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Behavior and design of multi-story masonry walls under in-plane seismic loading

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ABSTRACT: Two types of full-scale concrete masonry specimens with openings, reinforced and grouted, were constructed and tested in the laboratory. Each specimen was loaded vertically by constant loads representing gravity loads on the shear walls' tributary areas, and horizontally by quasi-static, reversed in-plane shear loads at the two floor levels. Observed behavior was compared with design assumptions and analytical predictions. Based on the test results and analytical predictions, a general design approach is proposed for coupled and perforated walls of reinforced masonry subjected to seismic loadings. The proposed approach leads to masonry walls with predictable strength and stable load-deflection behavior under many cycles of reversed cyclic load, and is recommended for design of reinforced masonry walls in seismic zones.

1 OVERALL OBJECTIVES

The research described here, identified as Task 3.1(c) of the U.S.-Japan Coordinated Program for Masonry Building Research (TCCMAR) (Noland 1990), is concerned with the in-plane seismic resistance of two-story reinforced concrete masonry walls with openings. The overall objectives of this task were to examine how the in-plane seismic resistance of multistory concrete masonry walls is affected by floor-wall joints, wall openings, and floor elements (Leiva 1991).

2 DESCRIPTION OF SPECIMENS

During this program, 6 full-scale reinforced concrete masonry walls, each two stories high, were designed, constructed, tested, analyzed, and evaluated. All specimens were of fully grouted hollow concrete masonry. Two specimens were single walls with door and window openings, and four specimens were pairs of walls, each coupled by a different floor system, with and without lintels.

The single walls with openings, termed Type 1 specimens, were of the general form shown in Figure 1. They had a base dimension of 16.67 ft (5.08 m), and a uniform story height of 8.67 ft (2.64 m). Their floor slabs were 8 inches thick (200 mm), with a width of 6.47 ft (1.97 m). They were intended to represent walls in a two-story building, perforated by window and door openings.

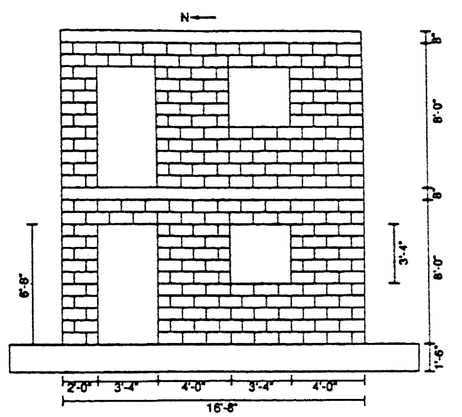

Figure 1. General form of Type 1 specimens

According to the particular design philosophy used, each specimen was intended to show a different response to lateral load excitations. The specific objectives of the Type 1 specimen tests were:

a) to examine the cyclic shear resistance of the perforated wall system which each specimen represented

b) to compare the effectiveness of two different philosophies for designing perforated wall systems, and

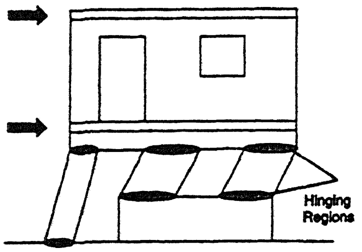

Figure 2. Pier-type collapse mechanism of Specimen 1a

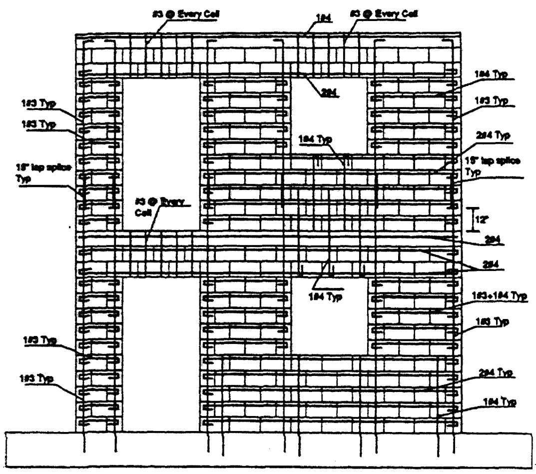

Figure 3. Reinforcement of Specimen 1a

c) to test the analytical models developed in this and in other U.S. TCCMAR tasks.

