Shear resistance and explosive cleavage failure of reinforced concrete members subjected to axial load

Shear Resistance and Explosive Cleavage Failure of Reinforced Concrete Members Subjected to Axial Load

Résistance au cisaillement et rupture cassante explosive d'éléments en béton armé sous charge axiale

Schubwiderstand und explosiver Sprödbruch der Stahlbetonsäulen unter Achsiallast

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1. INTRODUCTION

As one of the most essential problem for the ductility requirement of the dynamic behaviour of reinforced concrete buildings, it is discussed here the shear resistance and explosive cleavage failure of reinforced concrete members subjected to axial load. Tests were carried out mainly to make clear the influences of axial load level ratios, shear span ratios and web reinforcement ratios upon their shear resistances and fracture modes. An analytical approach is presented here and compared with test results. By this research, the behaviours of brittle fracture and the causes of the lack of ductility of reinforced concrete members become clear and it will be possible to avoid the explosive cleavage failure, which had caused very often heavy damage of reinforced concrete buildings under strong earthquakes, and to establish the design methods how to give them sufficient ductility.

2. OBJECTIVES and SCOPE

The importance of ductility of members or connections for the dynamic resistance of reinforced concrete structures was emphasized by professors Newmark and Hall[1] in the preliminary publication. The lack of ductility of reinforced concrete members is caused mainly by the presence of high axial load or by the presence of high shearing force.

The former problem was discussed by several researchers[2] or by the author[3][4][5] at the 7th. congress of IABSE. Under higher axial load ($\frac{N}{\sigma_p bD} > 0.5$)[6], the deformation energy of reinforced concrete beam-column is dissipated mainly by concrete and not by longitudinal reinforcement. Therefore it shows the lack of ductility. On the contrary, under lower axial load ($\frac{N}{\sigma_p bD} < 0.5$), the deformation energy is dissipated mainly by longitudinal reinforcement and so it shows sufficient ductility. The only way to improve it, is to use sufficient web reinforcement in order to increase the ductility of concrete. There exists not so sufficient ductility by the presence of axial load but the fracture mode is always mild and not so brittle as shear fracture.

On the latter problem there are not yet sufficient knowledge[7][8]. Moreovere this problem is more essential for ductility requirements, because the fracture mode under higher shearing force shows a very brittle nature, and that it shows often even explosive fracture, especially in the presence of axial compression (see Photo. 1). There exists no ductility and yet such a fracture mode were found very often in heavily damaged reinforced concrete buildings under strong earthquake motion. They had caused often the collapse of whole structures at earthquake (see Photo. 2).

This paper deals on the fracture mode of reinforced concrete members subjected to high shearing force under axial compression and on the contribution of web reinforcements for this explosive cleavage failure. Tests were carried out to make clear the influences of axial load level ratios, shear span ratios and web reinforcement ratios of reinforced concrete members upon the shear resistance and shear fracture modes of them. An analytical treatment, which is based upon the biaxial fracture criteria of concrete, is presented here and compared with test results.

3. TESTS

3-1. Test Procedures and Measuring Devices

Tests were carried out by loading frames, which were specially installed in testing machine as shown in Fig. 1. The constant axial compression load N $(=X \cdot N_0)$ was introduced by testing machine through roller and maintained steady at constant value throughout the test. The transverse load P was applied by oil jack with electric load cell at its head, which was installed in loading frames.

There were two loading systems for shear tests as shown in Fig. 2, i.e. type C with single curvature in uniform shear span "a" at the both ends of the specimen and type Z with double curvature in uniform shear span "2a" at the central part of the specimen. Type C is ordinary shear mechanism for beam test and it has a merit of ordinary case as beam but has a demerit of the influences of additional bending through axial load by deflection for column test. Type Z is a special mechanism for shear test and it has a merit for the case of column test to avoid the influences of additional bending through deflection and to make possible the tests under higher shear span ratios. This loading system simulate often the loading condition of columns, beams or beam to column connections under earthquake motion.

Longitudinal and transverse displacements between main points or diagonal displacements between diagonal points in test span were measured by 1/100 mm dial gauges, which were set in measuring frame, that was fixed at one end on one loading line. Wire strain gauges were pasted upon surfaces in test span or several other deformation measuring techniques like checkerboard printing on the testing surfaces were applied them too.

