U.S.-JAPAN COOPERATIVE RESEARCH ON R/C FULL-SCALE BUILDING TEST
(Part 6: ULTIMATE MOMENT-RESISTING CAPACITY)

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SUMMARY

This paper discusses the ultimate moment-resisting capacity of the building. The maximum base shear coefficient of the RC full-scale building reached as much as 0.35 (Ref.1), approximately 1.5 times that obtained from the preliminary inelastic plane frame analysis. From strain measurements in reinforcing bars, this discrepancy is shown to be attributable to (a) the slab reinforcement that significantly contributed to the beam negative moment resistance, and (b) transverse beams connected to the shear wall that resisted the upward movement of the tensile boundary column of the wall increasing the flexural resistance of the shear wall.

INTRODUCTION

This paper discusses (a) the strain distribution in slab reinforcing bars in order to determine the effective slab width that contributes to the beam flexural resistance, (b) the sequence of plastic hinge formation on the basis of the observed strain measurements at the critical regions of beams and columns, and (c) the ultimate resistance of the building using different effective slab widths in evaluating flexural resistance of beams. These problems are worth discussing in accurately evaluating the ultimate moment-resisting capacity of real buildings against the earthquake load.

EFFECTIVE SLAB WIDTH

Figure 1 shows crack patterns observed on the top surface of the second floor slab after SPD-4, in which the roof-level displacement reached as large as 1/64 of the total story height. Two kinds of crack patterns were observed on the slab; i.e., (a) cracks observed perpendicular to the loading direction mostly extending from the flexural cracks in the longitudinal beams, and (b) cracks radiating from the boundary columns of the shear wall. The latter kind of the cracks was believed to accompany the upward displacement of the tensile boundary column. As is shown in Fig.2, the boundary column in tension was observed to elongate as much as 42mm in the first story because the neutral axis of the shear wall located close to the compressive boundary column.

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Strains in the slab reinforcing bars were measured in the four sections (W-W, X-X, Y-Y, and Z-Z) at the second floor level as shown in Fig.3. The distribution of slab reinforcement strains observed in the four sections is shown in Fig.4 for peak roof-level displacements of +1/64 and -1/67 of the total story height. The strain measurements in positive direction in section X-X and in negative direction in section W-W are not shown because they remained small under positive bending in the longitudinal beams.

In section X-X, along the external frame 4 and perpendicular to the direction of loading, the distribution of strains in the top slab reinforcement clearly shows strains larger than the yield strain (2000 micro-strain) especially between column lines A and B. The strains were larger near the column lines or near the longitudinal beams. However, the slab reinforcement strains in the overhang portion were small and did not reach the yield strain. This is probably because (a) the overhang portion was cantilevered, and (b) the edge wall along the frame 4 was not extended to the overhang portion.

In section W-W, along the internal frame 3 and perpendicular to the direction of the loading, the distribution of strains in the bottom slab reinforcement shows a pattern similar to the strain distribution in section X-X, but the amplitudes of the strains were smaller than those in section X-X. One reason of the difference in amplitude may be related to the position of the slab reinforcement; i.e., top in section X-X and bottom in section W-W. It should also be noted that slab bars were spliced over the transverse beams in the second floor, as shown in Fig.5, with ample splice length due to error in supervision of construction work. Therefore, both slab reinforcing bars were effective in resisting negative bending in the longitudinal beams. This error was noticed after the construction of the second floor slab, and the position of the splice was changed to the center of the adjacent two transverse beams in the other floor levels.

In section Y-Y, along the central frame B and parallel to the direction of loading, the strains were generally small except in the slab reinforcement near the boundary column in tension; i.e., the strain in slab reinforcement at column line 3 at +1/64 displacement, and that at column line 2 at -1/67 displacement. The strain in the slab reinforcement at the compressive boundary column did not show such response. Therefore, the larger strain near the tensile boundary column was believed attributable to the vertical movement at the tension side of the shear wall. In section Z-Z, along the exterior frame A and parallel to the direction of loading, the amplitudes of strains in the bottom slab reinforcement were rather small.

On the basis of the observation in the slab reinforcement strains, effective width of slab that contributed to the flexural resistance of the longitudinal beam was so determined that,

$$\sigma y \times Aeq = \sum_{i=1}^{n} \sigma i \times Ai \qquad (1)$$

where

σy : yield stress of slab bar

Aeq : sum of the slab bar sectional areas in effective width

n : number of slab bars

σi : stress of slab bar derived from the measured strain

Ai : slab bar sectional area

As shown in Fig.6, the effective width for a longitudinal beam varied from 80 to 240cm, much larger than that suggested in the Architectural Institute of Japan (AIJ) Reinforced Concrete Standards or the American Concrete Institute Code. Note that slab along the transverse beam did not contribute to the flexural resistance of the beam under longitudinal loading except for the slab connected to the tensile boundary column of the shear wall.