The two Type 1 Specimens, termed Specimens 1a and 1b, were designed according to a "pier-based" philosophy and a "coupled wall-based" philosophy, respectively. The pier-based specimen was intended to form a collapse mechanism with flexural hinges at the tops and bottoms of the piers (Figure 2). The corresponding reinforcement in Specimen 1a is shown in Figure 3. The coupled wall-based specimen was intended to form a collapse mechanism with a flexural hinge at the base of the wall, and at the ends of the coupling lintels. This mechanism is shown in Figure 4. The corresponding reinforcement for Specimen 1b is shown in Figure 5.

Shear reinforcement in the Type 1 specimens was designed according to a capacity design philosophy. Shear capacities of individual elements, and shear transfer capacity between elements, were designed to exceed the shears consistent with the formation of the design flexural mechanism. In other words, the design

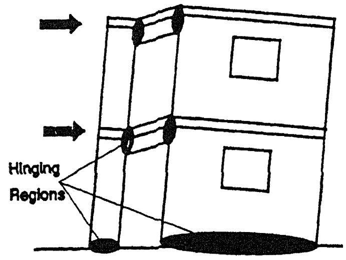

Figure 4. Coupled wall-type collapse mechanism of Specimen 1b

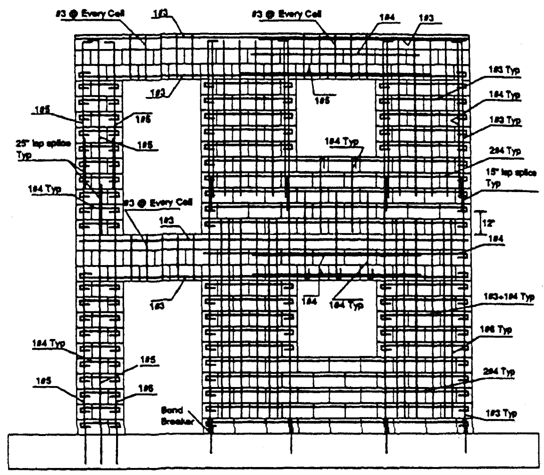

Figure 5. Reinforcement of Specimen 1b

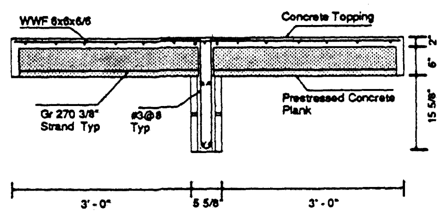

Figure 6. Floor system used for Type 1 specimens

shears were based on the flexural capacities of the hinging elements, rather than on the design loads themselves. Type 1 specimens generally had flexural reinforcement ratios of about 0.1%, and shear reinforcement ratios approaching 1%.

As shown in Figure 6, the Type 1 specimens had a floor system consisting of precast, prestressed planks 6 inches (150 mm) thick, covered by a cast-in-place concrete topping 2 inches (50 mm) thick. The topping was reinforced with welded wire fabric.

The four coupled wall specimens, termed Specimens 2a through 2d, were of the general form shown in Figure 7 (Leiva 1990a, Leiva 1991). They were intended to represent coupled walls in a two-story building. Each specimen had a different combination of floor and coupling systems. They had a total base dimension of 16.67 ft (5.08 m), consisting of two walls 6 ft (1.83 m), coupled by a slab spanning 4.67 ft (1.42 m). Their floor slabs were 8 inches thick (200 mm), with a width of 6.47 ft (1.97 m). The different floor systems are shown in Figure 8. The specific objectives of the Type 2 specimen tests were:

a) *to examine the cyclic shear resistance of the* coupled wall system which each specimen represented

b) to examine the shear strength and in-plane response of the floor-wall joints

c) to examine the coupling effectiveness (under reversed cyclic loads) of plank floor systems, with and without masonry lintels, and

d) to test the analytical models developed in this and in other TCCMAR tasks

As before, a capacity design philosophy was followed in designing the shear reinforcement. Reinforcement for Specimens 2a and 2b, which was typical, is shown in Figure 9. Type 2 specimens all had flexural reinforcement ratios of about 0.25%, and shear reinforcement ratios of about 0.44%.

