RESULTS FROM THE SIMULATED EARTHQUAKE TESTS OF A FULL SIZE REINFORCED CONCRETE BUILDING IN TSUKUBA, JAPAN

V. Sattary-Javid (I)
J. K. Wight (II)
Presenting Author: J. K. Wight

SUMMARY

This paper presents some of the results from a simulated earthquake test on a full scale reinforced concrete building. The discussion of the observed behavior of the building includes: the behavior of beam-column connections and the effect of the shear wall on the deformation of frame members. It is shown that the reduction of the stiffness of frame elements did not significantly reduce the overall stiffness of the building. The percentage of the total overturning moment carried by the shear wall was estimated from the measured deformations at the base of wall.

INTRODUCTION

A series of simulated earthquake tests were conducted on a full scale seven story reinforced concrete building at the Japanese government's Building Reseach Institute in Tsukuba, Japan. The tests were part of the U.S.-Japan Cooperative Testing Agreement Utilizing Large Scale Testing Facilities. The test building comprised three frames in the loading direction. A shear wall was located at the center of the middle frame (frame B). Two other frames, A and C, were connected to frame B through floor slabs and transverse beams. The foundation of the building was rigidly connected to the test floor of the laboratory. A computer-actuator on-line (psuedo-dynamic) testing procedure was employed to simulate dynamic response of the building to prescribed seismic exitation. The lateral displacement was applied along the building's principal axis by hydraulic actuators at each floor level. Over seven hundred channels of displacement and strain data were recorded during the tests. In the following sections some of the data is presented and the effect of the applied forces on the beam-column joints and the shear wall is discussed.

BEAM-COLUMN CONNECTIONS

In order to evaluate the design of beam-column connections, the proportions and reinforcement details of joints are compared to the draft recommendations of ACI Committee 352 [1]. The philosophy reflected in the draft recommendations of ACI-352 is that through the joint columns should have a minimum amount of transverse reinforcement regardless of the axial and shear stresses in the joint. The nominal joint shear stress should be

(I) Graduate Research Assistant,
(II) Associate Professor, Dept. of Civil Engineering, The University of Michigan, U.S.A.

limited to the specified values and flexural strength ratio should be less than 1.4.

The beam-column joints of the seven story test building had a flexural strength ratio of 2.3 and 3.8 for internal and external joints, respectively. The nominal shear strength was more than three times the calculated shear stress for both internal and external joints. However, the confinement reinforcement was only about 55 percent of the ACI-352's recommended value and the requirement on maximum spacing of column longitudinal bars was violated.

In order to investigate the potential slippage of bars through beam-column connections, the variation of strain along the length of reinforcing bars inside joint was studied. Figure 1 shows strain at three points along the length of a column longitudinal bar inside an internal beam-column joint at several load points. These load points correspond to a quarter cycle of loading history. This diagram also shows the lateral displacement at the roof level for each load point. For the quarter cycle shown, as displacement increases the tensile strain at the bottom of the reinforcing bar increases while the top of the bar is subjected to compressive strain. At the load point 496, the bottom of the reinforcing bar yields in tension. At this point the bond strength between the column bar and the concrete within the joint was apparently exceeded, because there was a sudden increase in the column bar tensile strains through the entire depth of the joint. The slip, or bar pullout, resulting from this increased strain over a substantial length of the column bar caused a significant increase in the rotation of the column region adjacent to the joint. Slippage of beam and column longitudinal bars was detected at several beam-column joints.

Beam Hinge Rotations

Beam hinge rotations were calculated using the data from resistant type displacement transducers that were installed at preselected joints in frames A and B. The variation of the maximum angle of rotation along the height of the building is shown in Figs. 2(a) and 2(b). Table 1 shows the maximum rotation angles at different locations vs. the average drift of building (average drift is defined as the horizontal displacement at roof level divided by the total height of building). Figure 2 and Table 1 indicate that for this structure there is not a clear relationship between average story drift and beam end rotation.

