U.S.-JAPAN COOPERATIVE RESEARCH PROGRAM ON LARGE-SCALE TESTING. PREDICTION OF PROTOTYPE RESPONSE FROM SMALL-SCALE MODEL TESTS

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SUMMARY

This paper presents the results of tests conducted on small-scale reinforced concrete models. Beam-column assemblies, an isolated shear wall, and a shear wall--frame unit were constructed to simulate portions of the full-scale test building at a scale of 1:12.5. Test results are used to determine the separate contributions of the wall, in-plane frame action, and transverse frame action in resisting lateral loads. These test results are then combined analytically to predict the response of the complete structure.

INTRODUCTION

The purpose of the U.S.-Japan cooperative research program is to assess experimental methods in earthquake engineering research and to gain insight into the seismic behavior of a representative structure and its components. The program consisted of the full-size structure test and a series of support tests on components and assemblies, ranging from full size tests to small-scale model tests. This paper reports on small-scale (1:12.5 scale) model tests of beam-column assemblies, a shear wall, and a shear wall--frame unit.

The objectives of this model study are (1) to investigate the feasibility and limitations of small-scale model testing in earthquake engineering, (2) to study the simulation accuracy of specific failure modes in small-scale models, and (3) to correlate results of tests at different scales to assess prototype response prediction from experimental studies. In order to fulfill these objectives, all model test specimens were made to be "exact" replicas of the prototype tested in Japan or of large-scale components tested by others.

TEST SPECIMENS AND LOADING PROGRAMS

Model Materials

Microconcrete was used in constructing the test specimens. The mix used consisted of Type III (high early strength) Portland cement, graded sand aggregate, and water mixed in the proportions of 1.0:3.5:0.75 by weight. A typical stress-strain diagram of the microconcrete is shown in Fig. 1 along with stress-strain results from the concrete used in the prototype structure.

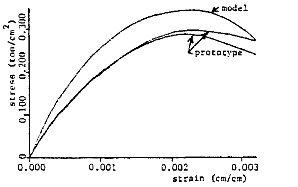

Fig. 1 Concrete Stress-Strain Diagrams

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Model reinforcing was manufactured from AISI 1060 steel wire. This wire was packed in a steel pipe and normalized to lower its strength from the hard-drawn condition. The wire was then cold rolled between two grooved rolls to form protruding ribs on the bars similar to those on prototype reinforcement. The deformed wires were then normalized by heating in a 900°C oven for 30 minutes and air cooling to obtain the desired stress-strain properties of sharp yielding, level yield plateau, and strain hardening similar to prototype bars. Stress-strain diagrams of the model #6 (19mm) bar and the corresponding prototype reinforcing bar are shown in Fig. 2. A photograph of the model reinforcing bar is included as Fig. 3.

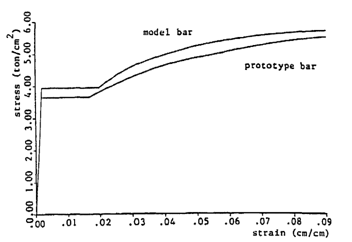


Fig. 2 Reinforcing Sress-Strain Diagrams

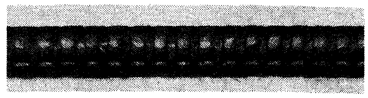


Fig. 3 Model Reinforcing Bar

Beam-Column Assembly Tests

Four 1:12.5 scale models of beam-column assemblies were fabricated. Two were exact replicas of prototype assemblies tested at the University of Texas, Austin (Ref. 1). In addition to the models of the Texas assemblies, (one exterior and one interior assembly with floor slab), one model each was tested of identical assemblies without floor slabs. The cross sections of the model columns and beams were 40mm by 40mm and 24mm by 40mm, respectively. The properly scaled reinforcement consisted of 8-#7 bars in the columns and 3-#6 bars (top) plus 2-#6 bars (bottom) in the beams. The layout of reinforcement for the interior assembly with floor slab is shown in Fig. 4.