3 MATERIALS

All specimens were of fully grouted concrete block masonry with deformed reinforcing bars conforming to ASTM A615, Grade 60. Tests were conducted on all materials: reinforcement;

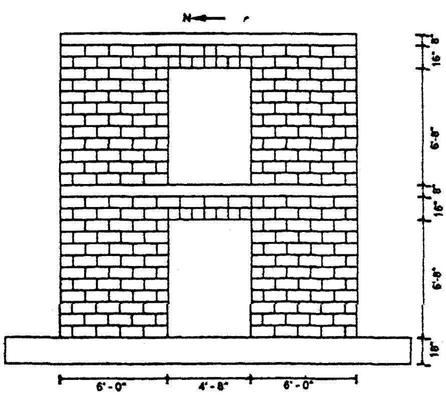

Figure 7. General form of Type 2 specimens

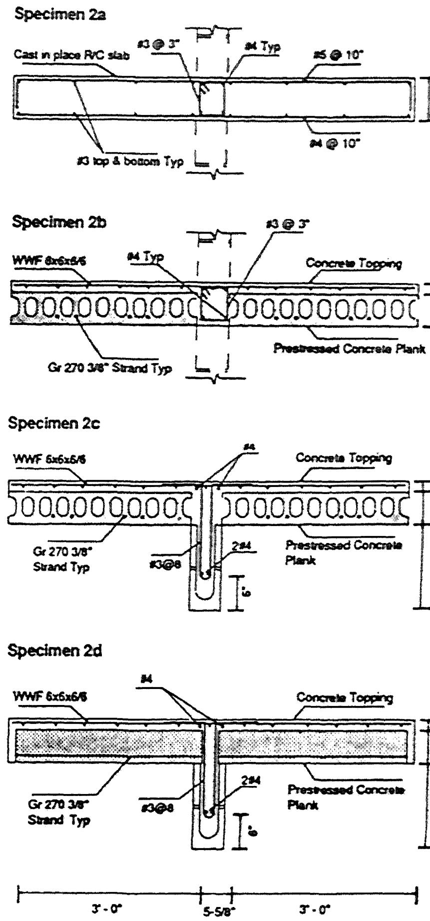

Figure 8. Different floor systems used for Type 2 specimens

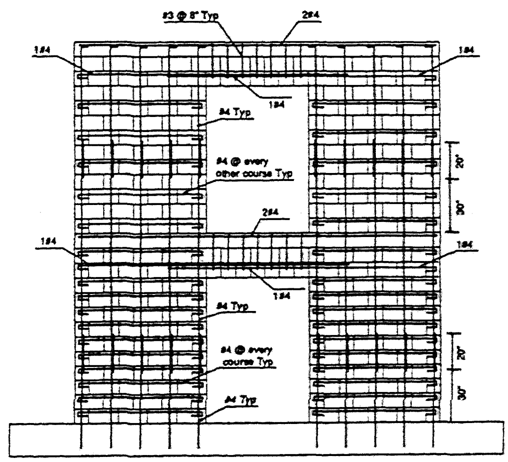

Figure 9. Typical reinforcement for Type 2 coupled wall specimens

units; mortar; grout; and prisms (Leiva 1990a). Compressive strength of grouted concrete masonry prisms averaged 2800 psi (19.3 MPa). Yield strength of reinforcement averaged 69 ksi (475 MPa), and the average ultimate strength was 107 ksi (737 MPa).

4 TEST SETUP, INSTRUMENTATION, AND LOADING HISTORY

Each specimen was loaded vertically by constant loads representing gravity loads on the shear walls' tributary areas, and horizontally by equal quasi-static, reversed in-plane shear loads at the two floor levels. The test setup is shown in Figure 10.

