REINFORCED CONCRETE SHEAR WALLS FOR ASEISMIC STRENGTHENING

by

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SYNOPSIS

Four one-story, one-half scale reinforced concrete frames were strengthened using different types of infilled walls. Reversed cycle tests showed that walls made using single or multiple precast panels demonstrated greater ductility than a wall cast monolithically with the surrounding frame.

INTRODUCTION

Aseismic strengthening of existing structures for improvement of their earthquake resistance may be accomplished before an earthquake occurs or along with the repair of a previously damaged structure. Such strengthening is particularly important for emergency care facilities, such as hospitals, so that they may remain operable after a severe earthquake. The improved resistance is designed not only to prevent collapse but also to limit structural deflections so that architectural and mechanical elements within the building are not damaged.

One method of strengthening which has been used for existing hospitals was to infill the reinforced concrete frames with reinforced concrete shear walls (4). An experimental program was undertaken at the University of Michigan to determine the structural, seismic behavior of frames strengthened using infilling techniques and to indicate those techniques best suited to rehabilitating existing structures (2).

EXPERIMENTAL PROGRAM AND RESULTS

Five one-story, one-bay reinforced concrete frames were constructed; four were infilled with walls as shown in Figure 1. For the frames the 6 in. square columns were reinforced with four #5 bars (5/8-in. diameter), and each beam was reinforced with two #4 bars top and bottom. All infilled walls had a thickness of 3 in., and they had the same ratio of vertical reinforcement, 0.46 percent. Horizontal wall reinforcement equaled 0.44 percent.

The four walls were constructed using different techniques. In Specimen 1 the wall was cast monolithically with the surrounding frame to provide a reference shear wall similar to "new" construction. In Specimen 2 the wall was cast to within 3 in. of the top beam; the 3 in. space was filled with drypack, portland cement mortar. The wall was connected to the frame by #3 bars epoxy grouted into holes drilled at

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9 inch spacing all around the frame and by a roughened concrete surface. In Specimen 3, the wall was a single precast panel which was bolted to the top and bottom beams using wedge anchors. The panel was spaced 3/8-in. from each column so that the panel would not bear against a column and cause a column shear failure. In Specimen 4 the wall was six, independent precast panels which were anchor bolted to the top and bottom beams, were spaced 1/4-in. apart, and were joined together by panel-to-panel connectors. Concrete strength, f'_c, averaged 4300 psi. The average yield stress of the column reinforcement was 51,000 psi and of the wall reinforcement was 63,900 psi.

The four infilled specimens plus the fifth frame without a wall were tested under static, reversed cycle deflections of increasing magnitude. Load was applied using hydraulic jacks located at the end of the "slab" of the top beam, Figure 2. The load-deflection curves of all four specimens were of the shear-slip type.

Figure 2 illustrates the failure mode of each specimen. Specimen 1 failed like previous shear walls tested under monotonic loads, cracking and yielding at the base of the wall followed by shear failure of the compression column. In Specimen 2, failure of the wall-to-top beam joint led to a brittle shear failure of the columns; this resulted in total loss of vertical loading capacity. Beam failure in Specimen 3 was precipitated by anchor bolt extraction. The individual panels of Specimen 4 behaved as a series of deep beams in flexure similarly to "slitted shear walls" (3); the panels failed by shear deterioration at the panel-to-panel connectors.

Figure 3 summarizes the results by plotting non-dimensional shear stress coefficient against deflection. The coefficient is calculated as $|V|$ divided by $hd\sqrt{f'_c}$ where

$|V|$ = absolute value of maximum lateral load in a deflection cycle
d = distance from extreme compression fiber to centroid of tension reinforcement
h = wall thickness

The non-dimensional deflection is given as the lateral story deflection, Δ, divided by the story height, h_s. The d values for Specimens 1 through 4 were resepctively, 75 in., 75 in., 56 in., and 55 in.

The stress coefficient envelopes illustrate the following: For Specimens 1, 2 and 3 the second and third cycle envelopes deviate from the first cycle envelope beyond the yield deflection, and all show rapid loss of load capacity at deflections beyond the maximum load. Specimen 4 envelopes diverged only at deflections greater than 0.02 radian, and showed sustained load capacity at deflections beyond yield.