In frame A, the flexural stiffness ratio of the joints along external columns was about 60 percent higher than that of the joints along internal columns. Consequently, the measured beam end rotations at the face of column line 1 were larger than those at column line 2. In frame B, due to the interaction of shear wall and frame elements, the beam end rotations along column line 2 were larger than those along column line 1 in the lower four stories. This trend reversed in the upper stories. The shear wall at the middle bay of frame B greatly affected the behavior of joints. The flexural yielding of the shear wall induced large rotations at the end of the beams directly connected to shear wall in the lower stories of the building. The maximum rotation of 0.026 rad. was measured in the second story level at the face of the shear wall boundary column. This rotation was beyond the ultimate rotational capacity of beam and resulted in crushing of the concrete in the bottom of beam at the beam-column connection.

Table 1- Maximum Beam End Rotations vs. Average story drift

Avg. story drift %	Frame A		Frame B	
	Col. 1	Col. 2	Col. 1	Col. 2
0.14	0.0007	0.0004	0.0011	0.0097
0.30	0.0013	0.0012	0.0024	0.0018
0.77	0.0055	0.0030	0.0011	0.0217
0.84	0.0060	0.0044	0.0070	0.0110
1.03	0.0085	0.0042	0.0143	0.0260

The Effect of Shear Wall Uplift

Shear wall essentially acts as a cantilever beam, resisting bending moment by the coupled compression force in concrete and tension force in the longitudinal bars. The yielding of longitudinal bars under tensile force causes elongation of the tension side of shear wall. During the test, about 2.5 cm (1 inch) uplift at the first story level was measured at the maximum lateral displacement. On the other hand, the shortening of the compression side of shear wall is limited by the closure of cracks. This unsymmetric deformation will induce additional rotation in the beams directly connected to shear wall. Figure 3 shows the maximum beam hinge rotations vs. the peak lateral displacements at the roof level for the beams in the fifth story level of frames A and B. When displacement was in the positive direction, the side of the shear wall connected to the beam in frame B elongated under tensile force and increased the rotation at the beam's end. During the next half cycle when displacement was in the negative direction, both beams had almost the same vertical displacement at two ends. Consequently, rotation angles in this range were very close.

Hysteresis Behavior of Beam Hinges

The plot of the beam hinge rotation vs. the total base shear is shown in Fig. 4 for the interior beam at the second story level of frame A. Almost all of the beams in the building showed the same characteristic behavior. The rotation angle varied linearly until the first major yielding occured. In the next cycles, despite slippage of bars through the joint and a reduction of stiffness at joints, the overall behavior showed only a slight pinching effect and the hysteretic loops remained stable. Stable hysteretic curves were also observed at the beams connected to the shear wall that were subjected to the largest rotations and suffered severe cracking and crushing. The stability of the hysteresis loops can be attributed to the fact that the ordinate of the plot (Fig. 4) was the total base shear. Although some of the beam-column joints would be expected to have a significant reduction in rotational stiffness, the overall building stiffness, which was dominated by the shear wall in frame B, was not significantly reduced.

SHEAR WALL

A reinforced concrete frame-wall structural system resists lateral loads by the interaction of wall and frame components. Shear forces and bending moments resulting from the seismic lateral loads are distributed between walls and frames according to their relative stiffnesses. Under cyclic reversing loads the strength and stiffness of different components will change depending on the level of flexural and shear damage in structure. Consequently, the proportion of the load carried by wall and frame will change continuously during large amplitude cycles of reversing load.

The shear wall in frame B of the test building provided very high stiffness against lateral loading and dominated the response of structure throughout the test. In order to obtain a detailed record of the behavior of shear wall under lateral loads, several displacement transducers were installed on the wall panel and on the boundary columns. Flexural deformation of the wall was measured at three layers by pulley type displacement transducers and two diagonal LVDTs measured shear deformation of wall panel.