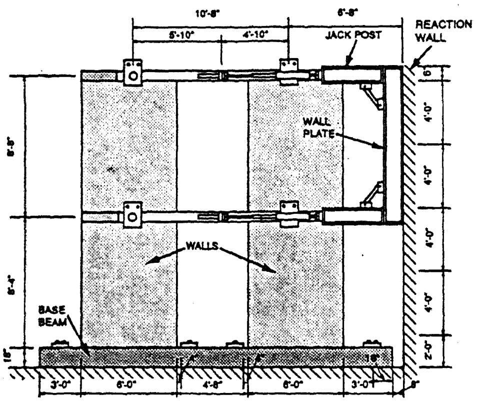

Figure 10. Typical test setup

Typical instrumentation for all specimens consisted of more than 130 data channels, including the following information: in-plane load applied at each floor level; in-plane displacement

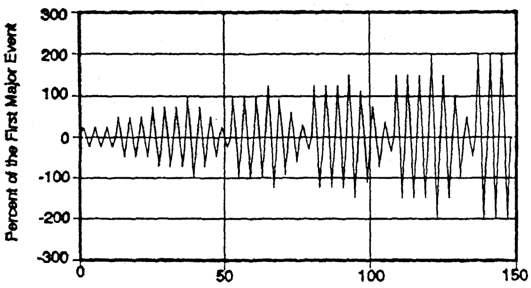

Figure 11. Lateral loading history

at each floor level; concentrated rotations at wall bases and ends of coupling elements; flexural and shearing deformations of wall sections; slip at wall bases; and stresses in vertical and horizontal reinforcement.

All specimens were subjected to the same loading history, shown in Figure 11. The loading history was a repetitive pattern of reversed cyclic displacements to increasing maximum displacement amplitudes. Displacements were expressed in terms of the displacement corresponding to the First Major Event (FME)--usually, yielding of the main longitudinal reinforcement in the wall. Each test was first conducted under load control, and then changed to displacement control as the specimen became more flexible during testing.

5 TEST RESULTS

Test results obtained for each specimen included cracking patterns, load-deflection data, strains in reinforcement, and sufficient information to compute flexural, axial, and shearing deformations in critical regions of the specimens. Typical damage patterns (for Specimens 1b and 2d) are shown in Figure 12. Overall load-displacement

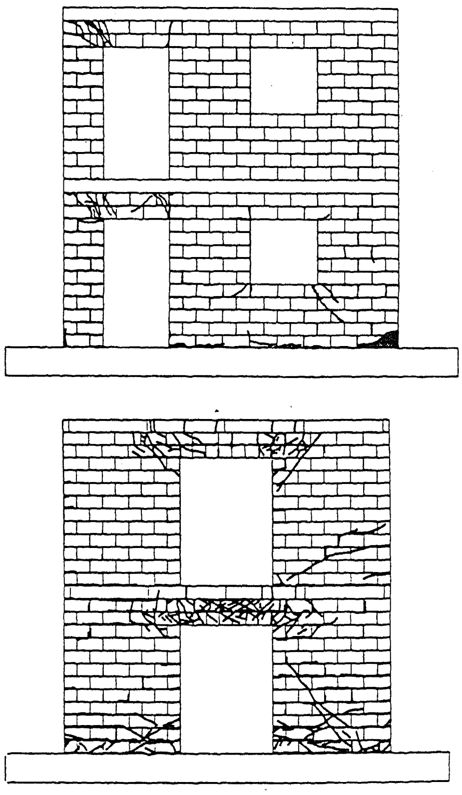

Figure 12. Typical damage patterns for Type 1 and Type 2 specimens

results for Specimens 1b and 2d are shown in Figure 13.

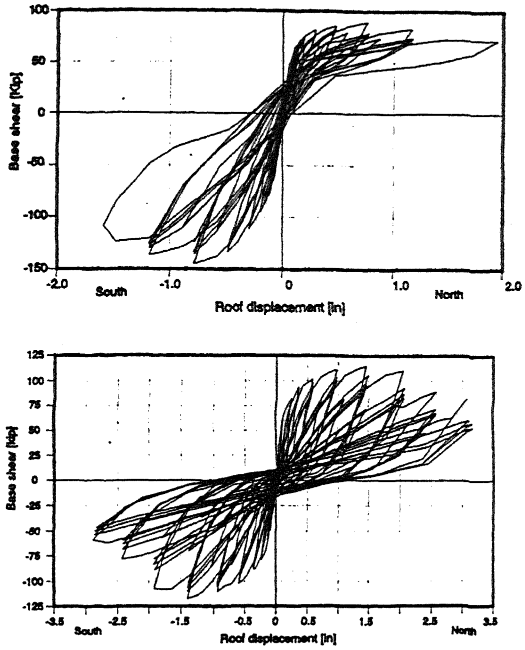

Figure 13. Typical load-displacement results (Specimens 1b and 2d)