Fig. 1 Testing Method for Z Type

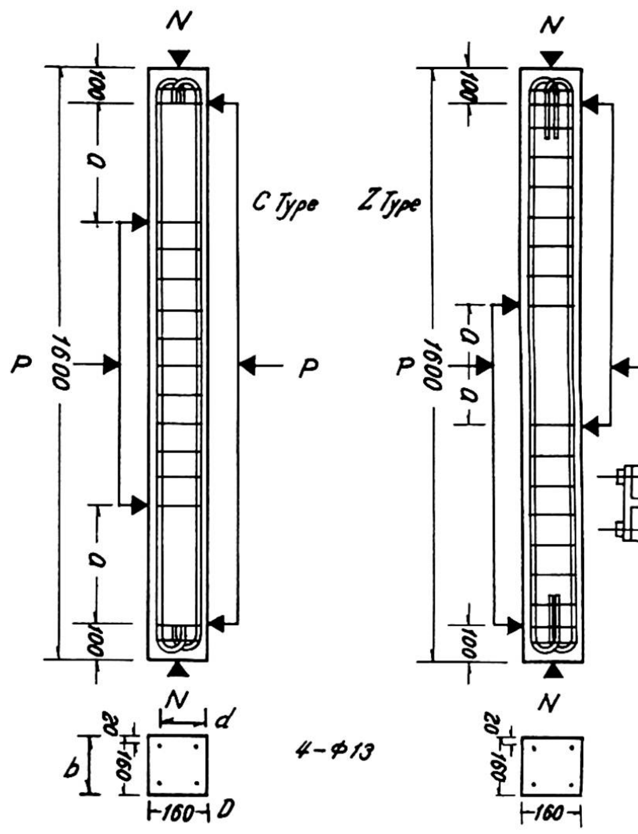
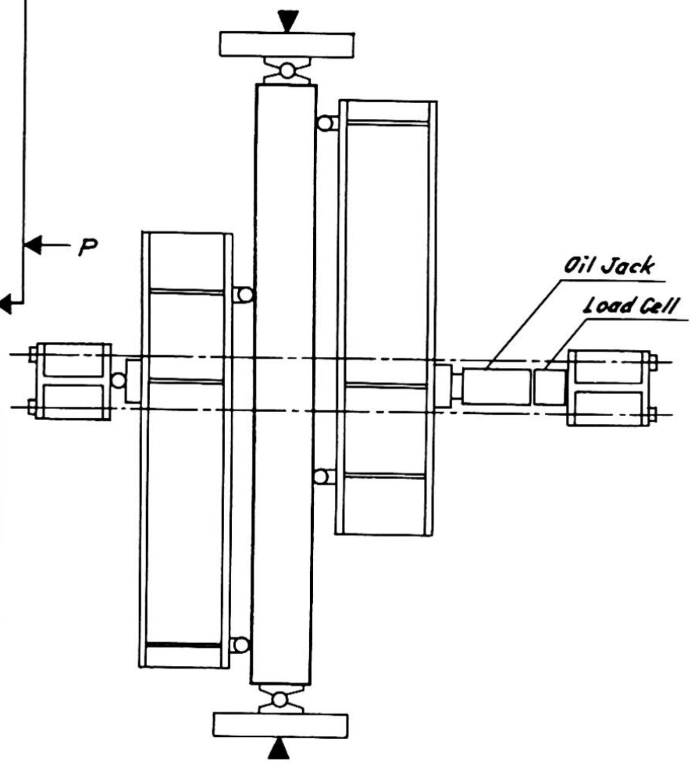


Fig. 2 Loading System & Specimens



3-2. Test Specimens and Test Series

Specimens had a length of 160 cm with a square cross section of width and depth of 16 cm x 16 cm and reinforced with 4 ordinary round steel bars of 13 mm diameter for longitudinal reinforcement, i.e.longitudinal gross reinforcement ratio p_s=1,04%. Test spans were reinforced with or without web reinforcement according to test series. Other spans were reinforced with web reinforcement of square type hoop of ordinary round steel bars of 6 mm diameter with 8 cm pitchs (see Fig. 2). Both ends of web reinforcement were welded together.

Concrete mix is 1 : 2,55 : 3,34 and W/C = 60%, the approximate concrete compressive strength $\sigma_P \cong$ 200 kg/cm², the approximate concrete tensile strength $\sigma_z \cong$ 20 kg/cm², the approximate yield point of longitudinal reinforcement of ϕ 13, $\sigma_y \cong$ 2800 kg/cm² or 3045 kg/cm² and of web reinforcement of ϕ 6, $\sigma_{yw} \cong$ 2144 kg/cm². The mechanical properties of materials are shown in Table 1.

Tests were carried out in three series:

Series I : For the research of the influence of shear span ratios upon shear behaviour, comparison between test mechanisms C type and Z type. The shear span ratio of Z type is defined here a/d. In this project the shear span ratios were varied 0,6 1,2 1,8 2,4 for Z type and 1,2 2,4 3,6 4,8 for C type. Test spans were not reinforced with web reinforcement.

Series II : For the research of the influences of the axial load level ratios upon shear behaviour. The axial load level ratio $X(=\frac{N}{N_0})$ is defined here as the ratio of acting constant axial compression N versus ultimate strength of centrally loaded column N_0. There were varied 0(beam), 1/6 and 1/3. Test spans were not reinforced with web reinforcement.

Series III : For the research of the contribution of web reinforcement. As web reinforcement it was applied here a square type hoop of round steel bar with 6 mm diameter. The hoop spacings were varied 0, 16 cm, 8 cm, 4 cm, i.e. web reinforcement ratios $\eta = 0$, 0,22, 0,44, 0,88%. It is defined here as the ratio of the area of longitudinal cross section of concrete versus the area of web reinforcement. contained in that cross section (see Fig. 4).

Series I and II were tested together. They are shown in Table 1.

3-3. Test Results

Test results are summarized in Table 1. Numerals following to B in the specimen notation of the table indicate the hoop spacing in cm and C or Z with numerals indicate the test mechanism and the ratio of the shear span versus the depth of specimen.

The fracture mode S in the table indicates cleavage shear explosion, B → S indicates initially bending and finally shear crack opening and B indicates bending crack deformation.

The deformation process and fracture modes under various shear span ratios and various axial load level ratios are shown in Fig. 3, as the relation between relative displacements of both ends of shear span "2a" for Z type and lateral load P.

