EARTHQUAKE TESTS OF SHEAR WALL-FRAME STRUCTURES TO FAILURE

by

Shunsuke Otani(I)

SYNOPSIS

Behavior of multistory reinforced concrete structure was studied through dynamic tests of model structures to a series of intense base motions on the University of Illinois Earthquake Simulator platform. This paper describes the observed behavior of shear wall-frame structures and frame structures to failure.

INTRODUCTION

An important aspect of earthquake simulator tests is to provide reliable data about the behavior of a simple structural model to failure under relatively realistic earthquake motion. The information is important in testing the validity of design methods and the reliability of analysis methods. Common to most of structural analysis and testing, the applicability of the results is limited within a domain defined by the assumptions of the test. The purpose of the test was to study the effectiveness of a structural wall during an earthquake with an intensity much higher than that considered in design.

DESCRIPTION OF TEST

Two types of small scale structures were tested on the University of Illinois Earthquake Simulator (1): one type (F1 and F2) consisting of a pair of identically designed three-story one-bay reinforced concrete frames, and the other type (W1 and W2) consisting of a pair of the above frames and a three-story slender structural wall. Two structures of each type were subjected to a series of scaled earthquake motions in one horizontal direction. An effort was made to design a test structure in accordance with the assumptions generally used in a design analysis. Scale factors were chosen to be $1/2.5$, $1/(2.5)^2$, and $1/(2.5)^4$ for time, line and force, respectively.

The constituent structures were connected, at each beam level, to rigid steel weights (1,970 lb/floor) through mechanical hinges. The dimension of a frame and wall, and the arrangement of reinforcement are shown in Fig.1. Small aggregate concrete was used. The average compressive strengths were 5,000 psi for the frame concrete, and 6,800 psi for the wall concrete.

Number 2 deformed bars (yield stress f_y = 43 ksi, ultimate strength f_{su} = 66 ksi, and area A_s = 0.049 in^2), and number 14 gage wires (f_y = 28 ksi, f_{su} = 47 ksi, and A_s = 0.0050 in^2) were used as longitudinal and lateral reinforcement in the frame. Number 8 gage wires (f_y = 44 ksi, f_{su} = 54 ksi, and A_s = 0.021 in^2) were used in the wall. The longitudinal reinforcement was provided with anchorage length sufficient to

---

(I) Department of Civil Engineering, University of Toronto, Ontario, Canada.

develop its tensile strength. All members and connections were provided with shear reinforcement sufficient to prevent diagonal shear failure.

The NS component of the 1940 "El Centro" record or the N21E component of the 1952 "Taft" record was used in each test run. The time axis was adjusted in accordance with the time scale of the test structure. The input earthquake records were closely reproduced on the earthquake simulator within a range of structural engineering interest. The intensity of the base motion was doubled after each test run until the structure failed (Table 1). The computed elastic base shears, during the first run of test structures F1, W1 and W2, were approximately 2.5 to 3.5 times the observed yielding base shears. Therefore, the intensity of base motion at the first test run of the three tests was roughly comparable to the intensity of a design earthquake.

OBSERVED BEHAVIOR

In spite of severe damage, the test structures withstood, without "collapse" a series of intense base motions. The observed maximum deflections at the first floor level exceeded one-twentieth of the story height. The shear wall-frame structure (W2) collapsed when the base motion at the third test run was repeated at the same intensity. A shear wall alone might not have been sufficient to prevent a structure from collapsing.

Wide cracks were observed, in the frame structure, at the base of the first story columns (accompanying crushing of the concrete), at the top of the second and third story columns, at the ends of the first and second level beams. Yielding at these locations was sufficient to form a "collapse mechanism" of an elasto-plastic frame structure. Even after reaching this damage stage, the structure was able to respond to the intense base motion in a stable manner.

