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Literature Review of Strengthening Methodologies of Existing Structures

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ABSTRACT

This report reviews research studies relevant to structural strengthening of existing reinforced concrete members and frames. The majority of these studies dealt exclusively with restoring or improving seismic resistance of concrete columns and frames. A number of case histories where various strengthening techniques were applied in practice are reviewed. Most studies identified ultimate failure in the strengthened structures as being primarily due to failure of the joining elements. Improved load resistance and ductility in concrete structures have been reported in most of these studies.

Key words: Anchors, beams, columns, deflection envelopes, ductility, epoxy adhesive, hysteresis curves, infill walls, lateral load carrying capacity, lateral stiffness, reinforced concrete frames, strengthening, steel braces, wingwalls, walls.

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1. INTRODUCTION

1.1 GENERAL:

Strengthening of existing buildings is often called for when there is a need to upgrade them to satisfy new building code requirements or to improve the load carrying capacity. This report presents a summary of experimental studies on strengthening methods (chapter 2), case histories of field applications of strengthening methods to existing structures (chapter 3), and recommendations for areas needing further research (chapter 4). While all building codes clearly specify the structural requirements for the design of new construction, the design for strengthening of an existing building is still based mostly on engineering judgement. This is due largely to the lack of an established approach based on research on methods of strengthening and for assessing the structural performance of strengthened structures. The 1982 Rehabilitation Guidelines [1.1] developed by the U.S. Department of Housing and Urban Development provided general guidance for damage assessment of common building structural systems such as masonry bearing walls and simple wood, steel, and concrete frames. However, these guidelines are intended for use on a voluntary basis in conjunction with existing building codes and standards, and are only applicable to repair work. For strengthening of structures, an engineer must rely on his own judgement in assessing areas of weakness in a structure and then develop an appropriate strengthening scheme based on that assessment. Furthermore, techniques which have been used to strengthen existing structures are not based on experimental data. Thus, accurate assessment of the expected performance of the strengthened structure is difficult to make.

1.2 SCOPE OF REPORT:

To identify relevant studies in strengthening methodologies, a literature search was conducted using the data base of the Engineering Index System and the National Technical Information Service. These two data bases identified over 200 abstracts based on key word input. Review of the abstracts revealed a limited number of papers and reports which dealt with structural members and frames. Additionally, papers published in proceedings of national and international conferences on earthquake engineering, proceedings of the U.S.-Japan seminars on repair and retrofit of structures, and research reports of U.S. and Japanese universities have been reviewed. Review of the literature clearly revealed that a disproportionally large number of studies dealt with seismic strengthening of reinforced concrete and masonry structures. This indicates that there is a considerable concern for strengthening of existing reinforced concrete and masonry structures in earthquake regions. On the other hand, the limited number of reports on strengthening of steel or timber structures suggests that strengthening of these types of structures is rather straightforward or has been less frequent and therefore not of a great concern. This is probably because attachment of new structural members to steel or timber structures can be accomplished simply through the use of mechanical fasteners or welding for steel structures.

Thus, this report reviews studies of structural strengthening of damaged and undamaged reinforced concrete and masonry structures. Studies which dealt primarily with repair of damaged structural members are not included in this review.

2. SUMMARY OF RESEARCH STUDIES

2.1 INTRODUCTION:

Much of U.S. research efforts in strengthening of existing structures have focused on developing means to improve seismic performance of various structural members such as beams, columns and walls. Since the 1971 San Fernando earthquake, a number of research programs were initiated in the U.S. to examine the effectiveness of various methods for repairing damaged structural members and for seismic retrofitting of old buildings.

Having experienced extensive damage to concrete and wood structures during the 1968 Tokachi Oki earthquake and the 1978 Miyagi Ken Oki earthquake, extensive studies on repairing of damaged structural members and retrofitting of existing buildings have been undertaken by Japanese researchers. Most of their results have been published in Japanese.

This chapter reviews experimental studies reported in both U.S. and Japanese papers and reports. These include strengthening methods for reinforced concrete and steel frames, concrete and masonry walls, concrete beams and columns.

2.2 REINFORCED CONCRETE AND STEEL FRAMES:

Methods used in strengthening reinforced concrete and steel frames may be grouped into three main categories: infill walls, steel braces, and wingwalls.

