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| Author(s) | Ohno, Kazuo; Shibata, Takuji; Hattori, Takashige |
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# AN EXPERIMENTAL STUDY ON THE FAILURE OF COLUMN

Kazuo OHNO*
Takuji SHIBATA**
Takashige HATTORI***

Relating to the destruction of reinforced concrete building in the Tokachioki earthquake, 1968, an experimental study was carried out on the models of columns. The specimens were prepared in a frame type, instead of a single column type which is usually used in experimental studies on the strength of columns.

## (1) Specimen

The specimen was prepared in a two-storied frame as will be seen in Fig. 1. The columns in this specimen had a cross-section of 16 cm x 10 cm and its length was 40 cm. These dimensions were one-fifth scale model of the actual column of the destroyed building. The relative stiffness ratio of the top-girder to the column was about 21. The ratio of axial reinforcement to the gross cross-sectional area of the column was 6.37 % and the ratio of hoop was 0.324 %. The corresponding ratios in the actual building were 2.47 % and 0.14 %.

The concrete consisted of A portland cement, river sand (smaller than 5 mm) and river gravel (smaller than 25 mm). The mixing ratio was 1:3.39:3.43 by weight with a water-cement ratio of 0.76. The test was carried out at the age of 13~15 days. The mean values of compressive strength at the age of test, the secant modulus at the one-third of the strength and the tensile strength by the cylinder-splitting method were respectively 108 kg/cm$^2$, $1.29\times10^5$ kg/cm$^2$ and 11.9 kg/cm$^2$.

The properties of the steel are shown in Table 1.

## (2) Loading

The constant axial forces were previously introduced into each column by means of coil-springs. The magnitude of axial forces were specified in two levels of 15 kg/cm$^2$ and 30 kg/cm$^2$ for the gross sectional area of the column. These axial forces were checked by electric strain gages and the deviations from these two values were less than 1.5 kg/cm$^2$ through the test. In the first-story columns of the destroyed building of the Hakodate College, the axial stress due to dead and live load was also presumed to be about 30 kg/cm$^2$.

These specimens were set on the table of a 100 ton Amsler testing machine supported at the two ends and the load was given at the center as is shown in Fig. 2. The load was at first increased up to 4 tons by an increment of 2 tons and then the load was released. The load was again applied in the opposit direction. The program of these alternately repeated loading is shown in Table 2.

The change of the compressive stress in the columns of frame under the loading is estimated as follows:

$\pm$ 0.644 P/bD for elastic calculation
$\pm$ 0.65 P/bD when yield hinges are formed at the ends of columns

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*Professor, **Assistant Professor, ***Lecturer, Hokkaido University

where, P is the total shearing force of the story and bD is the cross-sectional area of column.

(3) Test results

i) Crack formation under the load

There was not a big difference of crack formation between the columns with introduced axial compression and those of no axial force. A typical pattern of cracks is sketched in Fig. 3. by abstracting only the cracks due to the one-side loading. The sequence of crack formation is as follows:

a) The flexural cracks occurred near the both ends of column.
b) Then, some flexural cracks grew toward the compression corner. The inclination of these cracks was about 45 degree against the axis of column.
c) With the increase of shearing force, many fine cracks due to the excess of bond stress occurred along the axial reinforcement. At the almost same time, the concrete at the compression corner in bending began to crush.
d) Crushing of concrete was developed to the extent of about 5~10 cm and the diagonal cracks developed to link these crushed zone with the bond cracks.

Fig. 3(e) shows the whole crack pattern under the alternately repeated loading. The concrete in the middle part of column would tend to crush in pieces by numerous shearing cracks, if the repetition of loading was further continued. However, in the cases of two specimens out of three specimens prepared to have the initial compression of 30 kg/cm$^2$ , crack formations were limited in the vicinity of ends of the columns (in the range within the distance almost equal to the depth of column from their ends) until the shearing force reached its maximum value, and a large shearing crack suddenly appeared across the middle part of column after the maximum load was attained. These features of crack formation would suggest the cause of the destruction of the building in Hakodate City by the Tokachioki earthquake, 1968.

ii) Strength

The critical shearing forces of the column derived from the test result are shown in Table 3. The data indicated with symbols I~V in the table were obtained in the corresponding steps of loading described in Table 2. And $\tau_{sc}$ and $\tau_u$ mean the shearing stresses derived by the formulae $\tau_{sc} = Q_{sc}/bj$ and $\tau_u = Q_u/bj$, where b is the width of column and j = 7d/8 is the lever arm of internal couple.

