;	distance from extreme tension (or compression) fiber to centroid of tension (or compression) reinforcement (cm)
d	;	$= D - d_t$, effective depth (cm)
g	;	$= d - d_t - d_c$ (cm) S; spacing of web reinforcement (cm)
p_t	;	$= a_t/bD$, tensile reinforcement ratio
p_w	;	$= a_w/bs$, web reinforcement ratio
$_s\sigma_y$	;	yield stress of tensile reinforcement (kg/cm^2)
$_s\sigma_{wy}$	;	yield stress of web reinforcement (kg/cm^2)
$_c\sigma_B$, $_c\sigma_t$	;	compressive and tensile strength of concrete, respectively (kg/cm^2)
Q	;	shear force (kg) P; = Q, Lateral cyclic load (kg)
σ_o	;	$= N/bD$ (kg/cm^2) N; constant axial force (kg)
M	;	bending moment at critical section (kg·cm)
a	;	$= M/Q$, half of the column length in this test (cm)
a_1	;	$a/D = M/QD$, shear span ratio
δ	;	displacement δ_y; displacement at yielding
μ	;	$= \delta/\delta_y$ ductility factor R; $= \delta/2a$, Rotation angle of member

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SUMMARY

Experimental research on ductility of reinforced concrete columns under constant axial compression and multi-cyclic lateral forces, and the results are reviewed. 165 short column specimens with many variable factors were tested. The important factors which influence ductility of column were buckling of axial compression bars at the ends of column, shear failure and bond split failure along axial tension bars. The behaviour and the analysis of the above failures and crackings, and the method to obtain good ductility are discussed about the results of 125 specimens.

RESUME

Ce rapport présente la recherche expérimentale sur la ductilité de colonnes en béton armé soumises à une compression axiale constante et à des forces latérales multicycliques ainsi que les résultats obtenus. 165 colonnes courtes ont été essayées sous de nombreuses conditions diverses. Les facteurs importants influençant la ductilité des colonnes étaient le flambage des barres d'acier comprimées aux extrémités des colonnes, la rupture au cisaillement et la rupture de l'ancrage des barre longitudinales. Le comportement et l'analyse des différentes formes de rupture indiquées ci-dessus ainsi que les mesures à prendre pour atteindre une bonne ductilité sont discutées en s'appuyant sur les résultats de 125 essais.

ZUSAMMENFASSUNG

Experimentelle Untersuchungen über das Verformungsvermögen von kurzen Stahlbetonstützen unter konstanter Normalkraft und wiederholter seitlicher Belastung werden mitgeteilt. 165 kurze Stützen wurden unter Variation vieler Einflussgrössen geprüft. Die Verformbarkeit wurde massgebend beeinflusst durch das Ausknicken der Längsbewehrung an den Stützenenden, durch Schubversagen und Verankerungsbruch der Längsbewehrung. Das Verhalten und die Berechnung dieser Versagens- und Bruchercheinungen sowie die Massnahmen zur Erzielung eines guten Verformungsvermögens werden anhand der Resultate von 125 Prüfkörpern diskutiert.