Table 1 Test Series & Test Results

Specimen	a/d	Steel σ_y kg/cm²	Concrete Comp. σ_P	Concrete Tens. σ_z	Const. Axial Load	Max. Bending Moment	Max. Shear Force	Fracture Mode
RC:C1:B0:C1:1/3NoQ	1.2	2800	291	24.7	30t	1.95t·m	12.2t	S
RC:C1:B0:C2:1/3NoQ	2.4	2800	360	27.7	36	2.24	7.00	B
RC:C1:B0:C3:1/3NoQ	3.6	2800	360	27.7	36	2.30	4.79	B
RC:C1:B0:C4:1/3NoQ	4.8	2800	360	27.7	36	2.40	3.75	B
RC:C1:B0:C3:1/6NoQ	3.6	2800	360	27.7	18	1.96	4.10	B
RC:C1:B0:C3: 0NoQ	3.6	2800	360	27.7	0	1.00	2.09	B
RC:C1:B0:Z1:1/3NoQ	0.6	2800	291	24.7	30	1.05	12.7	S
RC:C1:B0:Z2:1/3NoQ	1.2	2800	291	24.7	30	1.60	9.40	S
RC:C1:B0:Z3:1/3NoQ	1.8	2800	291	24.7	30	2.29	9.50	S
RC:C1:B0:Z4:1/3NoQ	2.4	2820	218	19.7	24	1.97	6.17	B→S
RC:C1:B0:Z1:1/6NoQ	0.6	3045	213	19.7	12	0.73	9.10	S
RC:C1:B0:Z2:1/6NoQ	1.2	3045	213	19.7	12	1.12	7.00	S
RC:C1:B0:Z3:1/6NoQ	1.8	3045	213	19.7	12	1.70	7.10	S
RC:C1:B0:Z4:1/6NoQ	2.4	3045	213	19.7	12	1.92	6.00	B→S
RC:C1:B0:Z1: 0NoQ	0.6	3045	197	19.4	0	0.53	6.60	B→S
RC:C1:B0:Z2: 0NoQ	1.2	3045	197	19.4	0	1.04	6.50	B→S
RC:C1:B0:Z3: 0NoQ	1.8	3045	197	19.4	0	0.91	3.80	B→S
RC:C1:B0:Z4: 0NoQ	2.4	3045	197	19.4	0	1.25	3.90	B
RC:C1:B0:Z3: 0NoQ	1.8	2800	291	24.7	0	1.15	4.80	B→S
RC:C1:B4:Z2:1/3NoQ	1.2	2800	202	20.2	23	1.81	11.3	B→S
RC:C1:B8:Z2:1/3NoQ	1.2	2800	202	20.2	23	1.51	9.42	S
RC:C1:B16:Z2:1/3NoQ	1.2	2800	202	20.2	23	1.38	8.66	S
RC:C1:B0:Z2:1/3NoQ	1.2	2800	202	20.2	23	1.13	7.06	S

B: Bending Fracture
S: Cleavag Shear Fracture (Explosion)
B→S: Bending → Shear Crack Opening

Z.R. in the figures of deformation characteristics indicates the formation of tensile crack, S.R. the formation of shear crack opening, L.R. the formation of a diagonal tension crack between loading points and X-mark the explosive cleavage shear

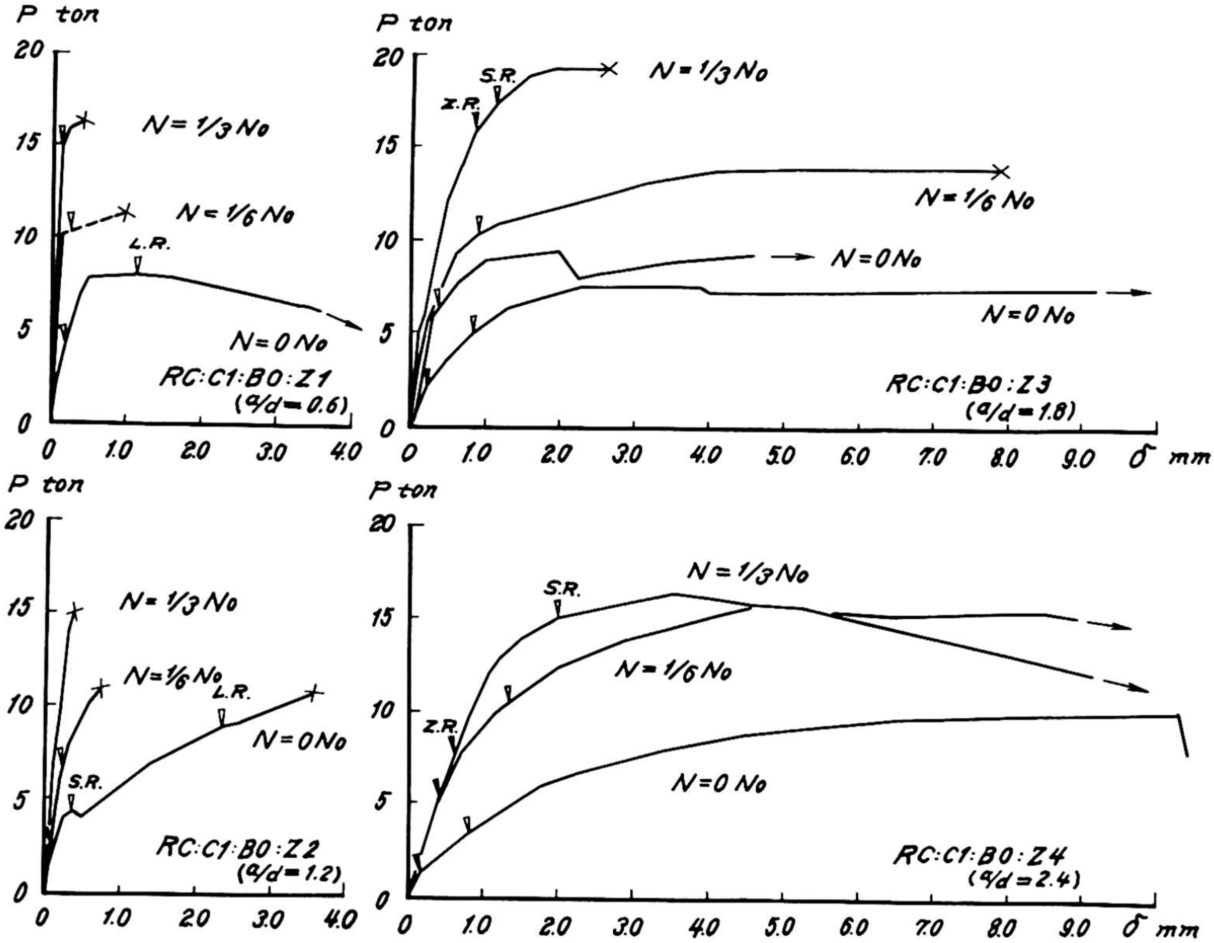
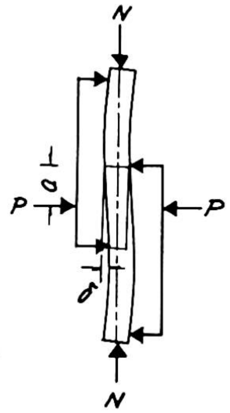