The shearing stress corresponding to the initial shear crack and to the ultimate strength of column are again shown in Fig. 4.

There can be seen some influences of axial compression in the column, but the difference is small. The ratio of the mean values of the ultimate strength that corresponded to the introduced axial stresses $\sigma_o$ = 0, 15 kg/cm$^2$ and 30 kg/cm$^2$ was 1:1.01:1.12. The estimated values of $\tau_{sc}$ and $\tau_u$ derived by the empirical formulae of Dr. Arakawa for beams are also indicated in Fig. 4. The test results were slightly less than the values by Arakawa's formulae. It is supposed that the slight difference may be originated by the alternately repeated loading and the alternating additional axial stresses due to reactions, because his formulae are quite acceptable to the

beam subjected to monotonously increasing load.

The results of a preliminary calculation on the ultimate strength of column in flexure are shown in Table 4. The values in the table were computed by replacing the flexural strength with the shearing forces for the column of each side in the frame of specimen. The magnitude of shearing forces was larger than 4.4 tons. In most case, the ultimate strength would be controlled by the yielding of axial reinforcement in the tension side of column, prior to the compressive failure of concrete.

However, the test results were only a half of the presumed values. Therefore, the failure of specimens was not necessarily controlled by the bending stress alone in this case.

The shearing force corresponding to the appearance of bond cracks was 1.9~2.1 tons in the test. The bond stress estimated for these values was about 17~19 kg/cm$^2$. These magnitude of bond stress may be reasonable to the bond strength under the repeated loading. After the bond strength was failed, the shearing force must be transmitted by the concentrated diagonal compression in the direction of 45 degrees against the axis of column passing through the compression corners of the column ends or along the line connecting the both compression corners. Consequently, the concrete near the corner will be accelerated to suffer a compressive failure, even if the apparent shearing force is not so large.

For the evaluation of the bonding capacity, the bond ratio $\mu$ which can be derived by the following conception would be taken into consideration.

$$\frac{\text{max. bonding force of concrete}}{\text{yielding force of reinforcement}} = \frac{\tau_a \cdot \psi}{\sigma_y \cdot A} = \frac{\tau_a \pi \phi}{\sigma_y \cdot \pi \phi^2/4} = \frac{4\tau_a}{\sigma_y \cdot \phi}$$

where, $\tau_a$ = bond strength between concrete and reinforcement
$\psi$ = perimeter of reinforcing bars = $\pi\phi$ x n
$\sigma_y$ = yield strength of reinforcement
A = cross-sectional area of reinforcing bars = $(\pi\phi^2/4)$ x n
$\phi$ = diameter of a reinforcing bar
n = numbers of reinforcing bars

Assuming that $\tau_a$ is proportional to the compressive strength of concrete $F_c$, $\mu$ is introduced as $F_c/\sigma_y\phi$. When $\mu$ shows a large value, the safety against the bond failure will be increased. In this series of test, $\mu$ is calculated as 0.0257. On the other hand, the bond ratio in the case of the collapsed building in Hakodate City was 0.0228, when $\sigma_y$ = 3000 kg/cm and $F_c$ = 150 kg/cm$^2$ are assumed. Since the both values of $\mu$ are not so different, the feature of failure of the building might be similar to those of the test specimens.

iii) Deformation

Examples of the "shear—member rotation angle" diagrams are shown in Fig. 5. The effect of intensity of introduced axial stresses on the deformations at shear cracking and at the ultimate stage are shown in Fig. 6. With an increase of introduced axial load, the rotation angle of members slightly decreased at shear cracking and somewhat distinctly at the ultimate stage. The remarkable shear cracks occurred with about 70 % of the deformation at the ultimate strength in the case of the columns with

no axial load. However, it occurred with about 130 % of the deformation at the ultimate strength for the columns with axial stress of 30 $kg/cm^2$.