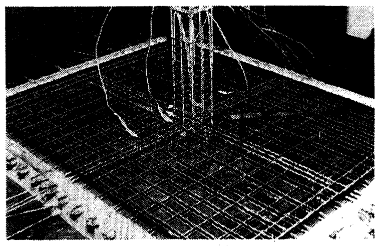
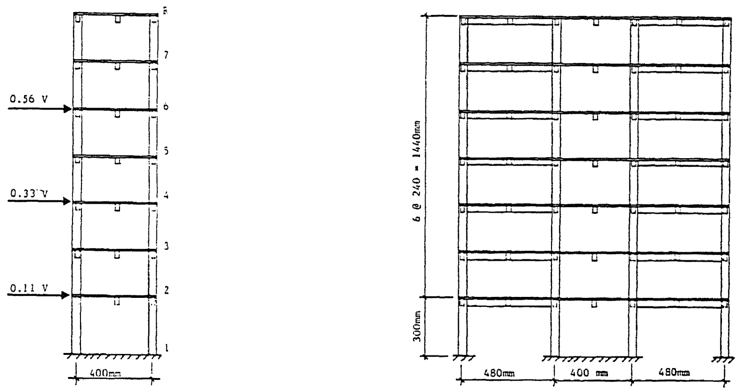


Fig. 4 Beam-Column Assembly Reinforcment

The assemblies, which were of a cruciform shape, were mounted in a test frame that restrained the column top horizontally and the column base in both horizontal and vertical directions. The beam-column assemblies were loaded with weights to simulate the dead load effects of the prototype. Hand operated screw jacks were used to apply vertical loads at the ends of each beam. Beam end loads were applied to reproduce the beam end displacement histories used in the full-scale assembly tests reported in Ref. 1. These displacement histories consisted of displacing one beam upward and the other beam downward in cycles of increasing magnitude. Typically, cycles of each magnitude were repeated three times.

Shear Wall and Shear Wall--Frame Tests

The shear wall and shear wall--frame specimens are shown in Fig. 5. The shear wall specimen is a replica of the central shear wall in the prototype test structure with properly scaled reinforcement and details. Two meter widths of the floor slabs on each side of the wall were included in the model specimen.



Shear Wall Model   Shear Wall--Frame Model

Fig. 5 Dimensions of Shear Wall and Shear Wall--Frame Models

The shear wall--frame specimen included the central shear wall as well as all frame elements of the central unit of the prototype structure. In this specimen, 4m of the slab and transverse beams of the prototype were included on both sides of the wall. Three-dimensional action was simulated in this specimen by restraining the ends of the transverse beams through vertical linkages attached to the base of the structure. These links can be seen in Fig. 6 which shows a photo of the shear wall--frame specimen with lead weights placed at each floor level to simulate gravity effects.

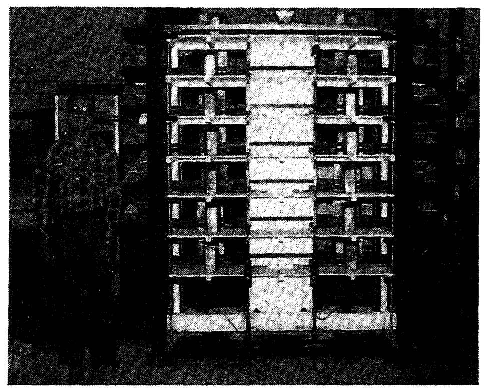


Fig. 6 Shear Wall--Frame Specimen

A whiffletree was used to simulate the lateral loading of the prototype structure. The story loads were adjusted to account for the absence of the exterior frames in the model specimens. This adjustment was achieved by predicting the story shears resisted by these frames and subtracting the corresponding story forces from the triangular load distribution for the full structure. This process resulted in the approximate lateral load distribution shown in Fig. 5.

Both the wall and the wall--frame specimens were loaded with the pattern established in this manner. In both specimens the scaled roof displacements of the prototype tests PSD-1 thru PSD-4 were used to control the loading histories.

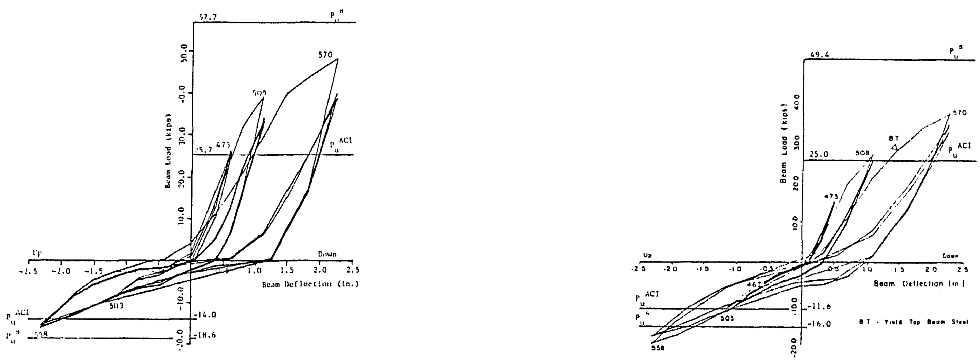
The instrumentation system consisted of standard and custom made measurement devices which were used to obtain continuous records of loads, horizontal and vertical story displacements, beam rotations, and strains in reinforcing bars.