Fig.3 Deformation Characteristics



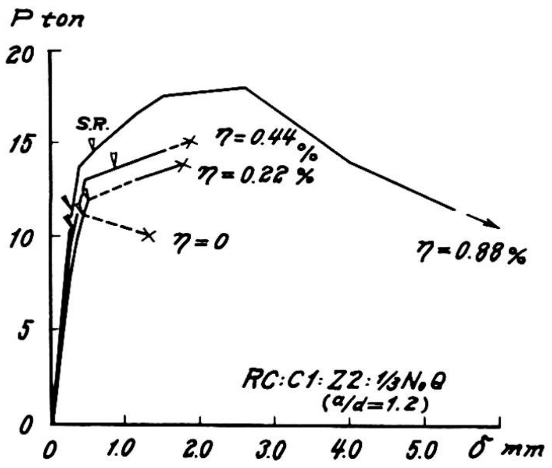
failure.

Under higher shear span ratios it appears at first bending crack Z.R. at the tension side of the loaded cross section and the deformation occurs mainly by the opening of this tensile crack, then this tensile crack shows inclination or it appears several shear cracks S.R. and finally the compression side of the loaded cross section is crushed down.

Under intermediate shear span ratios it appears initially tensile crack Z.R. by bending at the tensile side of the loaded cross section, then on the side surfaces of shear span it appears short diagonal cracks and its opening becomes larger.

Under lower shear span ratios it appears initially several short shear cracks on the side surfaces of shear span and gradually it increases their number accompanied by the increase of transverse load, then suddenly but in several seconds it occurs explosion by a large diagonal tension crack opening directly between loading points independently from formerly formed short shear cracks. This behaviour is intensiver, the higher the axial load ratios (see Photo.1).

The deformation process and fracture modes, for the case of a shear span ratio of 1,2 and an axial load level ratio of 1/3 with various web reinforcement ratios are shown in Figs. 4 and 12.



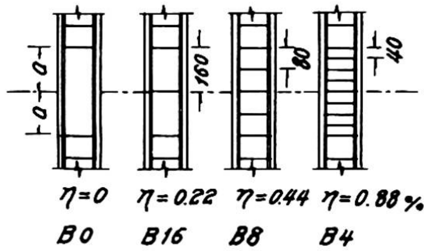


Fig.4 Contribution of Web Reinforcement for Shear Deformation & Fracture

4. ANALYTICAL APPROACH

4-1. General Description

For the analysis, the reinforced concrete member is devided into finite small rectangular elements with a central point (i,j) as their co-ordinates. The stresses and displacements of each element are represented by this cross point. Following to the increase of external load, the stresses and displacements are calculated and the fracture of each element is checked by critical fracture condition of concrete. If the stress condition of one element reaches the critical condition, the element is destroyed and it bears no more stresses and the stresses, which were born by the failed element, will be redistributed to another elements proportional to their stiffness. After the redistribution of stresses, it proceeds to next loading stage. So by repeating this procedure step by step, the elasto-plastic deformation behaviour of this member is able to followed. Through the decrease of the number of load bearing elements, finally it will bear no more increase of external load and it will reach the ultimate state. Fracture mode and ultimate strength will be so clarified.

4-2. Elements and their Fracture Condition

For the stresses and displacements of an element, it is assumed that:

(1) As the element, there are two kinds of elements,i.e. concrete element with reinforcing steel and concrete element without reinforcing steel. Reinforcing steel is estimated by equivalent cross section for normal stresses.

(2) The external forces are distributed to each elements proportional to their stiffness. They are represented with their central point. As the stresses of each element, it is considered σ_{ij} and τ_{ij} but it is neglected here the normal stresses perpendicular to the member axis.

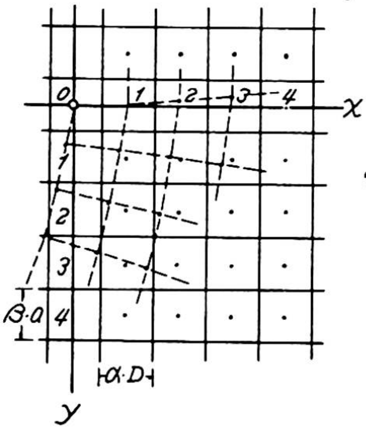
(3) The shearing stress τ_{ij} is decided by normal stress and for the element with reinforcing steel it is considered the

increment of shearing stress by the difference of stresses in reinforcing steel.
The displacements of each cross point are: (see Fig. 5)

$$V_{ij+1} = -\beta\cdot a\left[\frac{\sigma_{ij}+\sigma_{ij+1}}{2}\cdot\frac{1}{E_c}\right]+V_{ij} \quad (1),\qquad \varphi_{ij} = \frac{V_{i+1j}-V_{ij}}{\alpha\cdot D} \quad (4),$$

$$V_{i+1j+1} = -\beta\cdot a\left[\frac{\sigma_{i+1j}+\sigma_{i+1j+1}}{2}\cdot\frac{1}{E_c}\right]+V_{i+1j} \quad (2),\qquad \theta_{ij} = -(\gamma_{ij}+\varphi_{ij}) = \theta_{i+1j} \quad (5),$$

$$\sigma_{ij} = \frac{\tau_{ij}+\tau_{i+1j}+\tau_{ij+1}+\tau_{i+1j+1}}{4}\cdot\frac{1}{G_c} \quad (3),\qquad u_{ij+1} = \beta\cdot a\cdot\theta_{ij}+u_{ij} \quad (6).$$



