Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.VI)

9-2-12

SEISMIC BEHAVIOR OF THREE STORY FULL SCALE CLAY BLOCK PLANAR FRAME UNDER CYCLIC LATERAL LOADING

Takaaki NISHI[1], Takashi KAMINOSONO[2],
Masaomi TESHIGAWARA[3] and Mototaka MATSUNO[4]

[1] Technical Research Institute, Hasegawa Komuten Co., Ltd., Minato-ku, Tokyo, Japan
[2] Senior Research Engineer, Building Research Institute, Ministry of Construction, Tsukuba, Japan
[3] Research Engineer, BRI, MOC, Tsukuba, Japan
[4] Technical Research Institute, Hazama-gumi Ltd.,Minato-ku,Tokyo,Japan

SUMMARY

A three story full scale reinforced masonry planar frame using clay block units (Fig. 1) was tested under cyclic lateral loading. Objectives of this test are not only to know the total behavior of the planar frame, but also to get basic data for establishing the design guidelines of medium-rise reinforced masonry (RM) buildings. The test specimen kept its strength and deformation capacity that was requested in the tentative RM design guidelines.

OBJECTIVES

Structural elements in a building such as walls, beams, and wall-beam subassemblages were tested in 1985 and many useful informations concerning the structural behavior of elements were obtained. Main objectives of a full scale RM planar frame test are to know the total structural behavior of this test and to know behavior of structural elements which compose the test specimen frame. Construction technique was also checked while the specimen was under construction.

SPECIMEN

Prototype of Test Structure A prototype of this test structure is a five story concrete masonry apartment building (Ref.1) which designed in accordance with the tentative design guidelines of RM buildings. The specimen described in this paper is a part of this prototype structure. Plan of the prototype structure is illustrated in Fig. 2. Thickness of wall is 19cm. Ratios of total wall length to floor area of the structure are 15.2cm/m^2 in longitudinal direction (loading direction). The lower 3 story part of the Frame-Y3 is selected as the specimen, because the Frame-Y3 had various types of wall and beam elements. The tested portion is also shown in Fig. 2.

Details of Specimen Figure 3 shows plan, elevation, and reinforcing bar details of the specimen. The reinforced clay block masonry test specimen is a three story in each story height of 2.8m. Elements consisting of test building are listed as followes; 1) wall with 3.79m depth (Wall-A), 2) wall with 1.09m depth (Wall-B), 3) flanged wall with 1.09m depth and with varying axial loading (Wall-C), 4) short span beam whose depth and reinforcement is decreased (Beam-A),

5) long span beam (Beam-B) and 6) wall-beam joint panel. All of walls and beams were 19cm in thickness. The specimen had a RC loading beam at the top floor level. Small openings for piping were arranged in Wall-B and in wall-beam joint panels of Wall-A and Beam-A (See Fig. 4). Spiral reinforcing bars were placed around the flexural reinforcing bars at the bottom end of walls, where the flexural reinforcing bars were spliced, in order to prevent buckling and bond-slippage of reinforcing bars in early stage of loading.

Material Properties Prism compressive strength and compressive strength of slab concrete are tabulated in Table 1 and 2. Compressive strength for net area of clay block is 657 kg/cm^2. Mechanical properties of reinforcing bars are listed in Table 3.

Table 1. Prism Compressive Strength

Story	Strength[kg/cm^2]
1 F	273
2 F	272
3 F	231

$E = 239$ ton/cm^2
$G = 102$ ton/cm^2

Table 2. Concrete Compressive Strength

Story	Strength[kg/cm^2]
2 F	327
3 F	314
R F	319

Table 3. Mechanical Properties of Reinforcing Bars

Bar [mm]	Material [SD]	Area [cm^2]	Yield Stress [kg/mm^2]
10	30	0.71	39
13	30	1.27	37
16	35	1.99	39
19	35	2.87	40
22	35	3.87	41
25	35	5.07	40