Summary

Shear resistance of columns under alternately repeating shear force was examined with specimens which had a shape of two storied frames with single span. Three levels of axial stresses, 0, 15 $kg/cm^2$ and 30 $kg/cm^2$, were previously introduced into columns. The strength of columns was about a half of the presumed values calculated by an ultimate strength theory for flexure, and it almost agreed with the values derived by an empirical formulae for shear strength of beams. From the failing procedure of the columns, it was concluded that the failure of bond between axial reinforcement and concrete would promote the failure of columns. Since the bond ratios which were derived from the magnitude of bond strength against the product of yield strength and diameter of reinforcement in the specimens and the actual columns of the collapsed building in Hakodate City were almost same, the test result would suggest that the same kind of failing procedure might also be expected in the case of that building.

Table 1. Properties of Steel

| Size | Diameter mm | Yield, $\sigma_y$ kg/cm$^2$ | Strength, $\sigma_B$ kg/cm$^2$ | Elong. % | E $10^6$ kg/cm$^2$ |
|---|---|---|---|---|---|
| 13 | 12.73 | 3447 | 4832 | 32.7 | 1.99 |
| # 8 | 4.06 | — | 6902 | 11.3 | — |

Table 2. Program of Loading (Total load in ton)

| Number of repitition of loading / Specimen | I | | II | | III | | IV | | V | | VI | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $+P_1$ | $-P_1$ | $+P_2$ | $-P_2$ | $+P_3$ | $-P_3$ | $+P_4$ | $-P_4$ | $+P_5$ | $-P_5$ | $+P_6$ | $-P_6$ |
| N 0-1 | 4.0 | 4.0 | 8.0 | 8.0 | 8.2 | 7.0 | | | | | | |
| N 0-2 | 4.0 | 4.0 | 6.0 | 6.0 | 8.0 | 8.0 | 10.0 | 9.4 | 8.95 | | | |
| N 0-3 | 4.0 | 4.2 | 6.0 | 6.0 | 8.0 | 7.8 | 7.6 | | | | | |
| N15-1 | 4.0 | 4.0 | 6.0 | 6.0 | 8.0 | 8.0 | 9.05 | 7.6 | 7.6 | | | |
| N15-2 | 4.0 | 4.0 | 6.0 | 6.0 | 8.0 | 8.0 | 9.7 | 8.5 | 8.45 | 7.55 | 7.0 | |
| N15-3 | 4.0 | 4.0 | 6.0 | 6.0 | 8.0 | 8.0 | 8.75 | 7.75 | 7.7 | 6.2 | 6.55 | |
| N30-1 | 4.0 | 4.0 | 6.0 | 6.0 | 8.0 | 8.0 | 10.0 | 9.9 | 9.8 | 8.74 | 8.55 | |
| N30-2 | 4.0 | 4.0 | 6.0 | 6.0 | 8.0 | 8.0 | 9.9 | 9.45 | 9.2 | 7.9 | 7.7 | 6.6 |
| N30-3 | 4.0 | 4.0 | 6.0 | 6.0 | 8.0 | 8.0 | 9.9 | 8.8 | 8.5 | | | |