In the shear wall, wide flexural cracks formed in the first and second stories. Crushing of the concrete at the base was followed by "sliding shear failure" along the base. It is highly conceivable that such "sliding shear failure" might not have occurred, were the shear wall supported on a flexible foundation.

Effect of Previous Loading. A structure was subjected to one to three runs of intense base motions, during each test run the structure was deformed beyond yielding. The behavior of the test structure might be affected seriously by the damage in the previous test runs. To study the effect, observed waveforms of frame structure F1 during the third test run were compared with that of test structure F2 during the first test run. The two structures were constructed under the same specification. The intensities of base motion were comparable (Table 1). Test structure F1 had already been damaged well beyond yielding from the previous two test runs.

It is interesting to note that the acceleration waveforms from the two test runs were virtually identical (Fig.2). The previous test runs at lower intensities did not effect the behavior measurably. This observation may be limited to the case, in which (a) the intensity of base motion was increased significantly from the last test run, (b) the major response occurred at relatively early part of the excitation, and (c) the structural stiffness did not deteriorate significantly with a number of cycles of load reversals.

Effectiveness of Wall. It is also interesting to observe a close similarity between the displacement waveforms in the third test runs of frame structure F1 and shear wall-frame structure W1 (Fig.3). The intensities of base motion were 50% higher in the shear wall-frame test than in the frame test (Table 1). An extensive flexural damage was already observed at the base of the wall from the previous two test runs. However, such a close similarity was not observed between the waveforms of the two test structures prior to the third test run.

When the test structure oscillated in the same phase at the three mass levels, flexural yielding occurred at the base of the wall, followed by crushing of the concrete. Consequently, the concrete area effective to resist shear and bending moment was reduced considerably, and the wall became very flexible at its base. The frames had to resist major earthquake forces. Therefore, the shear wall-frame structure behaved in a manner similar to the frame structure.

Although the wall lost its stiffness at the base, the wall was effective in resisting forces and stiffening the upper stories. The base shear waveforms observed in test W1 was clearly larger than that in test F1 even after the severe damage at the base (Fig.4).

BASE SHEAR AND OVERTURNING MOMENT

Various building codes, for example (2), require a column to have web reinforcement sufficient to develop the possible maximum shear, determined from the moment capacities at the ends of the column. The possible maximum base shear of a structure is the sum of these maximum shears of all first-story columns. The maximum base shears observed in the frame tests exceeded this base shear (Fig.5). The reason for this is not clear at present. However, the test results support the necessity of the code provision about web reinforcement. The observation is not limited to extraordinarily high intensity base motions, since the intensity at the first test run of test F1 was comparable to that of a design earthquake.

On the other hand, the base overturning moment in most design codes, for example (3), is based on the elastic analysis with earthquake forces reduced by a certain factor, relying on inelastic energy dissipation through ductile behavior of a structure. Actual overturning moment may exceed the design value during a design earthquake because yielding would take place in various locations of the structure.

The possible maximum base overturning moment of a shear wall-frame structure can be computed for a mechanism shown in Fig.6. The maximum overturning moments observed in the tests exceeded the possible maximum value (Fig.7). If the earthquake resistant design relies on inelastic behavior, the code should require a structure to be provided with a ductile mechanism to resist the maximum possible overturning moment. In other words, the exterior columns must be able to resist additional axial forces due to overturning effect without brittle compressive failure of the concrete or tensile fracture of the reinforcement.

CONCLUSIONS

The following conclusions may be drawn for the work reported: (a) The reinforced concrete test structures, designed carefully to prevent diagonal shear and anchorage failure, withstood a series of intense base motions without collapse. A structural wall alone might not have been sufficient to prevent the collapse. (b) A structural wall, after having a severe damage, became flexible at its base. However, the wall was effective stiffening the upper stories. (c) A design consideration must be given to provide a structure with a ductile mechanism to resist the possible maximum overturning moment.

