ASEISMIC STRENGTHENING OF EXISTING REINFORCED CONCRETE BUILDINGS

by

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SYNOPSIS

To determine the seismic behavior of reinforced concrete frames strengthened by using various types of infilling and bracing techniques, an experimental program was undertaken for ten one-story, one-third scale frames. The test results indicated that all the techniques selected for the program were satisfactory to improve the earthquake resistance of existing structures, though each frame strengthened by a different method showed characteristic behavior. Reviewing available test data as well as those discussed herein, design guidelines were proposed with emphasis on improving the lateral force capacity of existing buildings.

INTRODUCTION

A number of buildings damaged by recent earthquakes required extensive amounts of strengthening as well as repair for their rehabilitations. While recent practices in the analytical evaluation of seismic safety of existing buildings have indicated that a large number of buildings, particularly low and middle-height buildings designed and constructed on the basis of previous codes or standards, may need strengthening. Thus, the aseismic strengthening of existing buildings for improvement of their earthquake resistance may be accomplished before a severe earthquake occurs or along with the repair of previously damaged structures. The improved resistance should be designed not only to prevent collapse but also to limit structural deflection so that architectural and mechanical elements within the building will not be severely damaged.

The aims of aseismic strengthening are classified into three categories, that is, (1) to increase the lateral force capacity, (2) to increase the ductility or toughness, and (3) to balance the stiffness and strength of structural elements. The first category is considered be the most effective for low and middle-height buildings which may require large amount of lateral force capacity so that they may resist the considerably high range of expected seismic response. Adequate strength may also be required to avoid extensive inelastic displacement even if the ductility or toughness of the building is satisfactory. The third category can be simultaneously satisfied when the first one is accomplished by means of an appropriate design. To obtain guidelines of design and construction for the aim of the first categoy, several experimental studies were recently conducted with emphasis on techniques to infill or add new structural elements (2$\sim$8). Existing test data, however, have been limited and have not been systematically reviewed.

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The emphasis of the program was directed toward to investigate the seismic behavior of frames strengthened by various types of construction techniques based on static, cyclic loading tests. The techniques to brace as well as to infill new structural elements with the existing frame using different types of materials and connections were selected for the program. Test results were evaluated in terms of the lateral force capacity, ductility and energy absorption compared with those of reference frames. Available existing test data were also reviewed to discuss the results obtained herein.

EXPERIMENTAL PROGRAM

Test Specimens - Ten one-story, one-third scale reinforced concrete frames were constructed. Five of them were strengthened using different infilling techniques while two of them used bracing techniques. Remaining three, an unstrengthened frame and two shear walls with different thickness cast monolithically with the surrounding frame, were provided as reference specimens. Details of specimens are shown in Table 1.

The unstrengthened bare frame F had 20 cm square columns and a 15 cm by 25 cm beam both of which were reinforced with four D13 (#4) rebars. The transverse reinforcement in framing members was designed so that the frame may fail in shear before or immediately after yielding. Reference shear walls W-80S and W-40S were designed to have different thicknesses, 80 mm and 40 mm respectively, but identical reinforcement ratio, 0.70%, to the sectional area of each wall panel.

In specimens W-HA and W-CO, each frame was infilled with a cast-in-place concrete panel having identical thickness to that of W-80S. In the case of W-HA, the panel was connected to the frame by 10 mm dowels with wedge anchors drilled at 10 cm intervals all around the frame, while the panel was connected by mortar shear keys which were epoxied and bolted at 15 cm intervals all around the frame in the latter case of W-CO. Specimen W-S was infilled with a steel panel bolted at 10 cm intervals all around the frame, and the space between panel and frame was filled with mortar. In Specimen W-BL, precast concrete blocks in a special ⊏⊐-shape were infilled grouting holes in blocks provided for vertical reinforcements, and the space between blocks and the frame. Vertical reinforcements were connected to the top and bottom beams using wedge anchors. In Specimen W-40W, a thin monolithic wall identical to W-40S was thickened by a cast-in-place concrete panel having the identical thickness, however, no connection was provided between new and existing concrete.

Compression and Tension Braces of steel were used for Specimens B-C and B-T, respectively. Compression braces of H-sectioned steel were attached to the ends of beams and columns by screwing adjusting bolts which were set on cover plates welded with the end of braces. The space between frame and cover plates of braces were filled with mortar. In Specimen B-T, tension braces of 28 mmϕ plain bars were welded at each end with connecting plates which covered the ends of beams and columns.