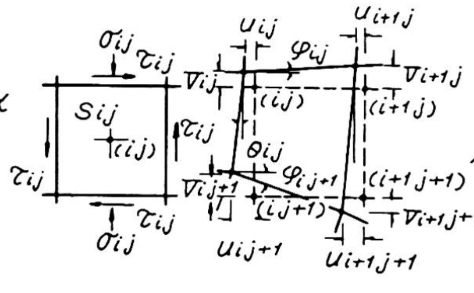


Fig.5 Finite Element

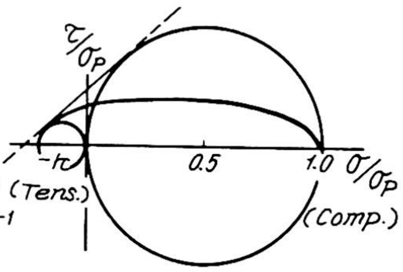


Fig.6 Fracture Condition of Concrete

The critical fracture condition of each element is assumed to occur under the following combined biaxial fracture condition of concrete by Mohr (see Fig. 6):

$$\frac{(1+K)^2}{K}\cdot\tau^2 + \left[\sigma - \frac{(1-K)}{2}\sigma_P\right]^2 = \frac{(1+K)^2}{4}\cdot\sigma_P^2 \quad (7),$$

here, $K = \sigma_Z/\sigma_P$, σ_Z and σ_P tensile and compressive strength of concrete respectively.

The normal stress σ and the shearing stress τ for the estimation of the critical condition of the element, which are influenced by redistribution of stresses from neighbouring elements, are calculated as follows:

$$\sigma = \frac{\sigma_{ij-1}+2\sigma_{ij}+\sigma_{ij+1}}{4} \quad (8),$$

$$\tau = \frac{\tau_{i-1j}+2\tau_{ij}+\tau_{i+1j}}{4} \quad (9).$$

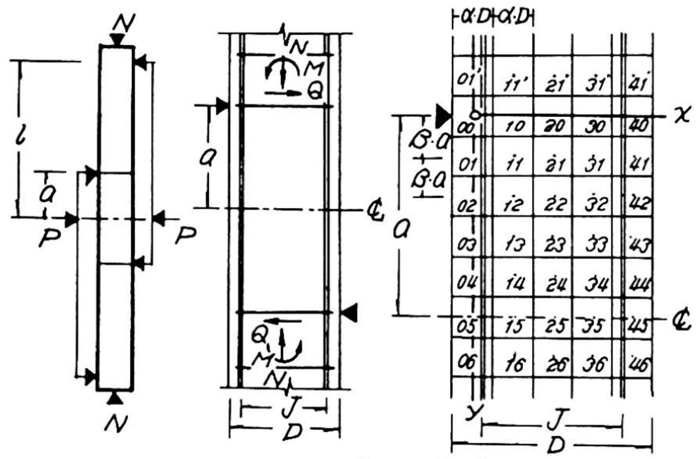


Fig.7 Reinforced Concrete Member

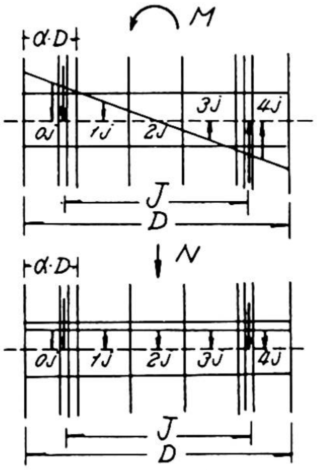


Fig.8. Nomal Stresses

4-3. Application for Reinforced Concrete Columns loaded by Bending Moment, Shearing Force and Axial Compression

Normal stress of each element is calculated as the sum of normal stresses $_M\sigma_{ij}$ through bending moment and $_N\sigma_{ij}$ through axial load. Within each element the stress is assumed to distribute uniformely. The steel stress is calculated by the concrete normal stress σ_{ij}, in which element the reinforcing steel is contained (S_{0j} , S_{4j} in Fig. 8), and if the reinforcing steel is placed excentrically to the center of the cross point, it is modified by the equivalent ratio of moduli of elasticity n' as follows:

$$\sigma_{sij} = n' \cdot \sigma_{ij} \quad , \quad \left(n = E_s/E_c \,, \quad n' = \frac{d/2}{2 \cdot \alpha \cdot D} \cdot n \right) \tag{10}$$

So the normal stress through bending moment is calculated by the equilibrium condition as follows:

$$_M\sigma_{0j} \cdot (\alpha \cdot D \cdot b + n' \cdot A_s) + {_M\sigma_{1j}} \cdot \alpha \cdot D \cdot b + {_M\sigma_{2j}} \cdot \alpha \cdot D \cdot b + {_M\sigma_{3j}} \cdot \alpha \cdot D \cdot b + {_M\sigma_{4j}} (\alpha \cdot D \cdot b + n' A_s) = 0 \tag{11}$$

$$_M\sigma_{0j} (2\alpha^2 D^2 b + n' \cdot A_s \cdot d/2) + {_M\sigma_{1j}} \cdot \alpha^2 D^2 b - {_M\sigma_{3j}} \cdot \alpha^2 D^2 b - {_M\sigma_{4j}} \cdot (2\alpha^2 D^2 b + n' \cdot A_s \cdot d/2) = M_j \tag{12}$$

Normal stress $_N\sigma_{ij}$ by axial compression is calculated by the ratio of moduli of elasticity n as follows:

$$_N\sigma_{ij} \cdot \alpha \cdot D \cdot b = \frac{\alpha \cdot D \cdot b \cdot E_c}{(5 \cdot \alpha \cdot D \cdot b + 2n \cdot A_s) E_c} \cdot N \tag{13}$$