Table 3. Test Result

| Specimen | | Shearing force for a column ton | | | | | | | | Shearing stress kg/cm$^2$ | | Additional axial force by reaction* |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | $Q_{BC}$ | | $Q_{SC}$ | | $Q_F$ | | $Q_u$ | | $\tau_{SC}$ | $\tau_u$ | |
| N 0-1 | + | 1.03 | I | 1.53 | II | 2.03 | II | 2.08 | III | 13.6 | 18.5 | 16.9 |
| | − | 1.17 | I | 0.92 | II | 2.17 | II | 2.17 | II | 8.2 | 19.3 | 17.9 |
| N 0-2 | + | 0.96 | I | 1.53 | II | 2.43 | IV | 2.53 | IV | 13.5 | 22.3 | 20.6 |
| | − | 1.12 | I | 1.67 | II | 2.17 | IV | 2.52 | IV | 14.7 | 22.2 | 20.4 |
| N 0-3 | + | 0.88 | I | 1.53 | II | 1.93 | IV | 2.03 | III | 13.5 | 17.9 | 16.5 |
| | − | 0.67 | I | 1.67 | II | — | | 2.12 | III | 14.7 | 18.7 | 17.2 |
| Mean | + | 0.99 | | 1.53 | | 2.13 | | 2.21 | | 13.5 | 19.6 | 18.0 |
| | − | 0.99 | | 1.42 | | (2.17) | | 2.27 | | 12.5 | 20.1 | 18.4 |
| N15-1 | + | 0.78 | I | 1.53 | II | 2.03 | IV | 2.29 | IV | 13.5 | 20.2 | 18.6 |
| | − | 0.67 | I | 1.55 | II | 2.12 | III | 2.17 | III | 13.7 | 19.1 | 17.6 |
| N15-2 | + | 0.78 | I | 1.48 | II | 2.28 | IV | 2.46 | IV | 13.1 | 21.7 | 20.0 |
| | − | 0.67 | I | 1.67 | II | 1.67 | III | 2.30 | IV | 14.7 | 20.3 | 18.7 |
| N15-3 | + | 1.03 | I | 1.28 | II | 1.98 | III | 2.22 | IV | 11.3 | 19.6 | 18.0 |
| | − | 0.67 | I | 1.67 | II | 2.12 | III | 2.17 | III | 14.7 | 19.1 | 17.6 |
| Mean | + | 0.87 | | 1.43 | | 2.10 | | 2.32 | | 12.6 | 20.5 | 18.8 |
| | − | 0.67 | | 1.63 | | 1.97 | | 2.21 | | 14.4 | 19.5 | 17.9 |
| N30-1 | + | 1.03 | I | 1.99 | III | 2.03 | V | 2.53 | IV | 17.4 | 22.2 | 20.5 |
| | − | 1.17 | I | 1.67 | III | 2.42 | IV | 2.65 | IV | 14.6 | 23.2 | 21.5 |
| N30-2 | + | 1.03 | I | 2.03 | III | 2.51 | IV | 2.51 | IV | 17.8 | 22.0 | 20.4 |
| | − | 1.17 | I | 2.17 | III | 2.42 | IV | 2.53 | IV | 19.0 | 22.2 | 20.6 |
| N30-3 | + | 1.03 | I | 1.53 | III | 2.48 | IV | 2.51 | IV | 13.6 | 22.3 | 20.4 |
| | − | 0.67 | I | 1.67 | III | 2.17 | IV | 2.37 | IV | 14.8 | 21.0 | 19.3 |
| Mean | + | 1.03 | | 1.85 | | 2.34 | | 2.52 | | 16.3 | 22.2 | 20.5 |
| | − | 1.00 | | 1.84 | | 2.34 | | 2.52 | | 16.2 | 22.1 | 20.5 |

$Q_{BC}$ corresponds to the first bending crack
$Q_{SC}$ corresponds to the first shearing crack
$Q_F$ corresponds to the beginning of the compressive failure
$Q_u$ corresponds to the maximum load

$\tau_{sc} = Q_{sc}/bj$
$\tau_u = Q_u/bj$

* Additional axial force by the reactions of the frame under lateral force. The values in this Table were computed by $\pm$ 0.65P/bD and are shown in kg/cm$^2$ by omitting $\pm$ signs

Table 4. Shearing Force for Column Corresponding to Ultimate Flexural Strength

| Initially introduced stress $kg/cm^2$ | Columns under the compressive reaction | | Columns under the tensile reaction | |
|---|---|---|---|---|
| | Max. shear force ton | Position of failure | Max. shear force ton | Position of failure |
| $\sigma_0 = 0$ | 6.04 | tens. side | 4.44 | tens. side |
| $\sigma_0 = 15$ | 5.90 | comp. side | 4.70 | tens. side |
| $\sigma_0 = 30$ | 5.60 | comp. side | 5.04 | tens. side |