2.2.1 Infill Walls:

Eight major experimental investigations on the effectiveness of infill walls to resist lateral forces and to increase lateral stiffness have been reported. These investigations included one-bay, one-story, and one-bay, three-story reinforced concrete frames and one-bay, one-story steel frames strengthened by various infilling techniques proposed by Hayashi et al [2.1], Sugano and Fujimura [2.2], Higashi et al [2.3], Kahn [2.4], Shiohara et al [2.22], Aoyama and Yamamoto [2.23], Makino et al [2.5], and Mallick [2.6].

(1) In the study conducted by Hayashi, Niwa and Fukuhara [2.1], a series of six 1/3-scale, one-bay, one-story reinforced concrete frames, W-1 to W-6, were tested. The specimens consisted of one rigid frame without infilled wall (specimen W-1), one monolithic wall/frame system (specimen W-2), and four rigid frames strengthened by cast-in-place infilled concrete walls (specimens W-3, W-4, W-5 and W-6). The primary objectives of this study were 1) to examine quantitatively the effectiveness of the infill wall technique as a method for strengthening frames, and 2) to study the influence of different methods of joining the new infilled concrete wall to the existing frame.

The four strengthened frames differed by the type of the joining elements used in connecting the infilled walls to the existing frames and in their distributions on the wall/frame interfaces. Two different types of joining elements were used in this study, precast concrete shear keys and wedge anchors. The precast concrete shear keys, approximately 3/4 in. (2 mm) thick, 1.5 in. (4 mm) wide, and 3.0 in. (8 mm) long, were epoxy-bonded onto the innerface of the frame for specimen W-3. Wedge anchors were used for the remaining three specimens W-4, W-5, and W-6. In specimen W-4, wedge anchors were installed only under the upper beam, with the other three inner sides of the frame roughened. In specimens W-5 and W-6, wedge anchors were installed on all four sides of the specimens, and the inner sides of the frame were roughened except for specimen W-6 in which only the bottom surface of the beam was roughened. Figure 2.1 shows the typical configuration of the test specimens, methods for strengthening, and the arrangements of the reinforcement in each specimen. The frames were subjected to reverse cyclic lateral load, applied on the sides of each frame at the level of the top beam center line, in combination with a constant axial load of 12 ton, maintained on top of each column, as shown in Figure 2.2. The test results, presented in the form of hysteresis curves and their envelopes, are shown in Figure 2.3.

The lateral stiffness of all of the infilled frames was significantly greater than the stiffness of the bare frame. In fact, the lateral stiffnesses of infilled frames approached or were even slightly greater than the stiffness of the monolithically-cast wall/frame. The lateral force capacities of the infilled frames were 3.5 to 5.0 times that of the unstrengthened frame, and 0.55 to 0.72 times that of the monolithic wall/frame. Hayashi, Niwa, and Fukuhara also observed shear failure of both types of shear connectors along the top beam/infill wall interface at large deflection.

(2) A similar but more comprehensive experimental program was conducted by Sugano and Fujimura [2.2]. This test program consisted of ten 1/3-scale, one-story, one-bay frames. For strengthening, cast-in-place concrete wall panels, steel panels, and specially shaped precast concrete blocks were used as infilled walls. Of the ten specimens proposed, five were strengthened using using infill walls while two were strengthened by steel bracing. The remaining three specimens included an unstrengthened frame and two monolithic wall/frames with walls of 1.5-in (40-mm) and 3.0-in (80-mm) thick. The behavior of the two specimens strengthened by steel braces, B-C and B-T, will be discussed in the next section under frame strengthening by steel bracing. Details of the five infilled frames specimens are described here; details of all specimens are given in Table 2.1.

1. Specimen W-HA was infilled with a cast-in-place concrete wall 3.0-in (80-mm) thick, the infilled wall was connected to the frame by 0.4-in (10-mm) diameter wedge anchors, spaced at 3.75-in (100 mm) intervals all around the entire frame.
2. Specimen W-CO was also infilled with 3.0-in thick cast-in-place concrete wall. However, the connectors were a combination of both mortar shear keys and wedge anchors. The mortar shear keys were epoxy-bonded and bolted at 5.6-in (150-mm) intervals all around the frame.