Shearing stress τ_{ij} is calculated for the element without longitudinal reinforcement:

$$\tau_{ij} = \frac{1}{2} \left[\frac{({_M\sigma_{ij-1}} + {_N\sigma_{ij-1}}) - ({_M\sigma_{ij+1}} + {_N\sigma_{ij+1}})}{2} + \frac{({_M\sigma_{i-1,j-1}} + {_N\sigma_{i-1,j-1}}) - ({_M\sigma_{i-1,j+1}} + {_N\sigma_{i-1,j+1}})}{2} \right] \cdot \frac{\alpha \cdot b \cdot D}{\beta \cdot a \cdot b} + \tau_{i-1j} \tag{14}$$

and for the element with longitudinal reinforcement:

$$\tau_{ij} = \frac{1}{2} \left[\frac{{_M\sigma_{ij-1}} - {_M\sigma_{ij+1}}}{2} + \frac{{_M\sigma_{i-1j-1}} - {_M\sigma_{i-1j+1}}}{2} \right] \cdot \frac{\alpha \cdot b \cdot D + n' \cdot A_s}{\beta \cdot a \cdot b} + \frac{1}{2} \left[\frac{{_N\sigma_{ij-1}} - {_N\sigma_{ij+1}}}{2} + \frac{{_N\sigma_{i-1j-1}} - {_N\sigma_{i-1j+1}}}{2} \right] \frac{\alpha \cdot b \cdot D + n A_s}{\beta \cdot a \cdot b} + \tau_{i-1j} \tag{15}$$

Through the fracture of each element, the stresses are redistributed. If the element S_{4j} in Fig. 9 is destroyed, the element S_{4j} bears now only by longitudinal steel and therefore the stiffness of S_{4j} for normal stress decreases. Then the neutral axis removes towards the compression side and through the equilibrium condition the new normal stress σ_{ij} is decided. Shearing stresses in S_{ij-1} and S_{ij+1} are redistributed by (14) and (15).

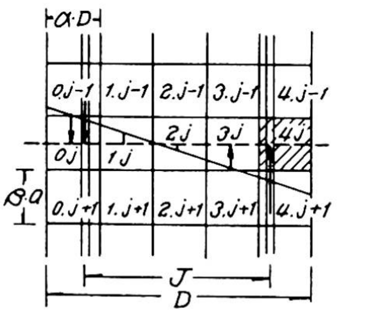
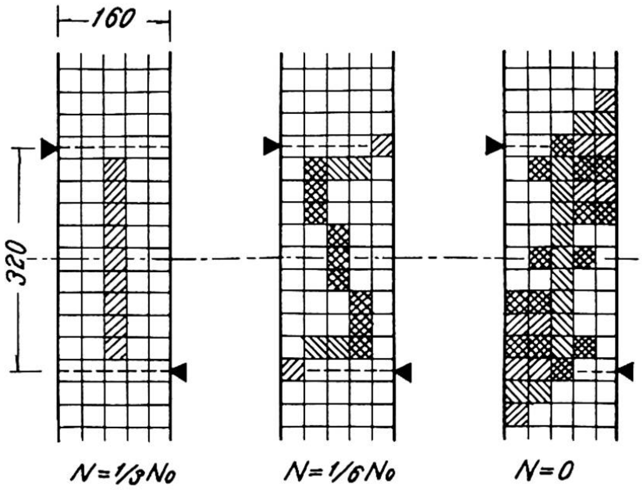


Fig.9 Destroyed Element

Fig.11 Fracture Process of Elements (Computed)



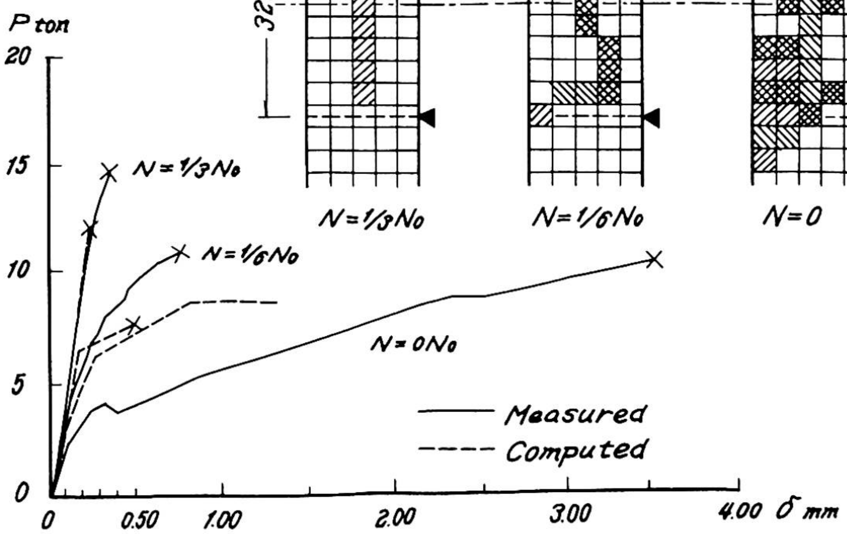


Fig. 10 Fracture Process of RC:C1:B0:Z2 (Measured & Computed).

Fig. 10 shows the test results with solid lines and corresponding calculated values of foregoing deformation analysis for the case of a shear span ratio $a/d = 1,2$ and axial load level ratios of 0, 1/6 and 1/3 with dotted lines.

Fig. 11 shows the fracture process of foregoing analysis. For the case of $N = \frac{1}{3} N_0$, the elements in the central part of the member is destroyed through compressive shear and the redistributed stresses destroyed another elements one after another and it increases no more load bearing capacity. For the case of $N = \frac{1}{6} N_0$, at first the extreme tension side element reaches the critical condition at the maximum moment section and then, following to the increase of external load, the inside elements are destroyed gradually and final state is decided. For the case of $N = 0 N_0$, at first the tensile crack occures and it penetrates into inside and, even when the central elements are destroyed by tensile shear, it shows the more increase of load carrying capacity.