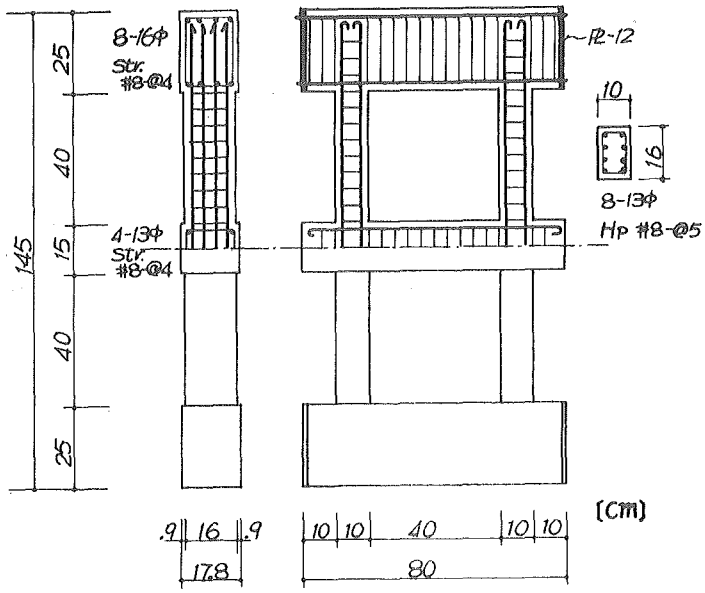



Fig. 1. Specimen

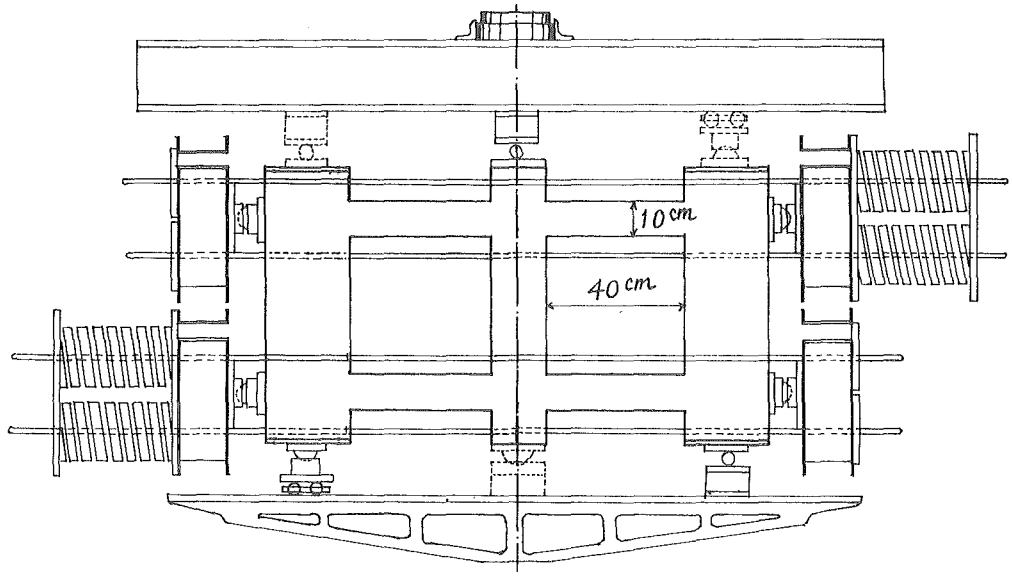



Fig. 2. Loading Method

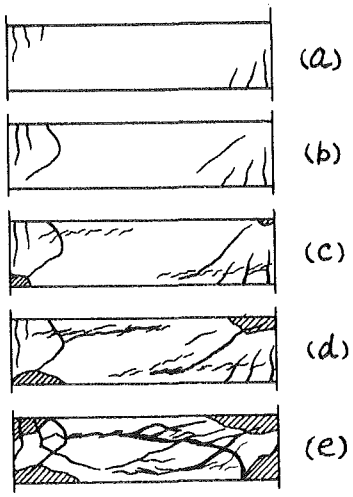



Fig. 3. Typical crack pattern