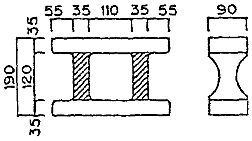


Fig. 1 Clay Block Unit

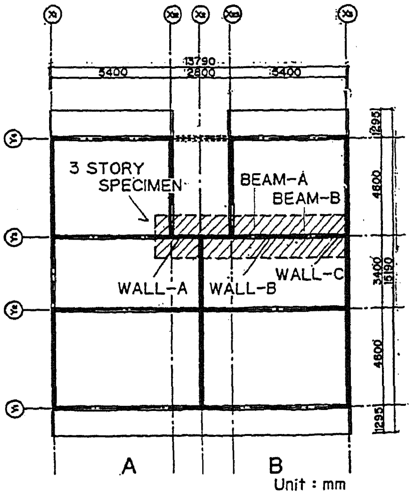


Fig. 2 Plan of the Prototype Building

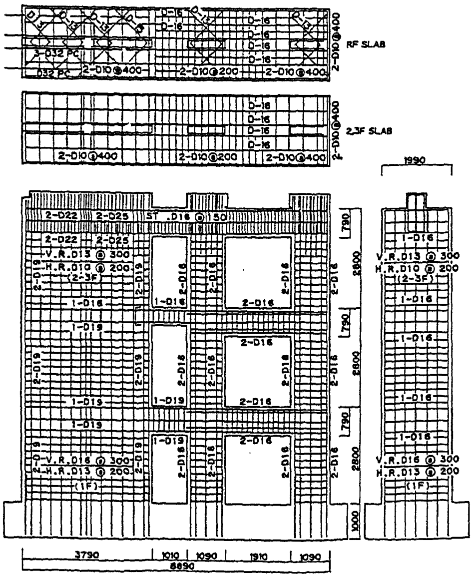


Fig. 3 Arrangement of Reinforcing Bars

LOADING METHOD

Test setup is shown in Fig. 4. Horizontal load was applied to the top of the Wall-A by four actuators. In pushing direction (negative loading), horizontal load would be distributed on walls-A, B and C through R/C loading beam, and in pulling direction (positive loading), loading is carried out by PC bars which were set in the loading block. This specimen was the model of lower three story parts of five story building. Vertical load were applied to the top of each wall by eight center hole jacks in order to adjust vertical stress at the bottom end of walls to that of walls in the five story building. Figure 5 shows the horizontal loading hysteresis. Experiment was controlled by the first story drift angle. But up to the nominal shear stress ($\bar{\tau}$)of 10 kg/cm^2 which was the value divided total horizontal load by wall area in loading direction; i.e.,11,343 cm^2, the experiment was controlled by the value of horizontal load.