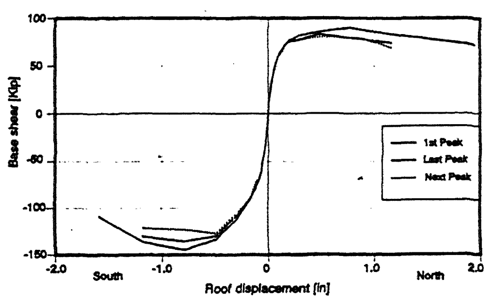

Figure 14. Typical load-displacement envelopes for first and last cycles to same displacement levels (Specimen 1a)

The specimens behaved generally as intended in design. Under many cycles of reversed loading, their response was stable up to story drift ratios as high as 0.7% to 1.0%. In Figure 14, typical load-displacement envelopes for the first cycle to each displacement level are compared with the envelopes for the last cycle to the same displacement. Degradation is seen to be low and response stable, even at higher drift levels. The energy dissipated per cycle did not degrade significantly, even at the maximum story drift levels. Failure was ductile, in that inelastic action was restricted to elements designed to respond primarily in flexure.

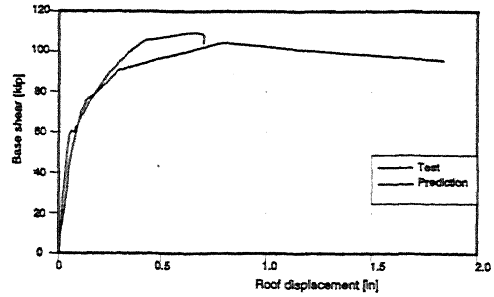

Figure 15. Predicted versus observed response, envelope of 1st peaks, Specimen 1a, southward loading

6 COMPARISON WITH ANALYTICAL PREDICTIONS

Several analytical approaches were used to predict the behavior of the specimens. In the design process, specimen capacity was predicted using simple plastic theory (Leiva 1991). Later, specimen load-deflection envelopes were predicted using a nonlinear analysis program developed in this study, based on moment-curvature behavior of masonry pier and column elements) (Leiva 1991). A typical relationship between observed load-displacement envelope and the behavior predicted analytically using that program is shown in Figure 15.

Maximum capacities and overall load-deformation behavior were well predicted by such inelastic analytical models. Specimen capacity was also compared with the results of nonlinear finite element analyses (Seible 1990).

7 *SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS*

Based on the test results and analytical predictions, a general design approach is proposed for coupled and perforated walls of reinforced masonry subjected to seismic loadings. The approach involves the following steps:

1) Select a stable collapse mechanism for the wall, with reasonable inelastic deformation demand in hinging regions. Using that collapse mechanism, predict the lateral load capacity of the wall in terms of its flexural capacity in hinging regions.

2) Using general plane-section theory to describe the flexural behavior of reinforced masonry elements, provide sufficient flexural capacity and flexural ductility in hinging regions.

3) Using a capacity design philosophy, provide wall elements with sufficient shear capacity to resist the shears consistent with the development of the intended collapse mechanism. Calculate the shear capacity of masonry elements, and the shear transfer capacity between adjacent elements, using expressions developed in this and previous research.

4) Using reinforcement details developed and tested in this and previous research, detail the wall reinforcement to develop the necessary strength and inelastic deformation capacity.

The proposed approach leads to masonry walls with predictable strength and stable load-deflection behavior under many cycles of reversed cyclic load. The proposed approach is recommended for design of reinforced masonry walls in seismic zones, and will be used for the design of the 5-story specimen to be tested later in the TCCMAR program.

8 ACKNOWLEDGEMENTS

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