TEST RESULTS

Beam-Column Assembly Tests

Except for elastic cycles, displacement control was used to simulate the prototype loading history used at the University of Texas. This history consisted of 5 segments (Test 1 thru Test 5) of increasing magnitude.

The load-displacement results from Test 4 of both the model and prototype interior assembly with slab are presented in Fig. 7. It is evident from this figure that strength, stiffness, and hysteresis loop properties are similar in the prototype and model domains. In the "positive" loading direction the model beam exceeds the strength of the prototype beam. This was due to the higher yield strength in the model slab steel than in the prototype slab steel (74.5 KSI vs 58 KSI).



(a) Model Assembly

(b) Prototype Assembly

Fig. 7 Load-Deflection Response of Interior Beam-Column Assembly

A more direct comparison of model and prototype behavior is accomplished by comparing each specimen with the predicted ultimate load values indicated in Fig. 7. Two predicted values are shown, one based on the ACI approach using a small "effective" slab width (P_u^{ACI}) and one based on assuming the full slab effective (P_u^s). Both model and prototype results indicate that the ACI approach severely underestimates the contribution of the slab and that in fact most of the slab width is effective in resisting moment.

Both model and prototype results exhibit severe pinching of the hysteresis loops. This was due to severe bond deterioration along the beam bars passing through the joint and was verified in the model study by a post-test examination. In both the model and prototype this bond deterioration is of similar severity since the pinching of the loops is comparable.

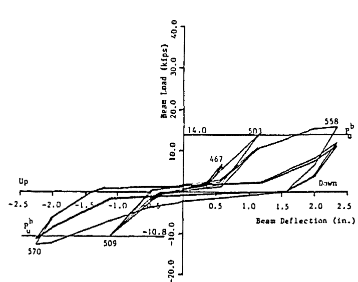


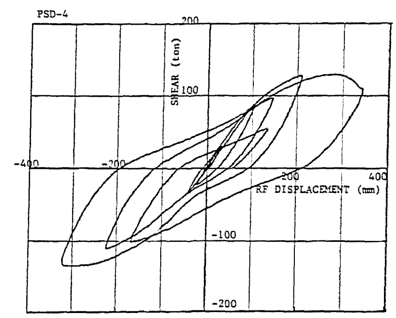
Fig. 8 Load-Deflection Response Model Beam Without Slab

The effect of the slab on beam behavior can be assessed by comparing Fig. 7(a) with Fig. 8. The latter figure shows the response of the interior assembly without floor slab subjected to the same deflection history. Not only is there a drastic difference in strength, but also a large difference in hysteresis loop shapes. Bond deterioration was much more severe in the specimen without slab, leading to severe pinching of the loops.

Shear Wall Test

The shear wall specimen was tested following the roof displacements of the prototype test structure during tests PSD-1 thru PSD-4. During test PSD-2 the first cracks were observed in the wall specimen. These cracks initiated as horizontal cracks in the boundary elements and then propagated down a diagonal path across the wall in later load cycles. During test PSD-3, these diagonal cracks spread throughout the wall below level 3 and the maximum load of 142 tons was observed. This load was only about one-third of the maximum base shear obtained in the complete prototype structure.

During test PSD-4, existing cracks lengthened slightly and two small cracks initiated above level 3. The crack at the base of the wall opened much more than in previous tests and horizontal sliding was observed. During the last cycle of test PSD-4, two of the boundary element bars fractured, causing the rounding of the last loop in the load-deflection plot in Fig. 9. At these large displacement cycles, the wall was observed to rotate about the base of the compression boundary element. Although some crushing of the boundary element was observed, this element was available to assist the wall in carrying both shear and axial forces.