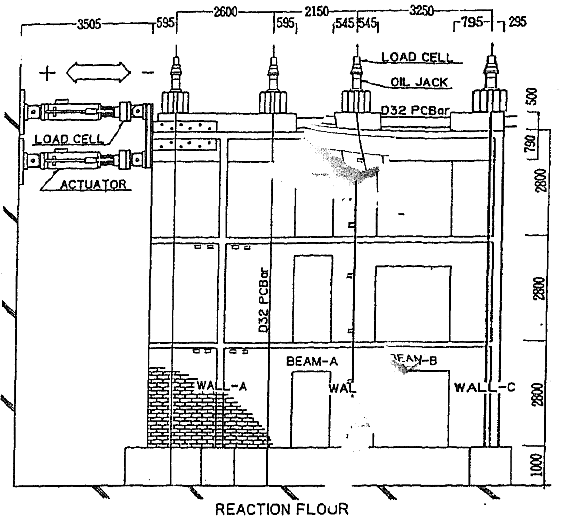


Fig. 4 Test Setup

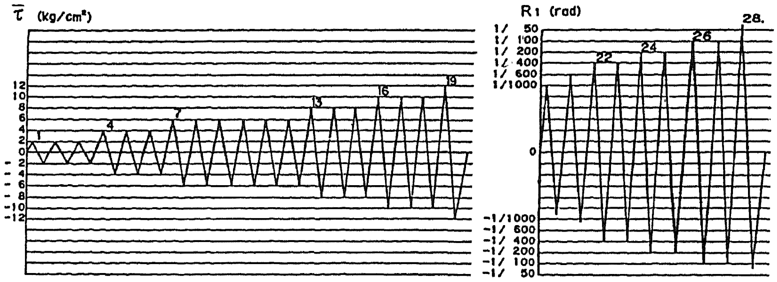


Fig. 5 Horizontal Loading Hysteresis

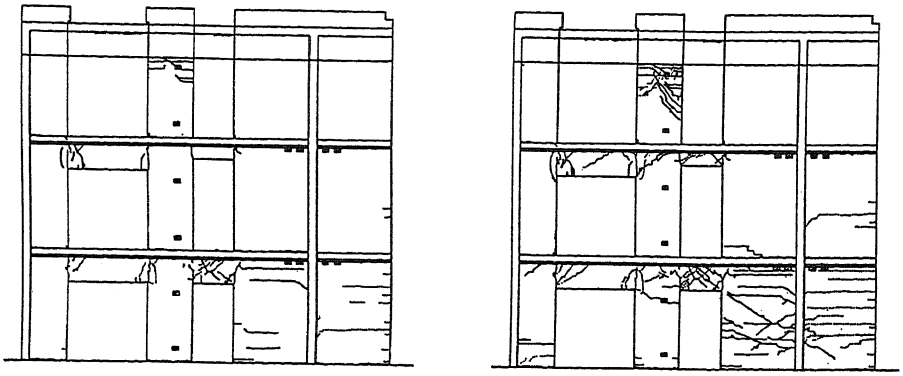
TEST RESULTS

Figures 6 and 7 show the crack pattern and the yielding development of reinforcing bar at several shear stress levels or deformation levels, respectively. In those figures deformation was described as the first drift angle story. Development of crack and yielding of reinforcing bars were as follows;

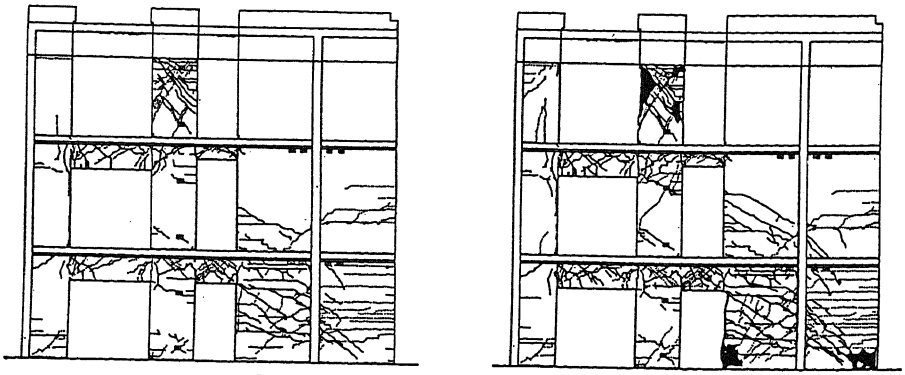
1) There were no cracks in the specimen up to the nominal shear stress of 4 kg/cm^2 that corresponded to temporary design stress.
2) Reinforcing bars at the bottom of Wall-A and short beam except for 2F lower part reached its yielding strain where the nominal shear stress of reached 10 kg/cm^2.
3) Shear cracks were observed at Wall-C in the 3rd story and at the short beam in the 2nd story where the nominal shear stress reached 12 kg/cm^2.
4) When the drift angle reached $2.5\text{x}10^{-3}$ radian, shear cracks occurred at the 1st story part of Wall-A, 3rd story Beam-A, 2nd and 3rd story Beam-B and wall-beam joint panel zone.
5) At the drift angle of $5.0\text{x}10^{-3}$ radian, maximum horizontal load, 176 ton ($\bar{\tau}$=15.5kg/cm^2) was recorded. From the yielding development of reinforcing bars, yielding hinges were estimated to be made at the bottom of all walls, the end of all beams and both ends of 2F Wall-B in negative direction.

6) When the drift angle reached almost $10.0x10^{-3}$ radian, Wall-A failed in compression at its compression toe and 2F short beam failed in shear.

7) Over the drift angle of $10.0x10^{-3}$ radian, the faceshell or grout concrete in 3F Wall-C, in 2F short beam, and in the compressive failure zone of Wall-A spalled. But it was observed that joint steel arranged in half part of 2F short beam prevented those faceshells falling down.



a) $R1=1.2*10^{-3}$ rad. b) $R1=2.5*10^{-3}$ rad.



c) $R1=5.0*10^{-3}$ rad. d) $R1=10.*10^{-3}$ rad.