A plot of the measured flexural rotation of the wall vs. total base shear (Fig. 5) indicates that the shear wall behaved as a linearly elastic member until longitudinal reinforcing bars in the boundary columns yielded in tension at an average drift of 0.3 percent. After yielding of longitudinal bars the shear wall underwent large plastic deformation which caused the yielding of frame elements of the building. Most of the beams in the lower story levels of frames A and B yielded before the average drift reached 1 percent. The shear deformation of the wall demonstrates the same characteristics as the flexural deformation hysteretic curve. After the first cycle of plastic deformations, stiffness reduced and the hysteretic loops become slightly pinched.

Estimation of Wall Bending Moment

In order to understand the effect of yielding of the wall and the mechanism of force redistribution, the forces in individual resisting elements should be known. The amount of bending moment at the base of the shear wall was calculated by using the measured vertical deformation at the base of the shear wall and assumed hysteretic rules for concrete and reinforcing bars.

The vertical displacement along the base of the wall was measured by five LVDTs. The gage length of these LVDTs varied from 5 to 12 percent of the overall width of the wall. The average strain at the base of wall was defined as the vertical deformation at the base divided by the gage length of the corresponding LVDT. The distribution of the measured average strain along the base of wall verified that the assumption of a linear variation of strain through the cross-section is not valid for reinforced concrete walls after cyclic load reversals beyond the cracking limit. In the analysis, the value of the strain in the interval between LVDTs was approximated by linear interpolation.

For analysis, the total length of the shear wall was divided into several layers and the strain at each layer of concrete and reinforcing bar was calculated by using the measured average strain. The axial stress at concrete layers was calculated according to a hysteretic stress-strain relation for concrete under uniaxial loading [2]. The stress-strain relation of reinforcing bars was idealized by an isotropic hardening model.

By using the measured average strain values and hysteretic rules, the axial force and bending moment in the wall were calculated. The time history of the calculated axial force shows that large axial compression forces, in addition to the gravity loads, are imposed on the shear wall under lateral loading. An important source of the calculated excess compression force is the resistance of the transverse beams to the vertical deformation of shear wall. The uplift of the tension edge of the wall creates large bending moments in the transverse beams connected to the wall. The measured rotations at both ends of transverse beams during the test were close to the theoretical yield rotation. The shear force associated with these moments increased the axial load on the wall.

Figure 6 shows the time history of the lateral displacement at the first story level and the calculated percentage of the total over-turning moment carried by the wall. The magnitude of the calculated wall bending moment will change slightly if different methods are used to model the strain distribution and hysteretic behavior of concrete and reinforcing steel. The results from different models used for this study indicates that the shear wall carries between 35 and 50 percent of the total over-turning moment. This proportion does not change appreciably during large amplitude cycles.

The calculated peak bending moment in the wall and the associated axial force were plotted on the axial force vs. bending moment diagram for the wall (Fig. 7) and they were compared to the theoretical failure and yield envelopes. Distribution of the points representing loading condition at the peak lateral displacement of structure indicates that the shear wall was not forced very much beyond the theoretical yield limit. After yielding of the wall, the proportion of the flexual stiffness of wall and frame elements changed and more load was transfered to frame elements. On the other hand, the resistance of the transverse beam to the uplift of the tension edge of the wall increased axial force and flexural strength of wall.

CONCLUSIONS

The results from the simulated earthquake test of a seven story reinforced concrete building were studied. In beam to column connections, the lack of adequate confinement reinforcement resulted in the slippage of the longitudinal bars through joints. Despite bar pullout and severe damage at the beam hinge zone of individual members, the overall response of the building to the applied forces was satisfactory. The shear wall showed flexural type behavior and resisted a large portion of the applied load throughout the test. The reduction of shear and flexural stiffnesss of the shear wall after yielding were compensated by the participation of frame elements in resisting the applied loads.

REFRENCES

1) Unpublished Draft Recommendations, ACI-ASCE Committee 352, March 1983.

2) Suharwardy, M.I.H., and Pecknold, D.A., "Inelastic Response of Reinforced Concrete Columns Subjected to Two Dimensional Earthquake Motion," Civil Engineering Studies, Structural Research Series No. 455, University of Illinois, Urbana, October 1978.