3. Specimen W-40W was a monolithic wall/frame with wall of 1.5 in. (40 mm) thick. The existing wall was thickened by a cast-in-place concrete wall panel of the same thickness. However, no connection was provided between the infilled wall and the frame.
4. Specimen W-BL was infilled with specially shaped precast concrete blocks with holes at the center to accomodate vertical reinforcement. The gaps between the concrete block wall and the frame were filled with mortar. Vertical reinforcement was connected to the top and bottom beams using wedge anchors, which were placed at 7.5 in. (200 mm) intervals.
5. Specimen W-S was infilled with a steel panel bolted at 3.75-in (100-mm) intervals around the entire frame. The space between the steel panel and the frame was filled with mortar.

Each specimen was subjected to a combined lateral reversed cyclic deformation of increasing magnitude and axial load. The axial load in each column was to simulate the vertical load and was provided by prestressing a non-grouted steel bar, embedded in the column, to about 13% of the specified concrete strength. The effectiveness of the strengthening techniques proposed in this testing program was evaluated based on the ultimate lateral load capacity, ductility, and the energy absortion of each specimen. The test results are summarized in Table 2.2. The hysteresis curves, associated envelopes, and a comparison of the strength of different connections are shown in Figures 2.4, 2.5, and 2.6. In terms of improving lateral force capacity, this study reached conclusions similar to those of Hayashi, Niwa and Fukuhara [2.1]. The lateral force capacities of the infilled wall were found to be 3.5 to 5.5 times that of the unstrengthened frame and 0.62 to 0.98 times that of the monolithic wall/frame. The lateral stiffnesses of the infilled walls were also significantly greater than that of the unstrengthened frame. However, however it was found that in contrast to the results of Hayashi, et al [2.1], the lateral stiffnesses were slightly less than that of the monolithic wall/frame, as indicated by the envelopes of the hysteresis curves (see Figure 2.5).

Sugano and Fujimura also examined the required strength of the shear connectors as a function of the lateral load capacity by plotting the ratio of the ultimate lateral force capacities of the infilled frames and the monolithic wall (Q_u/Q_{wu}) against the nominal shear stress on the wall/top beam interface, as shown on Figure 2.6. From this plot, Sugano and Fujimura concluded that in order to provide a bare frame with a lateral force capacity of at least 60% of that of a monolithic wall/frame, the connection between the top beam and the infilled wall should be designed to have a shear strength of at least 10 kg/cm^2. Further, the lateral force capacity of an infilled frame could be increased to 98% of that of a monolithic wall when a shear strength of at least 20 kg/cm^2 could be provided by the shear connectors, as in the case of specimen W-HA. Data obtained from Kokusho and Endo also indicated that a frame infilled without the use of shear connectors could possess a lateral force capacity of at least 40% of that of a monolithic wall.

(3) Another major research study in strengthening of frames was conducted by Higashi, Endo and Shimizu [2.3, 2.7]. This study, spanning over several years, consisted of four series of reinforced concrete and mortar frame specimens. The 1977 and 1978 series included fourteen 1/3-scale, one-bay, one-story concrete frames, the 1979 series included eight 1/8-scale, one-bay, one-story reinforced mortar frames, and the 1981 series [2.7] consisted of four three-story two-bay reinforced mortar frames.

Various strengthening techniques were evaluated in this study, including:

1. Infilling with cast-in-place concrete panels which were connected to the frame by wedge anchors.
2. Complete or partial infilling with precast concrete panels, also connected to frame by wedge anchors.
3. Bracing with steel frame, steel truss and steel braces.
4. Enhancing web reinforcement in columns with steel plates.

Description of the test specimens and the strengthening techniques used in this study are given in Table 2.3. The details of all 26 specimens in the four test series are shown in Figure 2.7 and details of the connection between the infilled wall panels and a frame are shown in Figure 2.8.

All specimens were subjected to reversed cyclic loading in combination with a constant axial load applied and maintained on top of each column throughout the loading history. The axial load, which was designed to simulate gravity load in the structure, was selected such that a compressive stress of 30 kg/cm^2 would result in each column of all specimens. The typical test setups for one-bay one-story, one-bay three-story, and two-bay three-story specimens are as shown in Figure 2.9.