Fig. 6 Crack Pattern

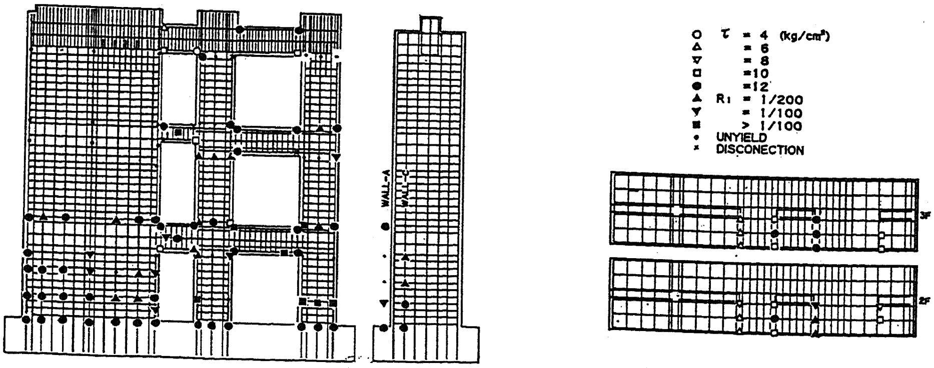


Fig. 7 Yielding Stage of Reinforcing Bars

The relationship of first story horizontal drift of Wall-A vs. horizontal load is shown in Fig. 8. As above mentioned, from the drift angle of 5.0x10^{-3} rad. to that of 10x10^{-3} rad., the 2nd story short beam failed in shear and Wall-A failed in compression at the critical section and strength degradation occurred. However, even strength degradation occurred, the nominal shear stress level at 10x10^{-3} radian drift angle exceeded 12kg/cm^2 which almost corresponds to the shear stress level required by the RM design guidelines.

DISCUSSIONS ON TEST RESULTS

Horizontal Stiffness   Change of the story horizontal stiffness in accordance with increase of the first story drift angle is shown in Fig. 9. The stiffness obtained from elastic analysis is also indicated in this figure. The stiffness obtained from elastic analysis agrees with those obtained from the test at the nominal shear stress of 4 kg/cm^2 which is considered as temporary design shear stress. Therefore, the specimen was still elastic under the temporary design force level. At the first and second stories, the stiffness degradation becomes larger at the nominal shear stress of 12 kg/cm^2. The reason is considered to be that the yield mechanism was totally developed at this nominal shear stress level.

Drift Angle of Beams   The drift Angle of Beam-B was almost equal to the story drift angle. On the other hand, the drift angle of Beam-A was 1.6 to 2.0 times as large as the story drift angle. It is considered that Beam-A was forced to produce large deformation by Wall-A.

Behavior of Wall-A   Figure 10 shows the deformation mode of Wall-A at several loading stages. It is evident that the deformation mode of Wall-A was flexural. Until the first story drift angle reached 5x10^{-3} radian, Wall-A deformed only with the rotation at its bottom.