Material and Construction - Compressive strengths of concrete for existing frames, additional walls and blocks were 240, 380 and 300 kg/cm^2, respectively. It was important to use higher strength concrete for additional walls. The value of mortar grout was 175 kg/cm^2. Nominal yield strengths of plain bars 4 mm, 6 mm and rebar D13 (#4) were 3790, 3380 and 3760 kg/cm^2. The values for steel panel, compression and tension braces were 2140, 3550 and 3080 kg/cm^2. A dowel of 10 mm had high yield strength of 7180 kg/cm^2. The strengths of a wedge anchor and an epoxied mortar shear key were 2600 kg/cm^2 and 55.7 kg/cm^2, in terms of the pull-out stress and shear stress, respectively.

The construction of specimens for both existing frames and strengthening elements except infilling concrete were performed in a vertical position. Additional concrete walls were cast in a horizontal position, though this is not realistic in actual buildings, to avoid difficulty in grouting the gap between the top of wall and the bottom of beam. In real size structures, this type of difficulty is solved by using expansive mortar or other grouting techniques. To simulate the axial stress of a column due to a vertical load, each column of the frames was prestressed by a non-grouted steel bar embedded in the column. The prestress was 13% the specified concrete strength. It was important to do this before the construction of strengthening to avoid possible stresses in strengthening elements associated with the column load which may be unlikely in real buildings.

<u>Testing</u> - All specimens were tested under static, reversed cyclic deflection of increasing magnitude. Two types of displacement histories were selected for unstrengthened and strengthened frames, respectively, because of different ranges of expected displacement. The displacement cycle was, in general, alternately once or twice at each step of displacement with interval of 0.001 radian. Lateral load was applied using a hydraulic jack located at the end of top beam. To simulate more realistic distribution of lateral force acting on the frame, side beams of steel boxes which were parallely placed to the top beam and "pinned" at stub beams were provided so that a part of the load applied at the end of beam may be transferred to the other end.

TEST RESULTS

A Summary of experimental results is given in Table 2. The load-deflection diagram and the failure mode of each specimen are shown in Fig. 1, and envelopes of load-deflection diagrams are summarized in Fig. 2.

<u>Reference Specimens</u> - After forming the hinge mechanism, the bare frame F failed in a brittle manner when the displacement reached 0.026 radian which was six times the displacement at first yielding. The failure was caused by a shear tension in a column. In monolithic walls W-80S and W-40S, the shear sliding initiated at the displacement about 0.004 radian after yielding of a boundary column, thereafter, the thin wall W-40S indicated immediate and considerable loss of load capacity while the thick wall indicated gradual loss of strength until the displacement about 0.01 radian.

<u>Infilled Walls</u> - Cast-in-place concrete walls W-HA and W-CO behaved as a typical shear wall having almost identical lateral force capacity and similar deterioration of strength to those of the monolithic wall W-80. In

these walls, the loss of load capacity resulted from the shear failure at the top of both columns associated with a shear sliding at the top of the wall panel. The lateral force capacity of the ribbed steel wall W-S was 0.8 and 4.3 times those of Specimens W-80S and F, respectively. The pull-out of wedge anchors and the shear failure of columns resulted in significant loss of strength despite of no damage to the steel panel. The block wall W-BL behaved in rather ductile manner, though the lateral force capacity was the lowest among the specimens, 0.63 and 3.5 times those of W-80S and F. In the thickned wall W-40W, the additional wall did not crack significantly while the existing wall and framing elements indicated brittle failures identical to that of the corresponding wall W-40S. The lateral force capacity was higher by 20% and the hysteresis loop was much richer than those of W-40S. Thus, the overall cyclic behavior of a thin wall was significantly improved by a simple construction.

Braced Frames - In Frame B-C having compression braces, the increased lateral force capacity was not so high since a shear sliding occurred at the bottom of a column in an early step of the displacement, 0.004 radian. The deterioration of strength, however, was not significant. While the frame B-T with tension braces indicated the most ductile behavior even though the increase of lateral force capacity was not significant (67% of W-80S). The hysteresis curves were as rich as those of steel structures. No loss of load capacity was observed until the displacement about 0.02 radian.

DISCUSSIONS

Evaluations of Test Results - As listed in Table 2 and shown in Fig. 2, the hysteretic behavior of strengthened frames was evaluated from viewpoints of the initial stiffness, load capacity, displacement ability and energy absorption compared with those of reference frames.