CONCLUSION

Even though it increased the lateral load resistance, the cast-in-place wall as constructed was considered unsatisfactory for seismic strengthening because its failure mode was brittle and resulted in loss of vertical load capacity. Both precast infilling techniques were satisfactory. The space between the columns and the panels proved most important because it prevented shear failure of the columns and, thus, resulted in a more ductile structure. The multiple panel technique shows the greatest promise for strengthening; it provides greater ductility and cycle load capacity than the others. Because the wall may be precast in narrow units, erection of the wall within an existing buidling appears to be easier and cleaner than the other construction methods. Further, the precast systems appear applicable for strengthening both reinforced concrete and steel frame systems.

The following guidelines are proposed for the design of infilled walls for aseismic strengthening: (a) In constructing cast-in-place walls, dowels should be epoxied into holes drilled all around the frame. An expansive cement concrete is recommended for the wall, and the joint between the cast-in-place section and top beam should be constructed using a highly adhesive and non-shrink grout. Steel bands should be secured around the columns before casting the wall to improve ductility (1). (b) Precast panel walls should be connected only to top and bottom beams to prevent shear loading and premature failure of columns. A gap with a dimension larger than approximately 0.01 times the clear story height should be left between the wall and columns. (c) For anchoring precast panel connectors to existing frames mild steel bolts of threaded rods should be epoxy grouted into drilled holes. (d) Single panel units of a multiple panel wall should be designed and detailed as fixed-ended deep beams. Concrete should be confined by use of closed stirrups as horizontal reinforcement.

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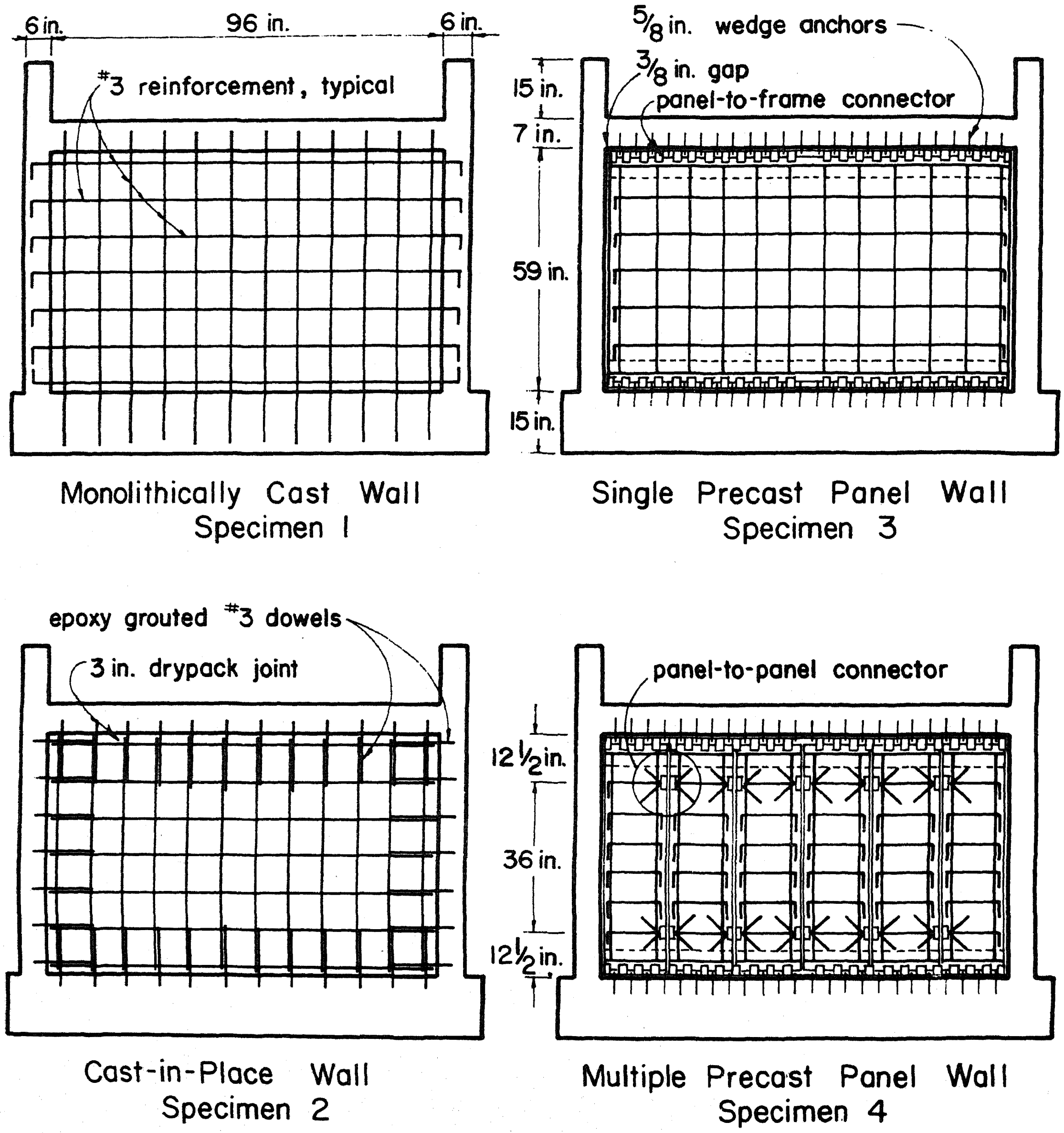
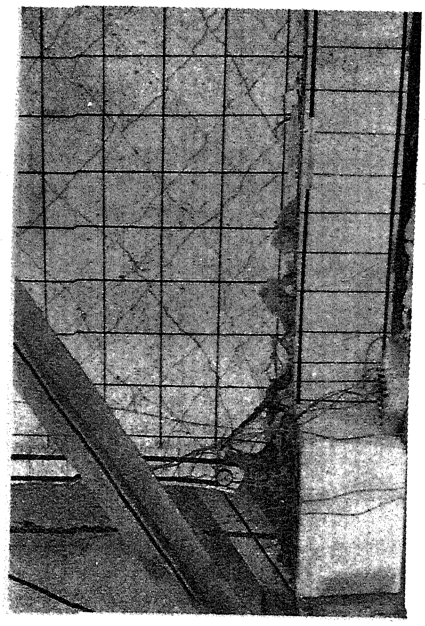
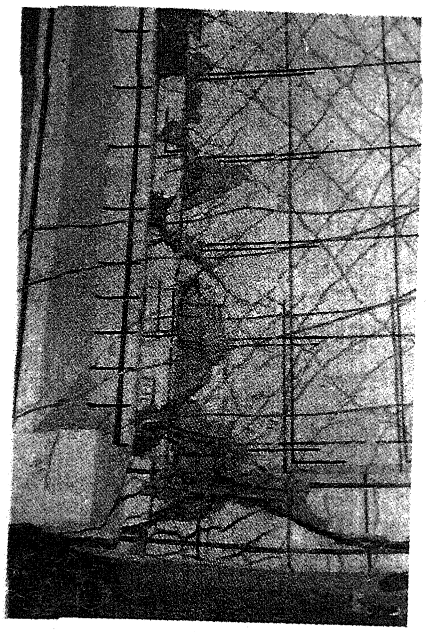