Both qualitative and quantitative comparisons of the effectiveness of different strengthening techniques studied in this test program were provided in the forms of the envelopes of hysteresis curves as shown in Figure 2.10, and the antiseismic capability indexes C_E as summarized in Table 2.4. The C_E index, defined as $(Q_u/2N)\sqrt{2\mu-1}$ where Q_u, N and μ denotes the ultimate lateral force capacity, the axial load in each column, and the ductility factor, respectively, was adopted by Higashi, Endo, and Shimizu as a measure of the antiseismic capability of each specimen. In general, strengthening by complete infilling of frames with either cast-in-place or precast concrete wall panels, connected by sufficient number of wedge anchors, was found to be the most effective strengthening technique in this study. This technique provided the infilled frames with almost the same lateral force capacity and ductility as that of monolithic wall/frame. In the one-bay, three-story specimens (series 1979), both the lateral force capacity and the ductility of frames infilled by cast-in-place concrete wall appeared to be higher than those of identical monolithic wall.

(4) The effectiveness of four different infilling techniques were investigated by Kahn [2.4]. In this test program, five one-half scale, one-bay, one-story reinforced concrete frames were tested statically

under reversed cycle deflections of increasing magnitude. The five specimens include one monolithically cast wall/frame (specimen 1), one bare frame (specimen 2), one frame infilled with cast-in-place wall (specimen 3), one frame infilled with a single precast concrete panel (specimen 4), and one frame infilled with multiple precast concrete panels (specimen 5). The reinforcement arrangements of frames and of infilled walls are shown in Figures 2.11 (a) to (e). The hysteresis behavior of all five frames are shown in Figures 2.12 (a) to (e), and the envelopes of hysteresis curves are shown in Figure 2.13.

The monolithic wall/frame (specimen 1) showed the greatest lateral load carrying capacity of all specimens (150 kips). Failure occurred at a deflection 1-1/2 times the yield deflection in this specimen. Specimen 3 attained the same ultimate lateral load capacity. This specimen failed due to deterioration of the joint between the cast-in-place wall and the top beam. Brittle failure also occurred at a deflection of 1-1/2 times the yield deflection. Specimen 4 attained 75% of the lateral strength of the monolithic wall/frame (specimen 1), while its ductility was twice as large. This specimen maintained its maximum load over 3-1/2 times its yield deflection in one direction. In the other direction, weld fracture and pullout of the panel-to-frame connections caused rapid loss in load capacity and ductility. The multiple precast panels (specimen 5) attained 50% of lateral strength of the monolithic wall/frame. Four interior panels failed in shear. This specimen showed little cyclic degradation until the interior panels failed in shear at a relative deflection angle of 0.02 radians.

The results of these tests showed that the multiple precast panel infilling technique is the most effective technique to provide increased ductility for frames, and the cast-in-place infilled wall technique is superior for providing frames with increased lateral strength and stiffness.

(5) Effects of anchorage deterioration on flexural strength of frames infilled with a cast-in-place wall were investigated by Shiohara,

et al [2.22]. Four 1/3-scale, one-bay, one-story reinforced concrete frames were tested: one was a monolithic wall/frame (specimen P2005), and three were infilled with concrete walls (specimens C2005-I to III). Prior to infilling, all of the inner sides of the frames were roughened to depths of about 0.2 in. (5 mm). Splice bars, 0.5 in. (13 mm) in diameter and 12 in. (300 mm) long, were inserted into predrilled holes in the frames and anchored using epoxy adhesive to provide connections between frames and infill walls. In specimens C2005-I and C2005-II, all splice bars were embedded to a depth of 5.0 in. (130 mm), and in specimen C2005-III, they were insetred to a depth of 4.5 in. (117 mm). All infilled walls were heavily reinforced for shear (0.85%) so that a flexural failure would prevail. Round bars were used as flexural reinforcements in two specimens, C2005-II and C2005-III, and deformed bars were used in specimen C2005-I. Specimen dimensions and the reinforcement arrangement are shown in Figure 2.14.

Gravity load was simulated by applying a constant axial load of 44 kips (20 ton) at the top of the columns. Five cycles of statically reversing lateral load were applied at both ends of the upper girder. In all specimens, ultimate strength was attained in the second cycle of loading, and the specimens were further loaded to failure in the fifth cycle.