DISPLACEMENT DUCTILITY OF BUILDING

Figure 7 shows the observed relationship of the roof-level displacement and the base shear. As the primary lateral load-resisting element of this building was the shear wall, the yielding displacement was defined as a displacement at which the yielding of the reinforcement in the boundary column was first observed. Note that the building behaved in a ductile manner even at a displacement six times the yielding displacement.

SEQUENCE OF PLASTIC HINGE FORMATION

The strains were measured at the critical sections of beams and columns in frames A and B. The sequence of plastic hinge formation was determined from the strain measurement (Fig.8) for the negative loading. The number in circle indicates the displacement level, as defined in Fig.7. The first plastic hinge formed at the base of the shear wall at a translational angle, the roof-level displacement divided by the total story height, of 1/480 - 1/320. At 1/320 - 1/170 translational angle, beam ends at column lines 1 and 3 in frame B and column line 1 in frame A, subjected to positive bending, succesively yielded. At 1/170 - 1/130 translational angle, the other beam ends in frame B and beam ends at column line 4 in frame A, subjected to negative bending, yielded. In subsequent loading with larger than 1/130 translational angle, interior beam ends in frame A and columns on line 2 in frame A yielded. The column yielded at the top and the bottom of the building in displacement level 3. Transverse beams connected to the tensile boundary column, although not shown, yielded in displacement level 5.

The followings summarize the observation about the sequence of plastic hinge formation;

(1) The beam ends in frame B subjected to positive bending yielded in the stages earlier than those subjected to negative bending,

(2) The exterior beam ends in frame A yielded in the stages earlier than the interior beam ends,

(3) The ends of beams located at one side of the same column line yielded at a similar displacement level,

(4) The strains measured in columns on line 2 in frame A were greater than the yield strain under negative loading, while the strain remained smaller than the yield strain under positive loading. Although the strains of columns on line 3 in frame A were not measured, they are expected not to have yielded in negative loading from the symmetry. In other words, columns on line 2 and 3 in frame A behaved in a different manner; i.e., plastic hinges formed in columns on line 2 while the beam yielding occured on both sides of column line 3. This discrepancy is explained by the fact that the upward displacement of the tensile boundary column gave rise to shear in the transverse beams, which in turn caused tensile load in columns on line 2 reducing the moment resisting capacity.

ULTIMATE RESISTANCE OF BUILDING

The ultimate resistance of the building was calculated by "upper bound theorem". The ultimate flexural resistance of beams was calculated with following effective slab widths;

Case 1 : Effective width defined by AIJ Standards (60cm on either side of the beam),
Case 2 : Entire slab width,
Case 3 : Slab width defined in Fig.6.

The ultimate moment of the shear wall was calculated on the basis of the stresses of the longitudinal reinforcements in the boundary column and in the wall panel corresponding to their strains measured in the test. To calculate the ultimate column moment, axial force was obtained assuming the yielding at the end of all beams. Furthermore, the tensile axial forces imposed on columns on line 2 in frame A, coming from shear forces in the transverse beams, were considered. Transverse beams connected to the tensile boundary columns were considered in calculating the ultimate resistance of the building to take account of their resistance to the upward movement of the tensile boundary columns due to the yielding of the shear wall. They were assumed to yield at the two ends.

Obtained collapse mechanisms are shown in Fig.9. At each beam-column joint, the moment capacities of the beams and the columns were compared to determine whether the beams or the columns yielded at the joint. In Case 1, beam ends yielded throughout the building except at the top and the bottom stories. In Case 2, the increased beam moment caused column yielding at most of the interior joints in frame A. In Case 3, the columns on line 2 in frame A were calculated to yield, while most beams yielded on both sides of column line 3 in frame A, similar to the observed behavior shown in Fig.8.

The ultimate base shear strengths obtained from the three analyses and the test are listed in Table 1. The ultimate base shears calculated from Cases 2 and 3 agree well with the observed strength, while Case 1 estimated an unreasonably low value.

The contribution of each member (frame) to the ultimate resistance was examined. The resistance ratio is defined as a resistance of each member (frame) divided by the ultimate resistance of the building for Case

3 (Table 2). The two open frames A and C carried 40% of the building resistance, transverse beams, by increasing the axial load in the shear wall, contributed 8% and frame B carried 52%, among which shear wall resisted 35% and frame members 17%. The contribution of the one-bay slender wall arranged in the building reached 35% of the building resistance, if the contribution of the transverse beams was included in it, it did as much as 43%.