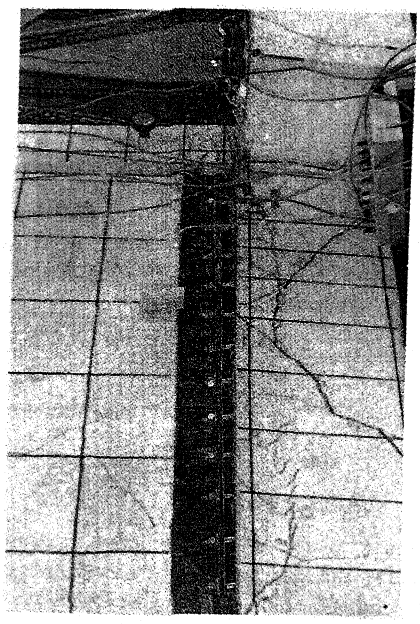
Figure 1. Reinforcing patterns of #3 bars for the four infilled walls.



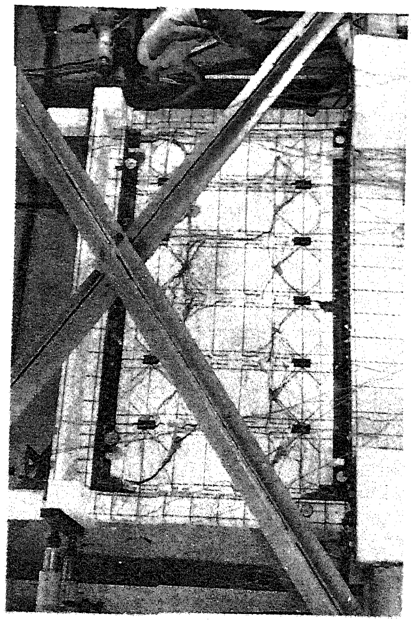
Specimen 1, Shear failure of compression column