Figures 2.15 and 2.16 show the relationships between the lateral load and the horizontal displacement of girders relative to the bases, and the displacement components. The load-deflection curves (Figure 2.15) show that the infilled frames had lateral strengths of up to 0.77 to 0.86 times that of the monolithic wall/frame. The ductility factor as measured by the ratio of the yield displacement to the displacement at failure of specimen C2005-I which had deformed bars for flexural reinforcements was twice as large as those of specimens with plain round bars (C2005-II and C2005-III). Degradation of lateral force resistance was not observed in any of the specimens up to a deflection angle of 1/100 rad. The displacement component plots (Figure 2.16), composed of measured shear displacement, flexural displacement, and slip at the interface between the girder and the infilled wall, showed that shear deformation accounted for 60-70% of total deflection up to a deflection angle of 1/150 rad. The remainder was mainly due to flexure deflection. As the specimens were further deformed, deflection due to shear gradually decreased while shear slip on the girder/wall interface increased until ultimate failure occurred. A lower strength observed in the infilled frames when compared with that of the monolithic wall/frame was attributed to pull-out failure of splice bars under combined tension and shear. The embedment depth of the splice bars had little affect on the strength and slippage at construction joints between the wall and the frame.

(6) The effect of number of connectors used in connecting steel infilled panels with concrete frames was investigated by Aoyama et al [2.23]. In this study, five one-bay, one-story concrete frames were infilled with 0.2 in. (4.5 mm) thick steel panels with 0.35 in. (9 mm) diameter headed studs which were welded to the rims of the steel panels. The 0.4 in. (10 mm) diameter resin anchors were inserted into predrilled holes in the concrete frames. The gap between the frame and the infilled steel panel was filled with non-shrink mortar. The number of connectors and location of openings in the steel panels were varied in each specimen. Dimensions of the test specimens are given in Figure 2.17. Specimen P-1-0 was made of welded steel plates 0.2 in. thick with no opening, and had 21 headed stud and 21 resin anchors. Specimen P-1-S had an opening at the center and twice the amount of shear studs and anchors. Specimen P-1-C had 59 welded shear studs and 63 resin anchors. The opening in this specimen was located at the top of the panel. Specimen P-2-G had 39 studs and 41 resin anchors. It had the opening located at the top of the panel. Specimen P-1-N had an infilled panel identical to specimen P-1-S (opening at center), except that the ratio of bending moment to shear force in this specimen was increased by changing the ratio of axial load to horizontal load. More longitudinal reinforcing bars were also provided for the columns of this specimens.

A constant axial load corresponding to the stress of 30 kg/cm^2 was applied on each column for specimen P-1-O, P-1-S, P-2-C and P-2G. For specimen P-1-N, the axial load N was changed in accordance with applied shear force Q ($N = 15.0 + 0.286$ Q ton). Reversed cyclic loads were applied at both ends of the girder. Hysteresis behavior and crack patterns for specimen P-1-O are shown in Figures 2.18 and 2.19, and the envelopes of the hysteresis curves of all five specimens are shown in Figure 2.20. The initial slopes of the hysteresis envelopes show that, prior to attaining ultimate load, the lateral stiffness of strengthened frames was not affected by either the location of the opening in the steel panel or the number of connectors. However, the complete hysteresis envelopes show that specimens P-1-O and P-1-S, which had a smaller number of connectors, had less lateral load resistance and ductility.

(7) The use of cast-in-place reinforced concrete infilled walls in strengthening portal steel frames has been studied by Makino, et al [2.5]. This test program, consisted of six portal steel frames of two types, A and B. Three type A specimens were frames with wide-flange steel columns oriented such that in-plane bending would be against the strong axes of the columns, and three type B specimens had columns oriented such that bending would be about their weak axes. Typical configurations for both types of specimens are shown in Figure 2.21.