ACKNOWLEDGEMENT

The reported work was carried out at the Structural Research Laboratory of the University of Illinois at Urbana-Champaign under U.S. National Science Foundation Grant GI-29934. Acknowledgement is due to Professors M.A. Sozen and V.J. McDonald, University of Illinois, for their advice and assistance.

BIBLIOGRAPHY

(1) Sozen, M.A. and Otani, S., "Performance of the University of Illinois Earthquake Simulator in Reproducing Scaled Earthquake Motions", Proceedings, U.S.-Japan Seminar on Earthquake Engineering with Emphasis on the Safety of School Buildings, Sendai, September, 1970, pp. 278-302.

(2) ACI Committee 318, "ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318-71)", American Concrete Institute, Detroit, Michigan, 1970.

(3) International Conference of Building Officials, "Uniform Building Code", Wittier, California.

TABLE 1: SUMMARY OF TESTS

Test Run	Spectrum Intensity (SI_{20}), inch	Max. Acc. g	Base Motion Type
F1-1	4.2	0.24	El Centro
F1-2	8.5	0.43	"
F1-3	16.5	0.91	"
F2-1	18.1	1.07	El Centro
W1-1	6.2	0.54	El Centro
W1-2	12.0	1.02	"
W1-3	24.0	3.78	"
W2-1	7.8	0.68	Taft
W2-2	15.2	1.54	"
W2-3	33.7	5.26	"

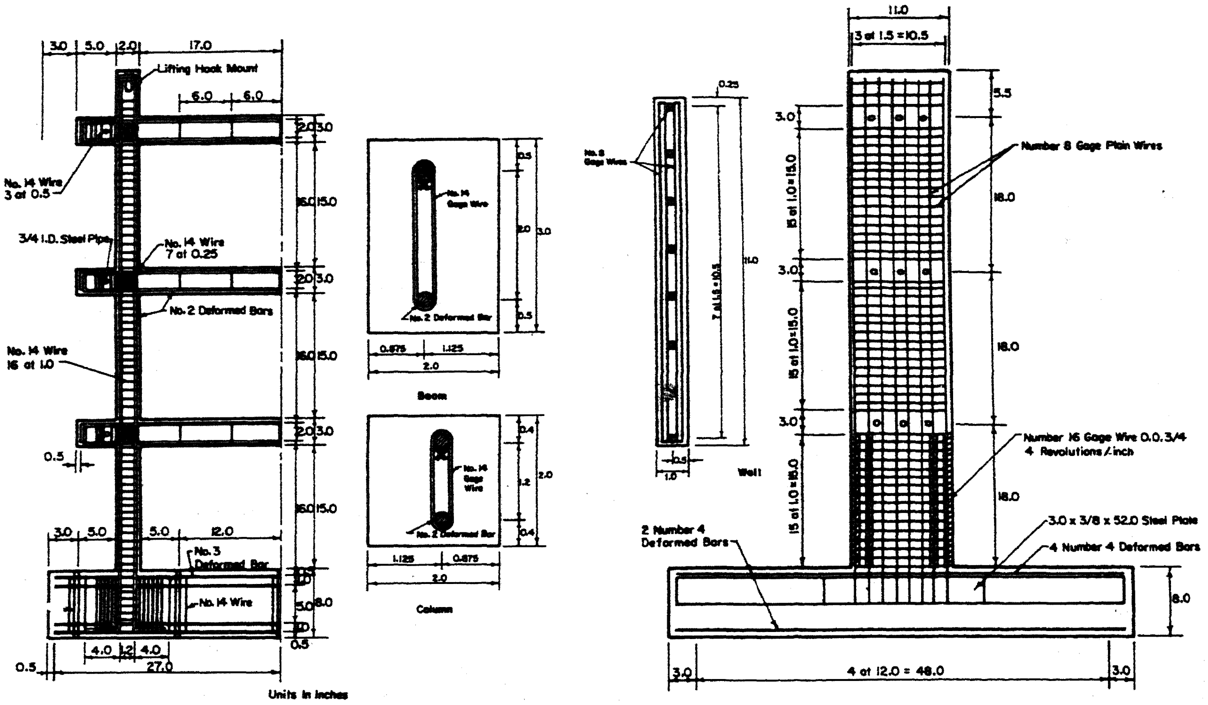
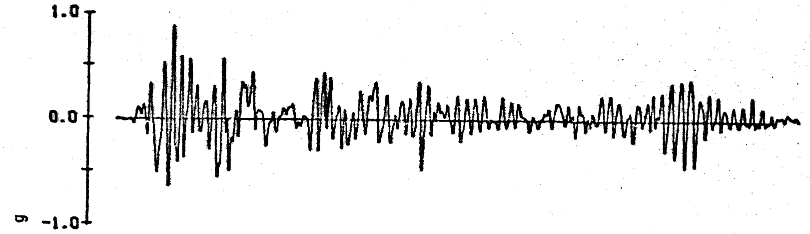
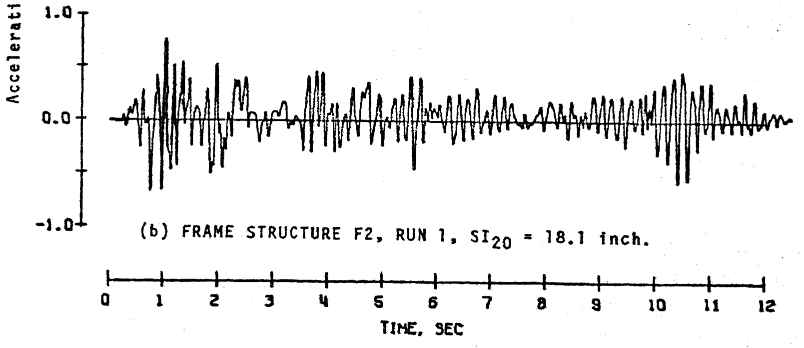