5. DISCUSSIONS

5-1. Interaction between Shear Span Ratios and Axial Load Ratios

Table 2 shows the interaction between shear span ratios (a/d) and axial load level ratios $(X = \frac{N}{N_0})$ upon the fracture behaviours of reinforced concrete members very clearly. The lower the shear span ratios and the higher the axial load ratios becomes the explosiver the fracture mode. On the contrary, the higher the shear span ratios and the lower the axial load ratios, the milder the fracture mode. The ductility of members is influenced and decided by this fracture mode. Ductility requirement of reinforced concrete members is satisfied under the condition of lower axial load ratios and higher shear span ratios.

It is a remarkable fact that there exists a very clear difference of fracture modes at a value of (a/d) between 1,8 and 2,4 for every axial load ratios. (See Photo. 3)

Bending resistance decreases under lower shear span ratios for columns as it was pointed out for beams by prof. Kani [9].

Table 2. Interaction a/d - X

a/d	Constant Axial Load Ratio		
	$N=0N_0$	$N=\frac{1}{6}N_0$	$N=\frac{1}{3}N_0$
0.6	B→S	S	S
1.2	B→S	S	S
1.8	B→S	S	S
2.4	B	B→S	B→S

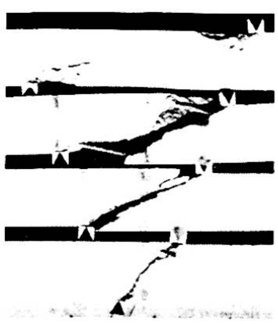


Photo. 3 Influences of Shear Span Ratios (a/d) upon the Fracture Modes ($\eta = 0$, $N = \frac{1}{3} N_0$)

5-2. Effect of Web Reinforcement

Ductility requirement under higher axial load ratios with lower shear span ratios is improved by web reinforcement. The effects are shown in Fig. 4 under an axial load level ratio of 1/3 and a shear span ratio of 1,2. It is a very severe condition, but it happens often in actual case, for the case of without web reinforcement $\eta = 0$, it shows a typical explosive shear fracture. For the case with lower web reinforcement ratios $\eta = 0,22$ and 0,44%, they showed a little ductility even with a little increase of resistance, but finally explosive cleavage failure. However for the case with fairly higher web reinforcement ratio $\eta = 0,88\%$, it shows no more shear fracture but sufficient ductility.

5-3. Damage of Reinforced Concrete Buildings under Strong Earthquakes

Photo. 2 shows one of the typical cleavage shear fracture of a reinforced concrete column of the gymnasium of Niigata high school, at the strong earthquake on the 16th. June 1964, Niigata/Japan. Photos 1 and 2 , i.e. test specimen and real case show a very good similarity of their fracture mode. Test specimen shows the cause of damage of reinforced concrete columns under earthquake very clearly. Such a explosive cleavage shear failure of column caused very often heavy damage of whole buildings under strong earthquakes. These photographs of damage under earthquake show very clearly the importance of the problem of shear resistance under axial compression for the dynamic behaviour of reinforced concrete buildings.

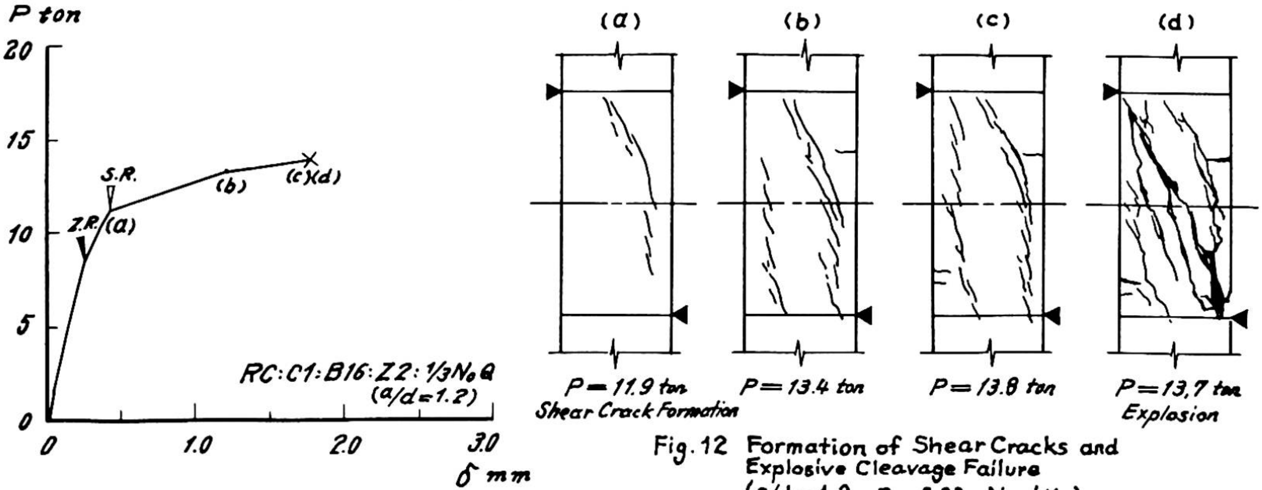


Fig. 12 Formation of Shear Cracks and Explosive Cleavage Failure ($a/d=1.2$, $\eta=0.22$, $N=\frac{1}{3}N_0$)