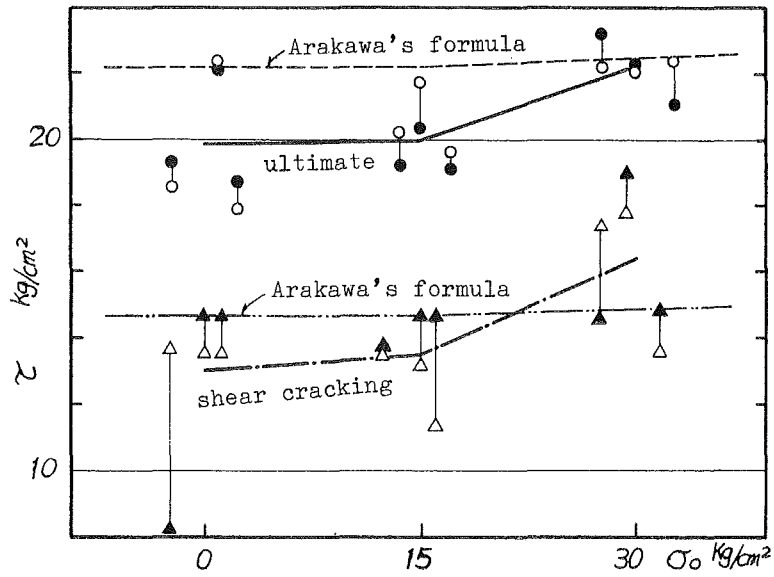



Fig. 4. Strength

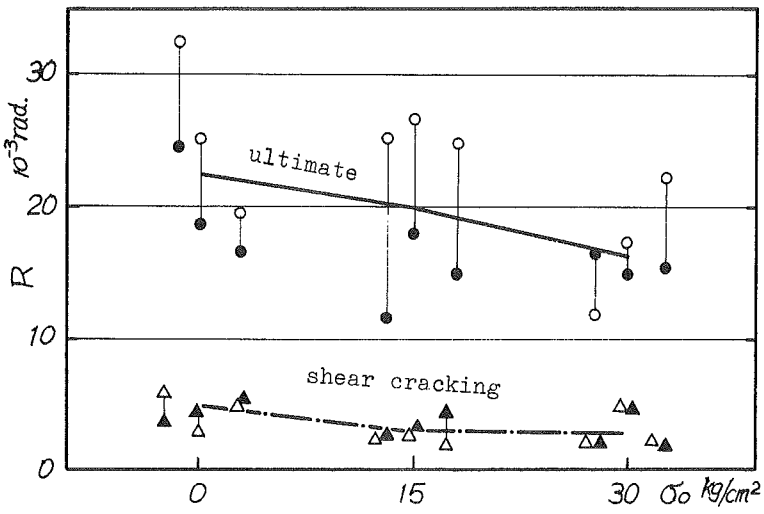



Fig. 5. Deformation

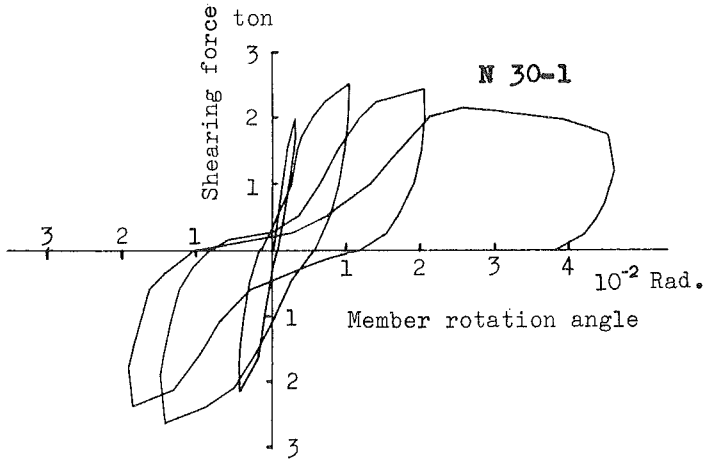



Fig. 6. Load — Deformation relation