Shear Wall Specimen
Fig. 9 Load--Roof Deflection Response

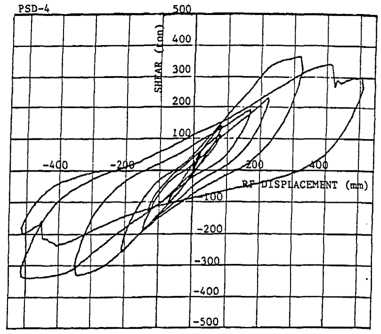
The maximum shear force in this specimen (142 t) was much smaller than the calculated shear capacity of the wall (approximately 310 t). The fracture of the boundary element bars and the large opening of the horizontal crack at the base show clearly that the wall failed in a purely flexural mode. The observed sliding at the base was a consequence of flexural cracking.

Shear Wall--Frame Test

The shear wall--frame specimen was subjected to the same roof displacement history as the shear wall. During test PSD-2, the first cracks occurred in the wall and boundary elements below level 2. At the same time, one crack across the top of a longitudinal beam was observed at the left boundary element.

Early in test PSD-3, flexural cracks were observed at the bottom of all longitudinal beams. Additional cracks occurred in the shear wall similar to those observed in the isolated wall test. At the top of each floor slab, cracks around the boundary elements were observed in the longitudinal and transverse directions. This demonstrated three-dimensional flexural action in the floor system caused by slabs warping as the boundary element and attached beams raised while the ends of the transverse beams were restrained vertically. During the larger cycles of this test, the bottom of all longitudinal beams suffered crushing where they connected to the boundary elements.

During test PSD-4, the diagonal cracks in the wall changed only slightly and none were observed above level 3. Torsional cracks in the transverse beams and flexural cracks at the column bases were observed at the exterior column lines. At the end of this test, the concrete at the base of the boundary elements started to spall and several of the exposed reinforcing bars buckled between the ties. The crack at the wall base grew steadily during this test and sliding of the wall was noticeable during the last few cycles. The load--roof displacement plot from this test is shown in Fig. 10.



Shear Wall--Frame Specimen
Fig. 10 Load--Roof Deflection Response

After the loading history of test PSD-4 was completed, the specimen was loaded to a roof displacement of 500 mm. During this cycle, each boundary element and the adjacent wall corner just above the base block crushed under the combined shear and axial load as shown in Fig. 11. This caused the sudden reduction in lateral load near the peak displacement in Fig. 11. After this crushing, the wall deformed by sliding on the base block rather than overturning. The results of this test indicate that the wall of this specimen failed in a combined flexure-shear mode.

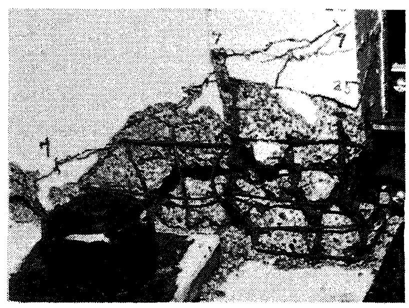


Fig. 11 Shear Wall--Frame Base Crushing

INTERPRETATION OF TEST RESULTS

To analyze the interaction of the shear wall with the surrounding frames, the base shear and the overturning moment are considered separately. The shear wall--frame specimen resisted a total base shear of 361 tons and a base overturning moment of 4494 t-m.

The maximum overturning moment resisted by axial load transferred to the boundary elements by the transverse beam shears (3-D action) was determined from the vertical linkage forces to be 510 t-m. This alone reduces the overturning moment resisted by the wall by 11% without affecting the base shear. From a limit analysis based on the maximum moments from the joint tests scaled to represent an effective slab width of 6 m, the base shear and overturning moment resisted by the frame elements in the plane of the shear wall were 32 tons and 1590 t-m, respectively. Acting alone, this would reduce the overturning moment on the wall by 35% while reducing the base shear by only 9%. The combination of these two effects reduces the overturning moment and base shear applied to the wall by 47% and 9% respectively. Thus, the shear wall of the wall--frame specimen resists an overturning moment of 2390 t-m. This value, as well as the moment value of the isolated wall (1770 t-m), match closely with predictions based on a standard M-P interaction curve (see Fig. 12). The increase in bending resistance of the wall in the wall--frame system, as compared to the isolated wall, can be attributed to the increase in axial compressive load caused by the 3-D action.