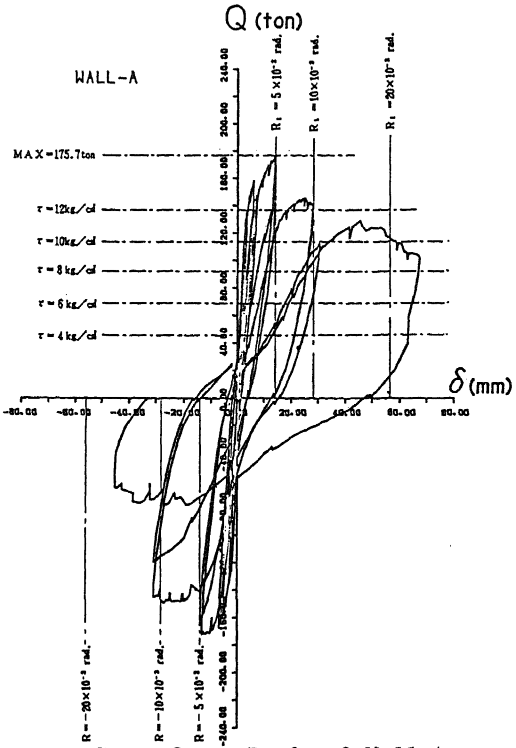


Fig. 8 First Story Drift of Wall-A vs. Lateral Loading Relationship

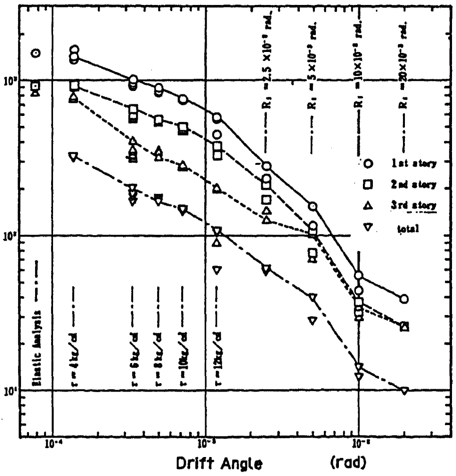


Fig. 9 Story Stiffness of Several Stage

Effect of Transverse Wall Figure 11 shows the strain distribution of reinforcing bars in the transverse wall. The strain was very large at the critical section. The reinforcing bar set in the outer edge of wall did not work so effectively up to the drift angle of 1.2×10^{-3} radian ($\tau=12\ kg/cm^2$). This reinforcing bar, however, worked after the drift angle of 2.5×10^{-3} radian.

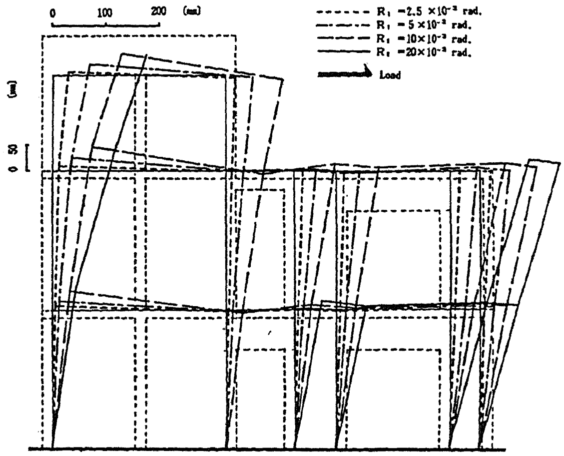


Fig. 10 Deformation Mode

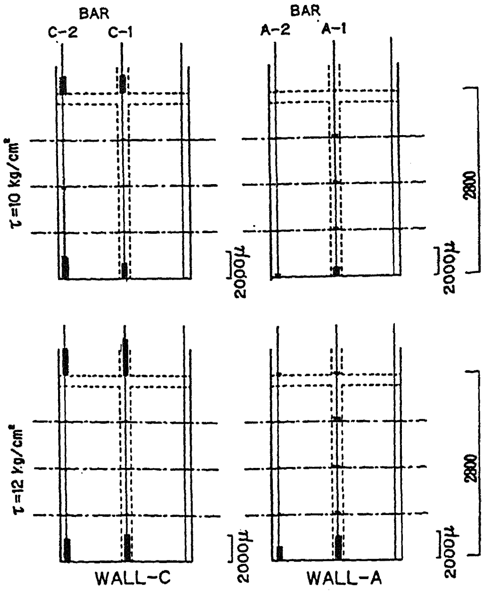


Fig. 11 Strain Distribution of reinforcing Bars

CONCLUSIONS

The major findings from this full scale test are as follows ;

1) The specimen was still elastic and had only minute flexural cracks within the temporary shear stress level.
2) Shear story at the drift angle of 10×10^{-3} radian was over the value that was requested by the tentative design guidelines.
3) Short span beam was forced into 1.6 to 2.0 times drift angle as large as story drift angle by Wall-A.
4) In large deformation range, story drift angle was mainly represented with the rotation at the bottom of wall.
5) Transverse wall was effective totally on the strength at the maximum shear stress level.

ACKNOWLEDGMENTS

The authors wish to express their gratitude to the members of Technical Coordinating Committee on Masonry Research and the group of experiment.

REFERENCE

1. Y.Yamazaki and M.Teshigawara, "Earthquake Response of Five Story Reinforced Concrete Masonry Test Building," Proceedings of the Third Conference on Dynamic Response of Structures, EM Div., ASCE, 1986 pp.55-70