Infilled concrete walls indicated the greatest values of both increased strength and stiffness. The degradation of strength after the maximum load was moderate. Although steel elements did not provide as much strength as of infilled walls, braced frames had the advantage of great energy absorption and/or displacement ability while the steel wall indicated higher strength but significant loss of capacity due to the failure of joints. More attention must be paied to the joint so that steel elements may sufficiently develop their capacities. Concrete blocks provided adequate ductility as well as moderate strength since they were improved in their shapes and were sufficiently reinforced. Even a simple construction to thicken a wall significantly improved the overall cyclic behavior of an existing structure.

Thus, the effect of strengthening was evaluated from different view points since the strengthened frames behaved in their own characteristic manners. Note that all the strengthened frames finally resulted in the failure of columns as shown in Fig. 1, however, without significant loss of vertical load capacities.

Effect of Strengthening - The effect of strengthening by different techniques is schematically illustrated in Fig. 3 together with other test results (4-6). Note that the figure provides only ideas how much strength and displacement we may have by using available techniques. As indicated in the figure, infilled concrete walls may have more than 0.6 or 3.5 times the strength of a monolithic wall or unstrengthened frame, respectively, when adequate connections are provided. Steel elements provide less increased capacity than of concrete panel, however, larger ductility. Strengthened columns by wing walls provide up to two times the strength of the original column and the displacement at the load capacity is more than 0.015 radian. Generally, the smaller the increased capacity, the larger the displacement ability.

Strength of Connections of Infilled Walls - The contribution of shear connectors to the increased lateral force capacity of infilled concrete walls was discussed on the basis of existing test data (5 through 9) as well as author's. The selected ten infilled walls, mostly one-story, were one-third through half scales and were 0.44 - 0.71 in height to span ratios. Dowels of 6 mm through 14 mm in diameter were used for connectors. The capacity of infilled walls was expressed in terms of the ratio to that of monolithic wall (Qw) or nondimensional shear stress. The strength of connection was evaluated in terms of the nominal shear stress to a wall section, τ_j.

The strength of a wall was more than 2 $\sqrt{Fc}$ or 0.6 Qw when the strength of connection τ_j was more than 10 kg/cm^2. It was interesting that the capacity of a wall might be more than 1.0 $\sqrt{Fc}$ or 0.4 Qw without any connections. As shown in Fig. 4, the strength ratio was proportional to τ_j while the increased strength was minor for τ_j more than 10 kg/cm^2 (Fig. 5). It was suggested that connections be provided all around the frame when the strength more than 0.6 Qw was required. It was also recommended for more strength of connection as well as infilled wall to use 1) high strength dowel, 2) mortal shear key together with wedge anchor, and 3) high strength concrete.

CONCLUSIONS

From the viewpoint of improving the lateral force capacity of existing structures, all the techniques selected herein were satisfactory because they provided considerably higher strength than that before the strengthening without any significant losses of vertical load capacity. It should be noted that each frame strengthened by a different technique indicated own characteristic behavior, therefore, the effect of strengthening should be synthetically evaluated in terms of the ductility, energy absorption as well as load capacity.

Although the data obtained herein have been very limited, the following guidelines are proposed: (1) Infilled concrete walls promise to significantly increase both the strength and stiffness of existing structures when the connection is appropriately designed. For the case with required strength of a wall more than 60% of that of a monolithic wall or 2 $\sqrt{Fc}$ kg/cm^2, the connection should be designed to have the strength more than 10 kg/cm^2, and it is desirable to provide connectors all around the existing frame. High strength materials for both connectors and a wall are also recommended, (2) Steel elements, particularly braces, are promising moderate strength as well as adequate ductility and/or energy absorption. Attention should be carefully paied to the detailing of connections since they seemed to strongly affect the overall cyclic behavior of braced frames, and (3) Simple construction techniques such as to thicken existing wall or to infill concrete blocks are also preferable because they avoid to harm the existing columns during the construction.