Specimen 2, Failure of drypack joint



Specimen 3, Anchor bolt extraction



Specimen 4, Flexure-shear failure of panels

Figure 2. Failure modes of infilled walls ($\Delta/h_s = 0.02$)

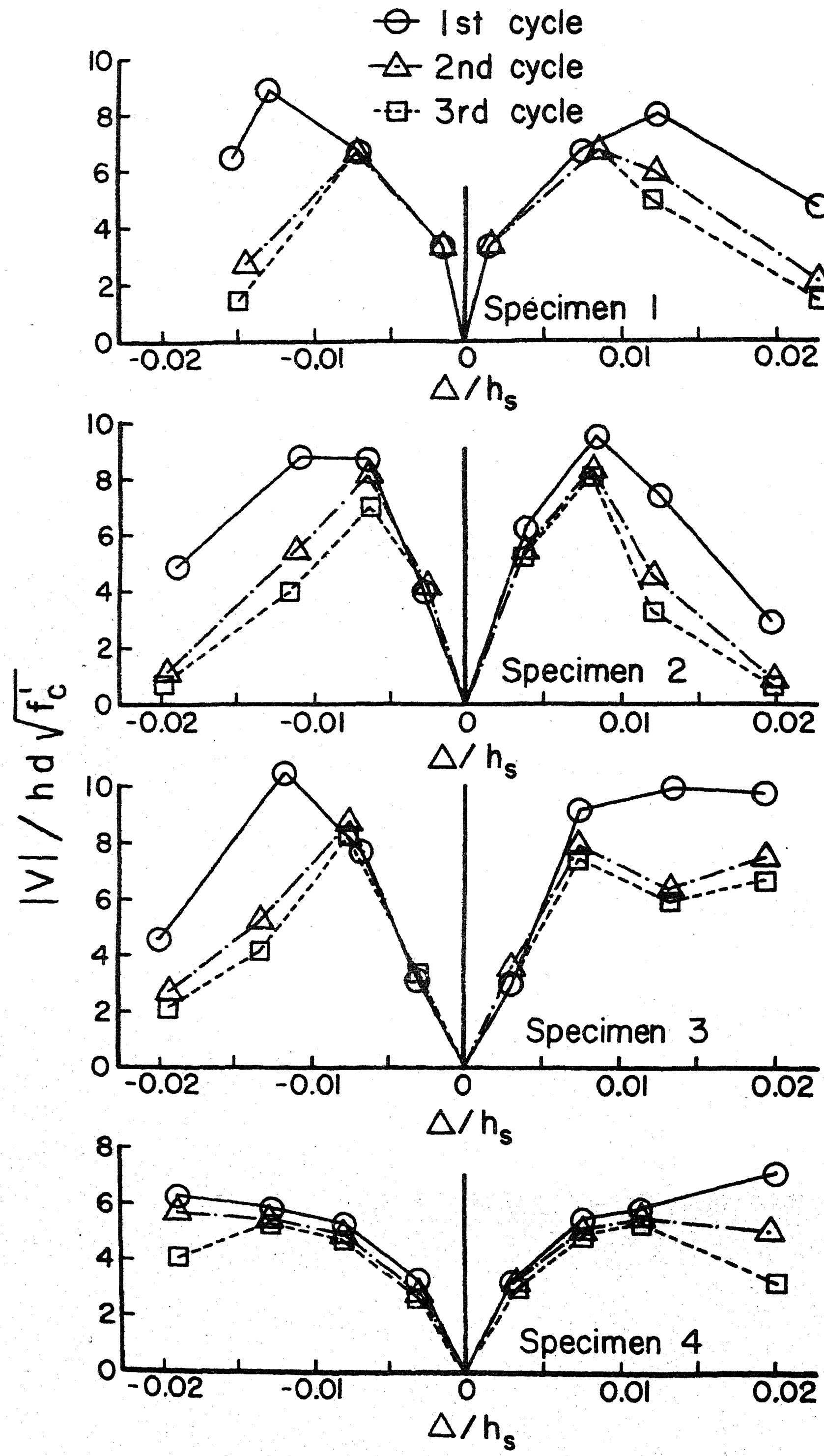


Figure 3. Stress coefficient envelopes for Specimens 1, 2, 3 and 4 from top to bottom.