FIG. 1 Dimensions and Reinforcement of Test Structure



(a) FRAME STRUCTURE F1, RUN 3, SI_{20} = 16.5 inch.



(b) FRAME STRUCTURE F2, RUN 1, SI_{20} = 18.1 inch.

FIG. 2 Effect of Previous Loading on First Level Acceleration Waveforms

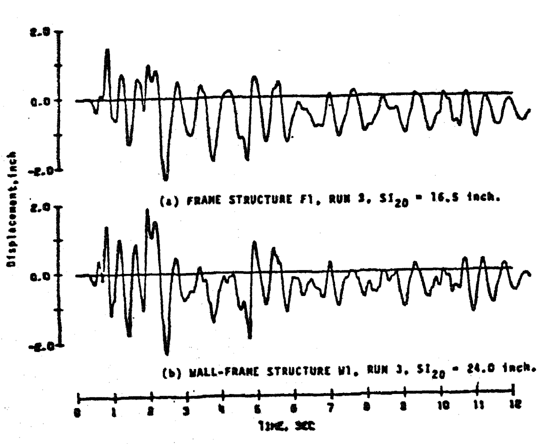


FIG. 3 Third Level Displacement after Damage

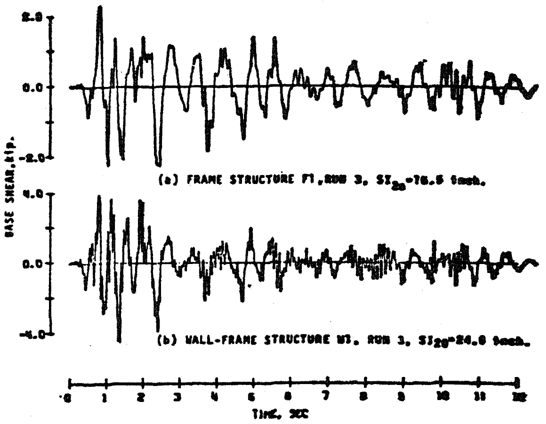


FIG. 4 Base Shear after Damage

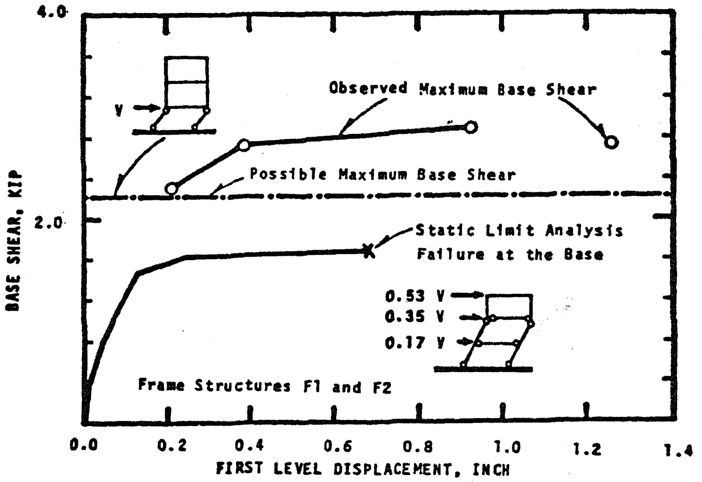
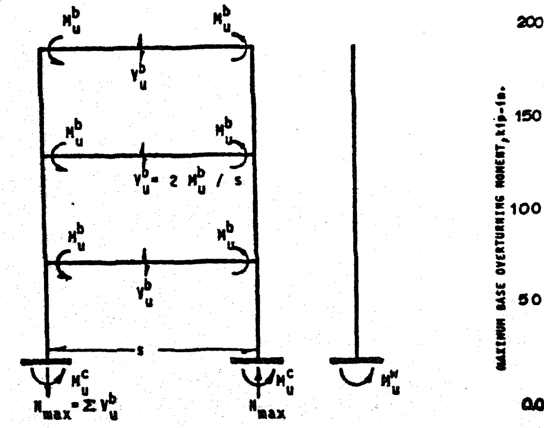


FIG. 5 Observed Maximum Base Shear Compared with Possible Maximum Value



Possible Max. OTM = $\Sigma M_u^c + \Sigma M_u^w + \Sigma N_{max} \cdot s$

FIG. 6 Possible Maximum Base Overturning Moment

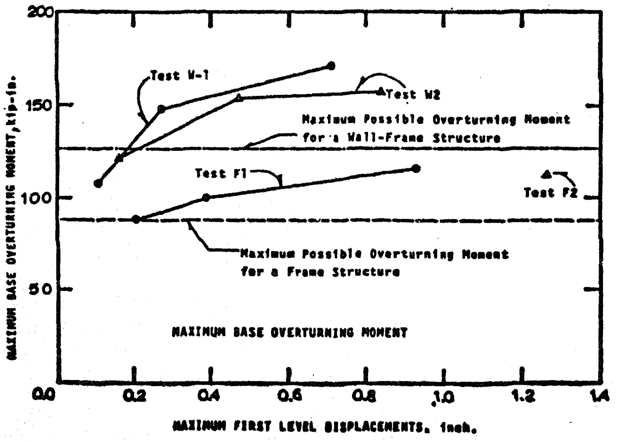


FIG. 7 Observed Maximum Base Moment Compared with Possible Maximum Value

DISCUSSION

Anil Kumar (India)

The bending moment diagram of the frame shown indicates that the frame is behaving like a shear beam near failure. Does it imply that there was no significant damage to the beams of the frame?

Author's Closure

With regard to the question of Mr. Anil Kumar, we wish to state that no bending moment diagram, which the discussor refers to, is present in the paper. The behaviour of the test structures was not like the one observed in a "shear-beam" structure. Extensive yielding and wide flexural cracks took place at the ends of all three-level beams. Flexural cracks distributed throughout the beam for both frame and frame-wall test structures.