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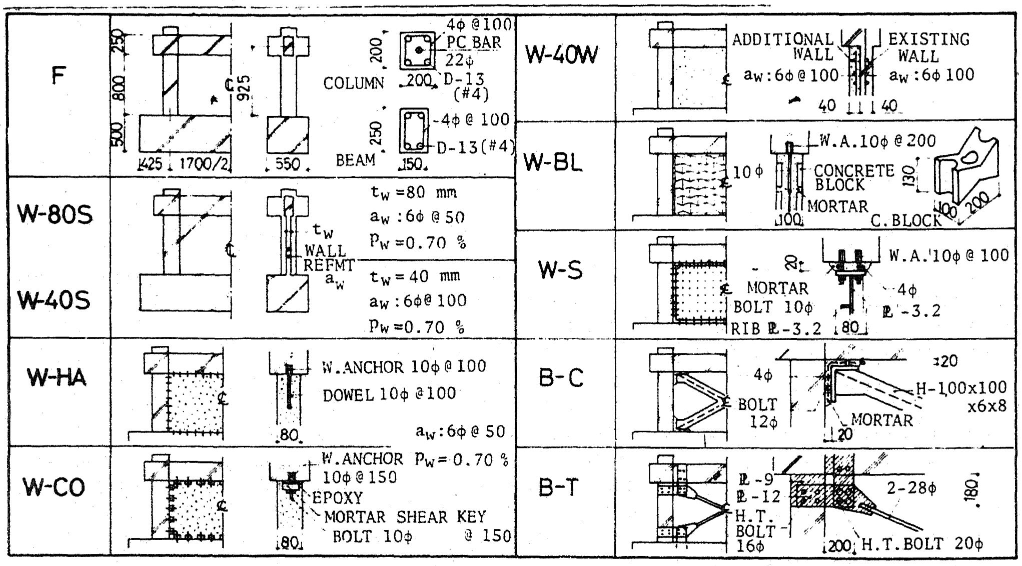
TABLE 1 TEST SPECIMENS