Each type of specimen investigated in this program consisted of one original unstrengthened frame (specimen A0 or B0), one damaged and then strengthened with infilled concrete wall (specimen A1 or B1), and one undamaged frame infilled with concrete wall (specimen A2 or B2). For the repaired and the strengthened specimens, headed studs 0.5 in. (13 mm) in diameter and 4 in. (100 mm) long were welded onto the inner sides of the frames to provide connections between the frames and the cast-in-place infilled walls. The infilled concrete wall had about 5% flexural reinforcement in both directions and was cast vertically through delivery mouths provided under the top beams. However, local buckling in the steel columns of the repaired specimens was not repaired in the subsequent test as shown in Figure 2.22.

All specimens were subjected to a combination of constant axial load applied to the top of each column and reverse cyclic lateral load applied at the top of the test frame, except for two specimens B1 and B2 where only one-directional lateral load was applied instead. The constant axial load had a magnitude corresponding to 30% of the yield strength of the columns. The test results, in the form of hysteresis curves, are given in Figure 2.23 and summarized in Table 2.5. From these results, it was found that the damaged and then strengthened specimens, A1 and B1, had higher lateral force capacity but smaller ductilities than those of the undamaged infilled frames (specimens A2 and B2). Makino, et al concluded that the infilling technique can be effectively used in recovering the lateral force capacity of locally buckled portal steel frames. Further, they suggested that for estimation of the lateral force capacity of the frames, the infilled wall can be considered as a compressive bracing of rectangular cross section with an effective width of about 5.4 times the thickness.

(8) The effect of combined axial and lateral loads on the behaviors of steel frames with infilled walls was investigated by Mallick [2.6]. Tests were conducted on a series of two-bay steel frames infilled with cement mortar panels, with and without shear connectors. The combined loading, applied at the joints as shown in Figure 2.24, consisted of vertical (V) and lateral (H) loads, and moment produced by the vertical load with an eccentricity (e). The frames were loaded so that a rigid base condition could be achieved along the central member as shown in Figure 2.24. The vertical load and the associated moment were applied by prestressing a high strength steel rod 0.2 in. (5 mm) in diameter passing through the extended arm of the steel frame. The lateral load was applied gradually by a hydraulic jack. Mallick found that, in general, the infilled frames with shear connectors were laterally stiffer than those without shear connectors up to the failure load. Further, the frames behaved linearly until the first tension crack occurred in the

infilled panels; this corresponded to approximately 66% of the ultimate load. In the infilled frames without shear connectors, the failure was due mainly to the crushing of one of the loaded corners along with the separation cracks at the boundary junction. The stiffness of both types of frames were observed to be increased as the magnitude of the vertical load and the associated moment decreased.

2.2.2 Frame Strengthening By Steel Braces:

(1) Steel bracing was used by Jones and Jirsa [2.8, 2.9] to strengthen a damaged two-bay, two-story reinforced concrete frame. The frame was a 2/3-scale model of a portion of typical exterior moment resisting frames, representative of strong spandrel beam-weak column frames.

The strengthening scheme adopted in this study involved the attachment of structural steel diagonal bracings to the exterior of the damaged frame through use of 5/8 in-diameter (16 mm) standard threaded dowels. The dowels were epoxy grouted into the concrete frame at locations along the columns and the spandrel beams. The vertical channels (MC6x15.1) were bolted to the sides of the concrete columns by dowels placed at 8 in. (20 cm) intervals and embedded 5-1/2 in. (14 cm) deep. The horizontal collector sections (WT3x6) were connected to the spandrel beams by dowels embedded 4-1/2 in. (11.4 cm) deep at 9 in. (23 cm) interval in the first and the third floors and 18 in. intervals in the second floor. With the column channels and beam collectors in place, all threaded dowels were tightened to a uniform torque of 75 ft-lb (102 N-m). The layout of dowels is shown in Figure 2.25, and the connection details of brace members are shown in Figure 2.26.

The strengthened frame was subjected to five sets of load cycles corresponding to five different levels of drift. The load-drift relationship of the strengthened frame is shown in Figure 2.27, and the envelopes of the load-drift relationships are shown in Figure 2.28. The results of the test indicated that the lateral stiffness of the strengthened frame was approximately 1.5 times that of the bare frame.

However, it should be pointed out that this lateral stiffness was contributed mainly by the added steel frame, since the contribution of the existing concrete frame had been reduced by the cracking caused by previous tests. Further, the lateral force capacity of the steel bracing strengthened frame appeared to be 6 times higher than the lateral strength of the bare frame. This capacity was governed by buckling of brace members and the quality of the joints that connect bracing members. The epoxy-grouted dowels, observed the authors, performed well as evidenced by limited pullout and no dowel shear failure.