FIGURES

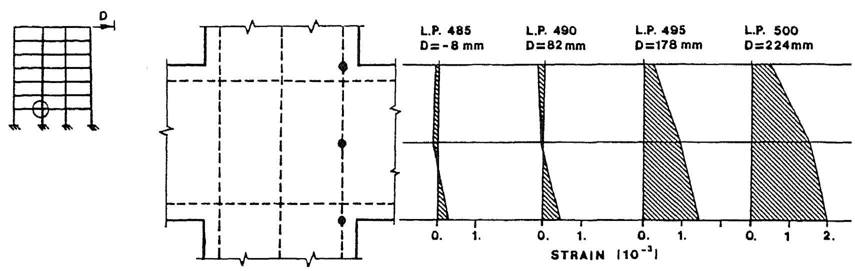

Fig. 1- Distribution of Strain Along Column Longitudinal Bar.

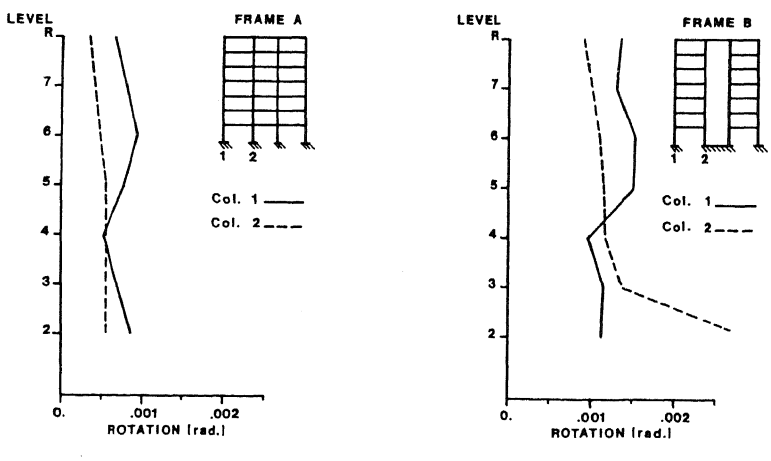

Fig. 2- Maximum Beam Hinge Rotations vs. Height of Building

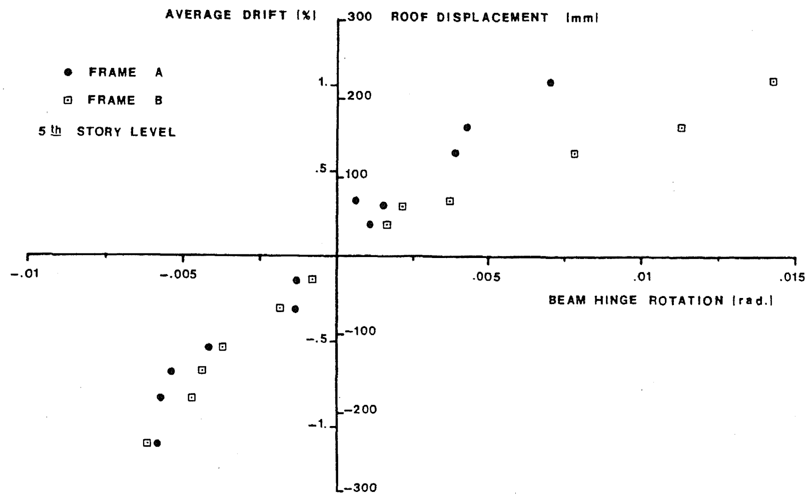

Fig. 3- Comparison of The Beam Hinge Rotations in Frames A and B.