CONCLUDING REMARKS

The following findings are obtained from the study;

(1) Slab reinforcement contributes to the flexural resistance of the longitudinal beam to the extent much larger than suggested in many building codes,

(2) The existence of a shear wall influences the behavior of members in the adjacent frames,

(3) The ultimate resistance of the building should be evaluated by taking into account the contributions of the slab to the beam flexural resistance and that of the transverse beams connected to the tensile boundary column, to the flexural resistance of the shear wall in terms of increasing the axial forces.

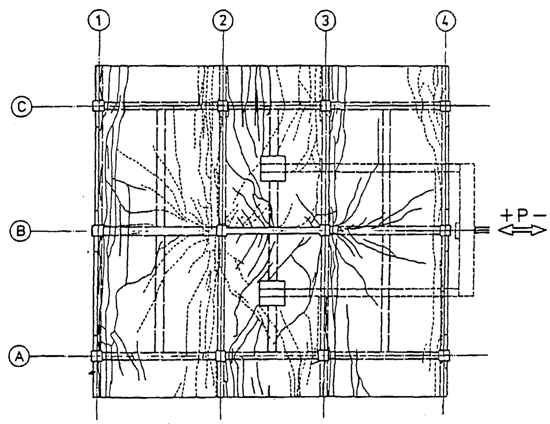


Fig. 1 Crack Patterns of Second Floor Slab

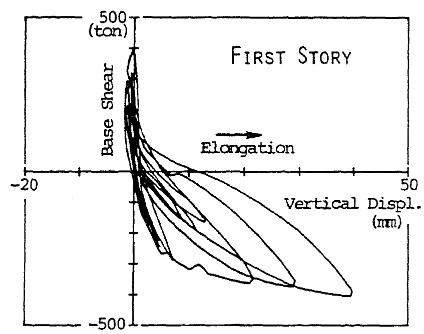


Fig.2 Elongation of Boundary Column

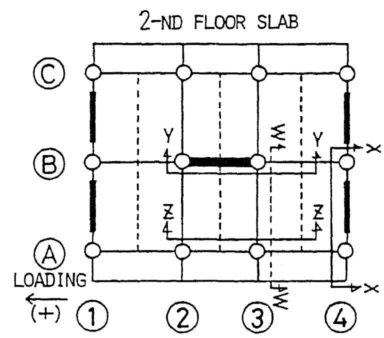


Fig. 3 Slab Bar Strain Measurements in the Second Floor Level

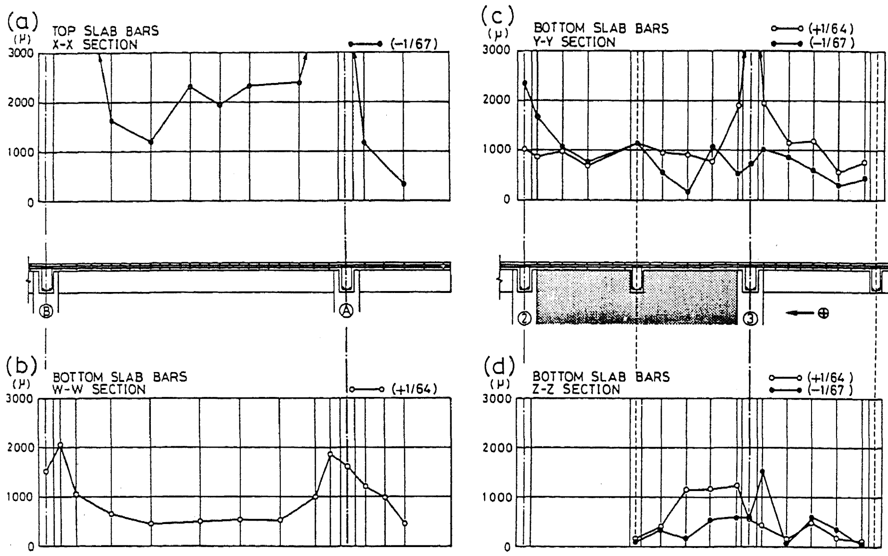


Fig. 4 Slab Bar Strain Distribution

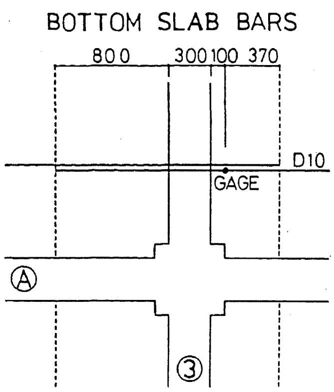


Fig. 5 Lap Splice of the Second Floor Slab bars

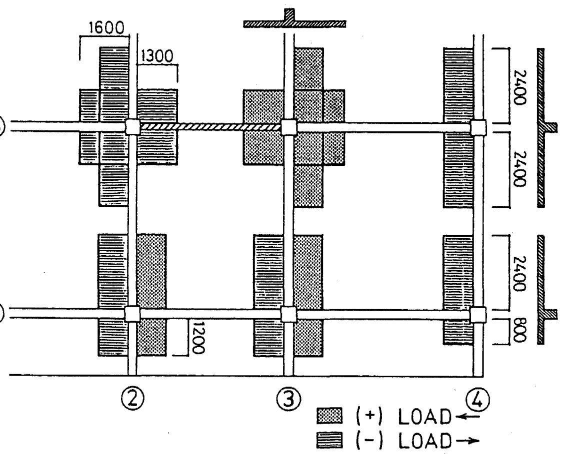


Fig. 6 Effective Slab Width