To compare model and prototype behavior, the results of the the model test were adjusted to account for differences in material properties and the absence of the two rigid frames. The resulting load-roof displacement plots for model and prototype tests PSD-3 and PSD-4 are given in Fig. 13. Overall, the model and prototype results are in good agreement. The only significant difference is in the unloading stiffnesses which result in fatter hysteresis loops for the model specimens. The peak loads at the large displacement cycles match within 5 %.

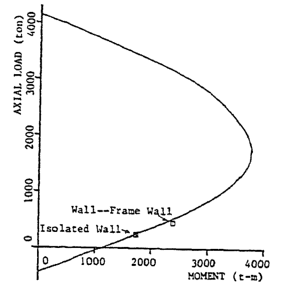
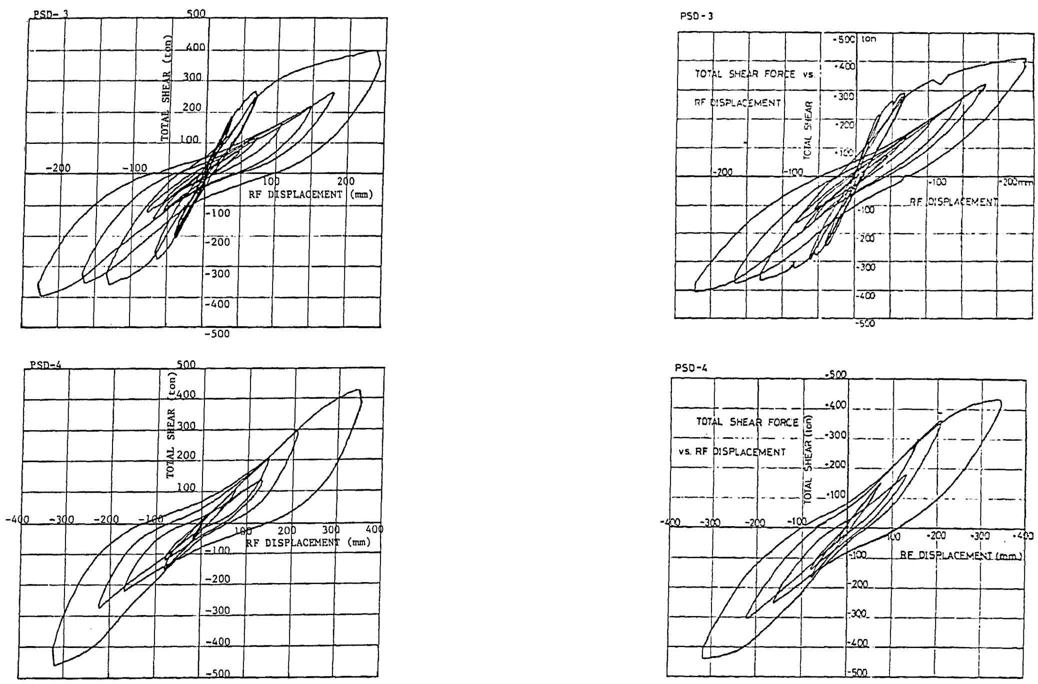


Fig. 12 M-P Interaction Diagram



Model Prediction

Prototype Response

Fig. 13 Load--Roof displacement Response

CONCLUSIONS

1. The experiments discussed in this paper indicate that the overall load--deformation response of components and structures can be reproduced adequately in tests of carefully designed and detailed small-scale models.

2. The beam-column assembly tests have shown that the use of properly deformed model reinforcing bars results in bond deterioration which is similar to that observed in the prototypes. Spalling and crushing of concrete, and buckling of reinforcing bars were observed in the wall--frame specimen at the same locations as in the prototype, except in the shear wall. Cracks in the model specimens occur at higher loads and at wider spacings. This will prohibit the exact simulation of localized failure modes which depend on crack distributions, such as shear failure in a wall element.

3. Floor slabs contribute much more to the bending resistance of beams than is assumed in presently used design procedures.

4. In a shear wall--frame unit, the frames surrounding the shear wall decrease considerably the overturning moment that must be resisted by the wall.

5. The effect of transverse beams framing into the edges of a shear wall is to further decrease the overturning moment in the wall and to increase the bending capacity of the wall. This increase in bending capacity is due to the additional compressive loads transferred to the wall by the shears in the transverse beams.

ACKNOWLEDGEMENTS

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REFERENCES

1. Murray, P.A., "A Study of Beam-Column Joints Under Seismic Loads: Component Tests Versus Building Response", Thesis submitted to the University of Texas at Austin, August 1982.