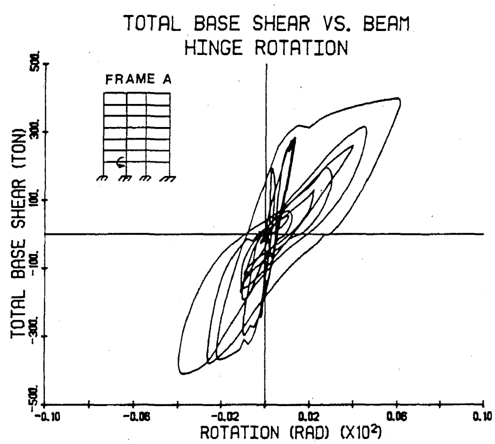

Fig 4- Total Base Shear vs. Beam Hinge Rotation

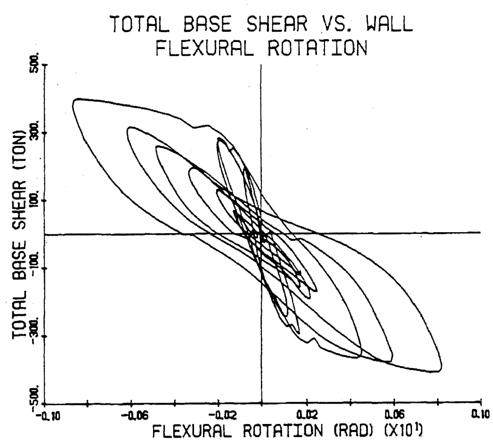

Fig. 5- Total Base Shear vs. Flexural Rotation of Shear Wall

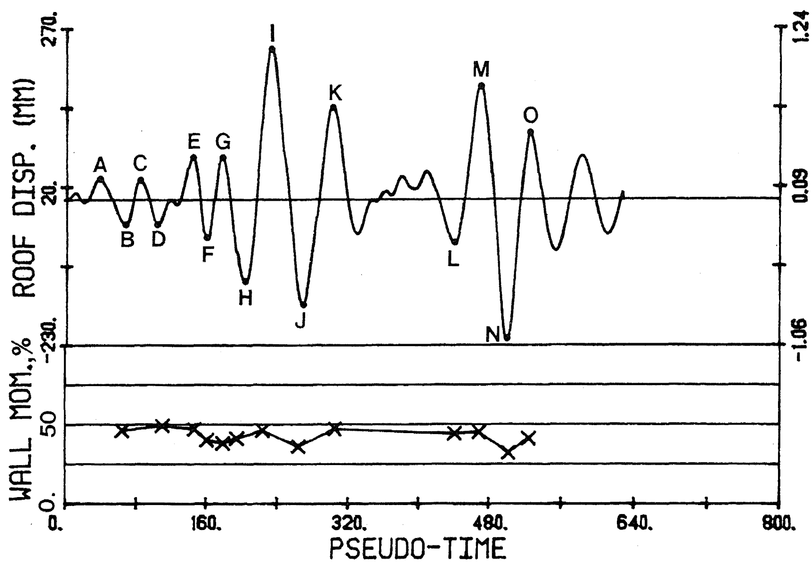

Fig. 6- Lateral Displacement at the Roof level and the Calculated Percentage of the Total Over-turning Moment Carried by Shear Wall.

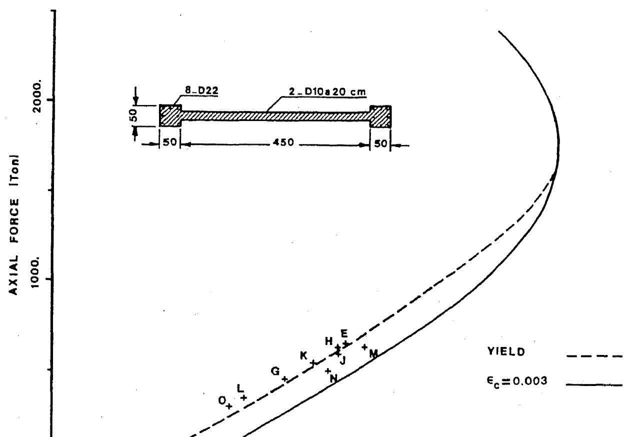

Fig. 7- Axial Force and Bending Moment on the Shear Wall.