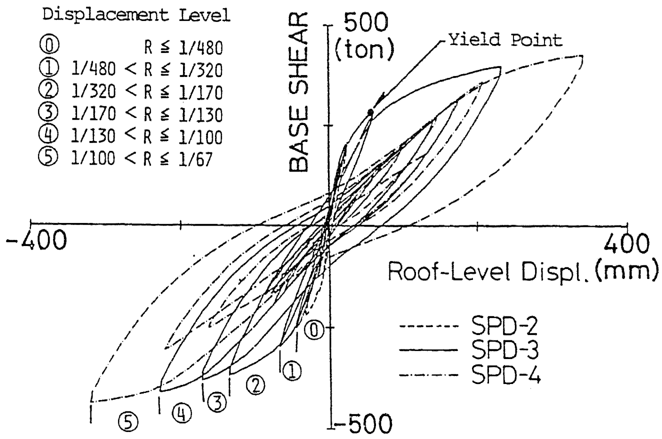
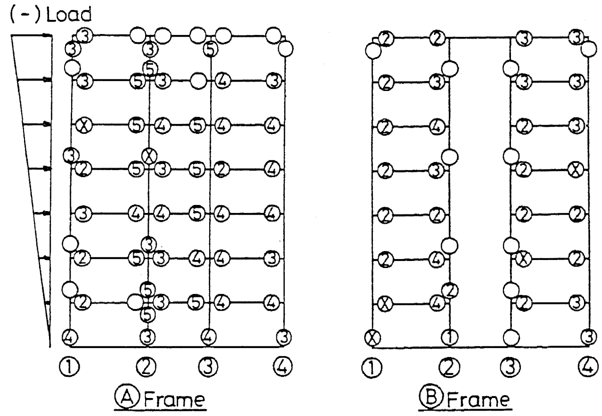


Fig. 7 Roof-Level Displacement and Base Shear



①: strain gauge location

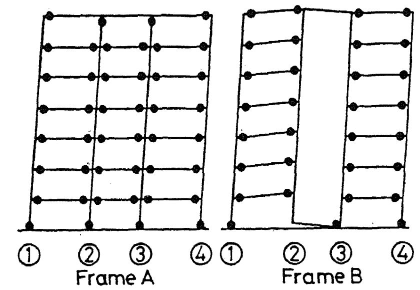
The number in circle indicates the displacement level.

⊗: failed to measure the strain

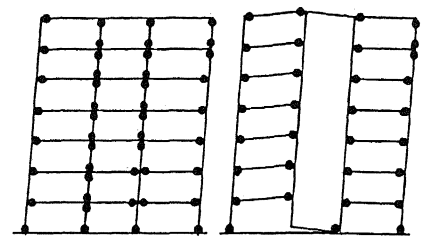
○: did not yield

Fig. 8 Sequence of Plastic Hinge Formation

Case (1) Slab Width (Suggested by A.I.J.)



Case (2) Entire Slab Width



Case (3) Slab Width (Fig.6)

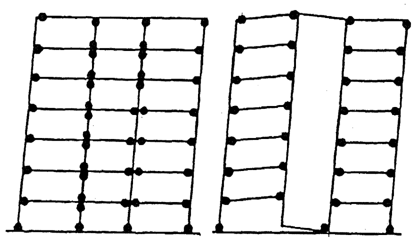


Fig. 9 Hinge Mechanism Obtained from the Analysis

Table 1 Ultimate Strength Capacity

Effec. Slab Width	Calculated Case (1)	Calculated Case (2)	Calculated Case (3)	Tested
Ulti. Capacity (t)	271	429	401	439

Table 2 Contribution of Each Member (frame) to Ulti. Strength Capacity

	A,C Frames	B Frame		Trans.Beams
		Wall	The Others	
%	40	35	17	8

REFERENCE

1) S. Okamoto, et al.,"A Progress Report on the Full-Scale Seismic Experiment of a Seven Story Reinforced Concrete Building - part of the U.S.-Japan Cooperative program", Building Research Institute Research Paper No.94, JAPAN, March 1982.