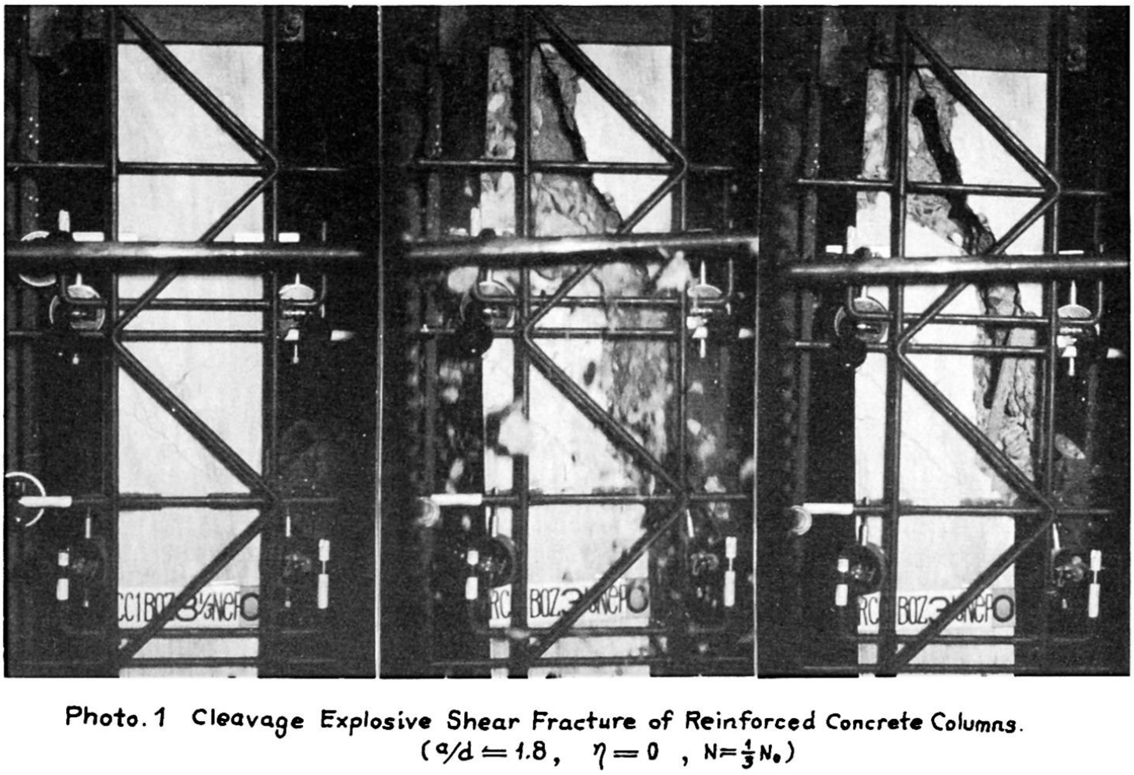


Photo. 1 Cleavage Explosive Shear Fracture of Reinforced Concrete Columns. ($a/d \fallingdotseq 1.8$, $\eta=0$, $N=\frac{1}{3}N_0$)

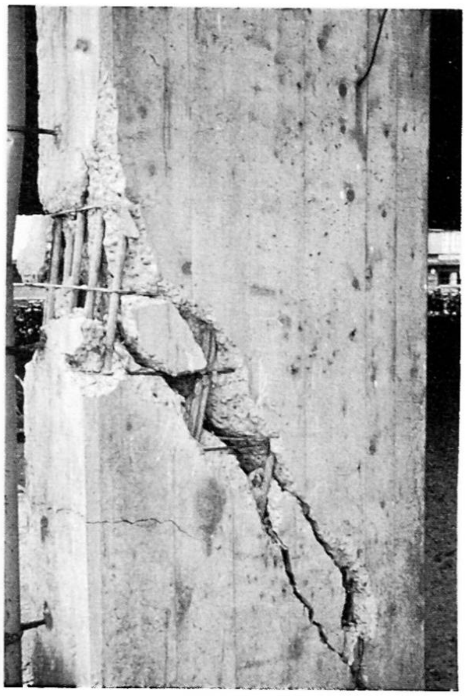


Photo. 2 Cleavage Shear Fracture of a Reinforced Concrete Column of Gymnasium in Niigata High School/Japan, by the Earthquake on the 16th. June, 1964.
(Photo.: Courtesy of ass.Prof. Mizuhata)

6. CONCLUDING REMARKS

The lack of ductility of reinforced concrete member is caused by high axial compression and high shear force. The simultaneous action of both forces had caused very often heavy damage of whole structures under strong earthquakes (see Photo. 2).

This paper intends to make clear the shear resistance and explosive cleavage failure of reinforced concrete members subjected to axial load as one of the most essential cause of the lack of ductility. Three series of tests were carried out to make clear the influences of axial load level ratios, shear span ratios and web reinforcement ratios upon deformation characteristics and fracture modes. Test results are shown in Figs. 3 and 4 and summerized in Tables 1 and 2. They show the fact that the higher the axial load level ratios, the lower the shear span ratios and the lower the web reinforcement ratios, the ductility of the member will be lost and it causes often explosive cleavage failure (see Photo. 1). Test specimens show a very good simulation with actual case under earthquake.

An analytical approach gives a fairly good agreement with the behaviours of test results (Figs. 10, 11).

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SUMMARY

As one of the most essential problem for the ductility requirement of the dynamic behaviour of reinforced concrete buildings, theoretical and experimental researches were carried out to make clear the influences of axial load ratios, shear span ratios and the contribution of web reinforcement ratios upon the shear resistance and fracture modes. It becomes clear one of the most important cause of the heavy damage of reinforced concrete buildings by the lack of ductility (Photos. 1 and 2).

RÉSUMÉ

Des recherches théoriques et expérimentales ont été faites concernant la ténacité et son influence dans le comportement dynamique de bâtiments en béton armé, pour déterminer l'influence de la charge axiale, de la répartition des forces de cisaillement, et la contribution du réseau d'armature sur la résistance de cisaillement et sur le comportement à la rupture. On voit que le manque de ténacité est une des causes les plus importantes des lourds domages dans les bâtiments en béton armé. (Fig. 1 et 2)

ZUSAMMENFASSUNG

Als eines der wichtigsten Probleme für die Zähigkeitsforderung des dynamischen Verhaltens von Stahlbetongebäuden wurden theoretische und experimentelle Untersuchungen angestellt, um den Einfluss der Achsialkraft, der Querkraftverteilung sowie des Bewehrungsnetzes auf den Schubwiderstand und das Bruchverhalten aufzuklären. Es wird klar, dass dies einer der hauptsächlichsten Gründe für den schweren Schaden bei Stahlbetongebäuden ist, wenn diese der Zähigkeit ermangeln. (Fig. 1 und 2)