unit in mm



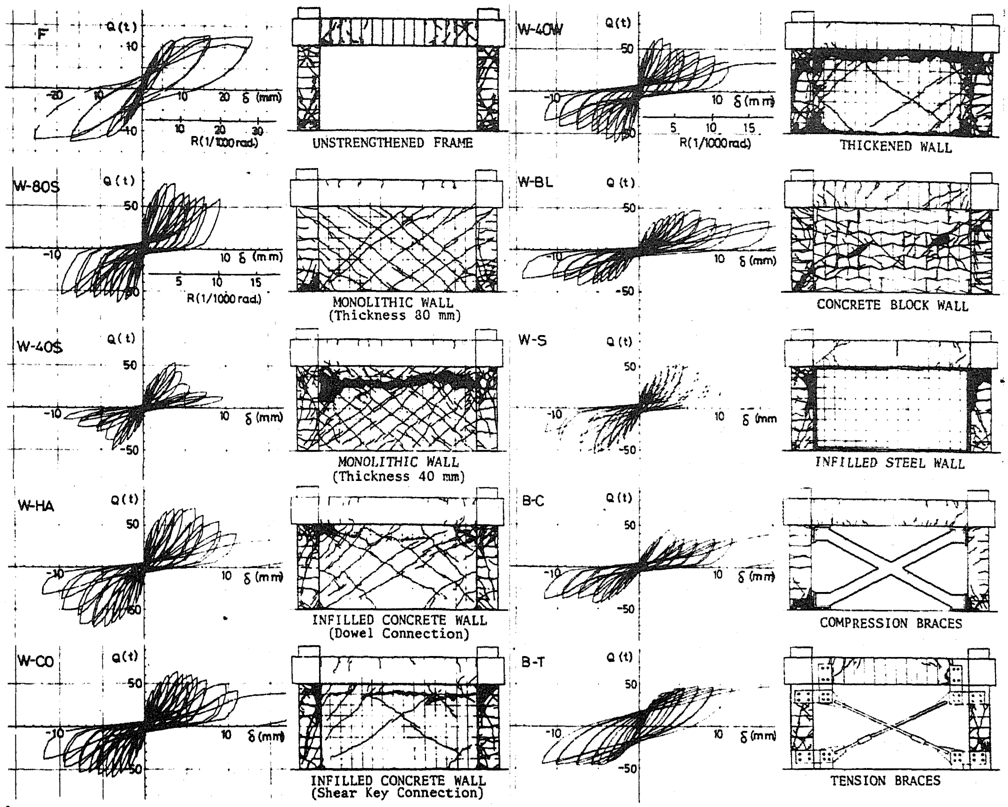


FIG. 1 HYSTERESIS CURVES AND FAILURE PATTERNS

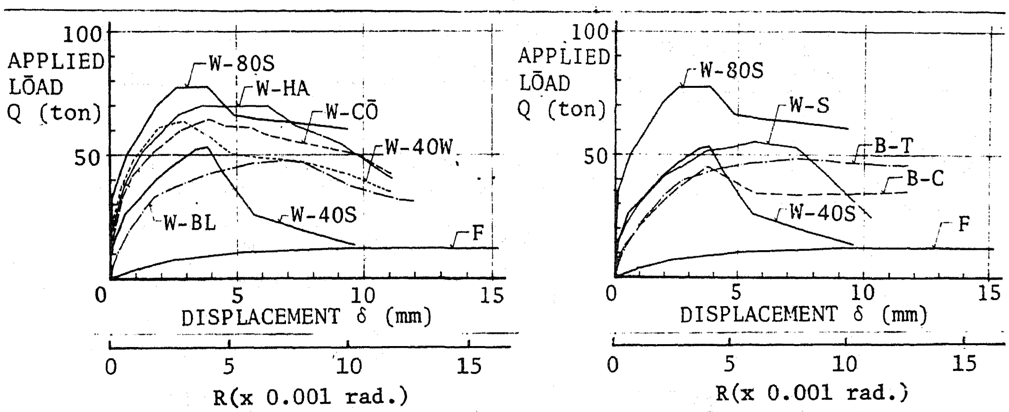
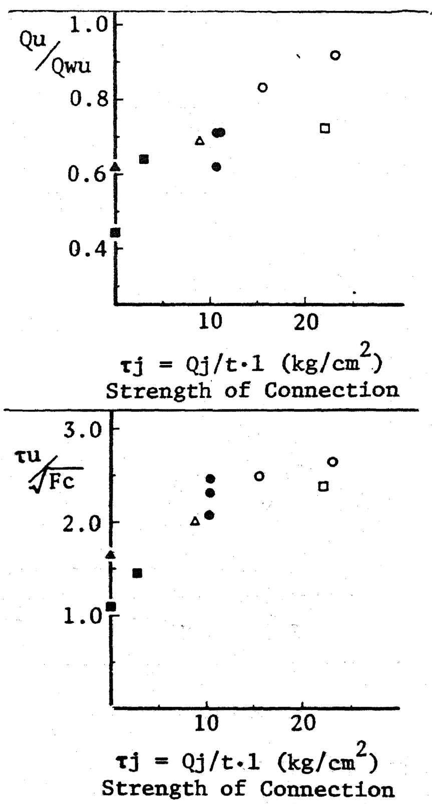


FIG.2 ENVELOPES OF HYSTERESIS CURVES

TABLE 2 EXPERIMENTAL RESULTS

Specimen	Initial Stiffness *1	Load Capacity Qu *3 *1	Load Capacity Qu *3 *2	Displacement at Qu ($\times 10^{-3}$ rad.)	Ultimate Displacement ($\times 10^{-3}$ rad.) *4	Energy Absorption (ton x cm) *5
F	1.0	1.0	--	17.7	28.6	
W-80S	22.0	5.6	1.00	4.2	8.7	241
W-40S	13.7	4.1	0.73	4.2	4.7	99
W-HA	24.0	5.5	0.98	6.7	9.5	227
W-CO	25.5	4.9	0.87	4.2	9.6	212
W-40W	26.3	4.9	0.87	3.1	4.5	234
W-BL	7.3	3.5	0.62	7.4	10.0	130
W-S	11.8	4.3	0.77	6.1	8.8	182
B-C	6.5	3.5	0.62	4.0	>10.0	114
B-T	5.7	3.7	0.66	7.9	>10.0	255

*1 Ratio to Frame F *2 Ratio to Frame W-80S *3 Mean of peak loads
*4 Displacement at the load 0.8 x Qu *5 Cumulative area of hysteresis loops until the displacement 0.01 rad.



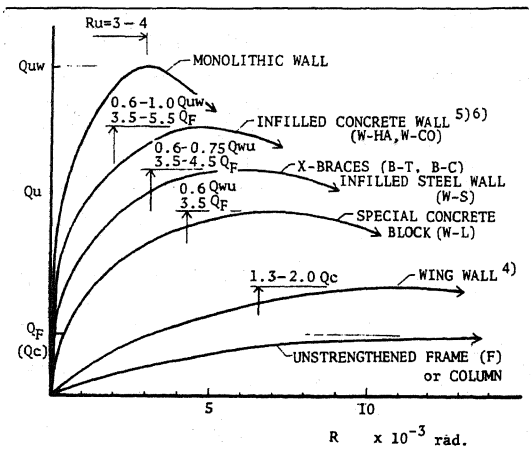


FIG. 3 OUTLINE OF LOAD vs DISPLACEMENT FOR DIFFERENT STRENGTHENING TECHNIQUES

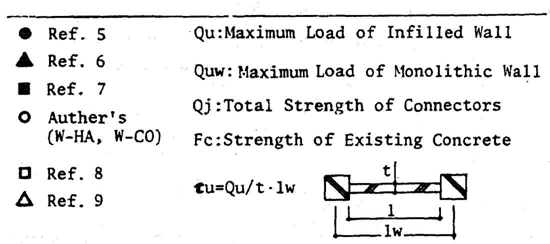


FIG. 4 STRENGTH OF INFILLED WALL vs STRENGTH OF CONNECTION