(2) Uses of in-plane steel braces in strengthening reinforced concrete frame were also studied by Sugano and Fujimura [2.2] and Higashi, Endo and Shimizu [2.3, 2.7] as mentioned in section 2.2.1. In the test series conducted by <u>Sugano and Fujimura</u> [2.2], compression braces of H-section steel and tension braces of 1.1 in. (28 mm) in diameter round plain bars were used to strengthen two frames, B-C and B-T (see Table 2.1). The compression braces in specimen B-C were attached to the frame at the beam/column corners by connecting bolts which were set on cover plates welded to the ends of the braces. The space between the frame and the steel cover plates was filled with mortar. The plain bar tension braces in specimen B-T were welded at each end with a connecting steel plate. This plate was in turn bolted by high-strength bolts of 0.6 in. (16 mm) in diameter to other plates that were anchored into the frame at each beam/column corner.

These braced frames were subjected to the same loading condition described in section 2.2.1, lateral reversed cyclic deformation in combination with axial stress of 13% the specified concrete strength. The test results are given in Table 2.2 and in Figures 2.4, 2.5 and 2.6.

From the test results, it appeared that in terms of lateral force capacity, both steel bracing schemes provided less increased capacity than infilled wall techniques. In frame B-C having compression braces, the lateral force capacity was measured about 3.5 times that of the unstrengthened frame (62% of that of the monolithic wall/frame system).

In frame B-T having tension braces, the lateral strength was 3.7 times that of the unstrengthened frame (66% of that of the monolithic wall/frame system). However, in terms of displacement ability and energy absorption, the specimen with tension braces (B-T) exhibited the most ductile behavior when compared with all other techniques.

(3) Higashi, Endo and Shimizu [2.3, 2.7], in their 1978 and 1979 test series, also experimented with steel bracing technique. In the 1978 series, three one-bay, one-story frames were strengthened using steel braces, a steel frame, and a steel truss (specimen No.7-SB, No.8-SF, and No.9-ST respectively). In the 1979 series, two one-bay three-story mortar frames, No.6-3SB and No.7-3SF, were strengthened by adding steel braces and a steel frame into all three stories of the frame (see Figure 2.7). All steel members were attached to frames through use of wedge anchors. The test results obtained from these specimens, shown in Figure 2.10 in the form of the envelopes of the hysteresis curves, indicated moderate increases in lateral stiffness and load capacity of the steel braced frames. However, the increase in ductility was substantial when compared with the infilled frames. The seismic capabilities of the specimens strengthened by steel bracing and steel frame appeared to be as large as that of the monolithic wall/frame, as indicated by the seismic capability index C_E in Table 2.4.

2.2.3 Frame Strengthening By Adding Wing Wall:

Addition of wingwalls is another strengthening technique which can provide reinforced concrete frames with increased lateral strength and ductility. This technique was used by Roach and Jirsa [2.9, 2.10] as an alternative strengthening scheme to the steel bracing scheme studied by Jones and Jirsa in the same testing program [2.8, 2.9]. The two-bay, two-story frame was strengthened by casting reinforced concrete wingwalls around the exterior three sides of the columns along their entire heights. The added wingwalls were designed in accordance with the ACI 318-83 code provisions [2.11] for structural walls and their widths were selected such that their nominal shear stress would be limited to about $4\sqrt{f'_c}$ (90 inches wide for the prototype structure or 60 inches for 2/3-

scale specimens). This selection was aimed at providing the strengthened frame with sufficient lateral stiffness to satisfy the code drift requirements without excessively reducing the window openings and to allow the spandrel beams to develop full flexural strength.

The newly cast wingwalls increased the column width from 12 inches (30 cm) to 60 inches (150 cm) and the thickness from 12 inches (30 cm) to 13.3 inches (34 cm) and were detailed to ensure monolithic behavior of the structural system as illustrated in Figure 2.29. Connections between the added wingwalls and the original frame included a coat of epoxy adhesive between the original and the new concrete and No.4 rebar dowels which were epoxy-grouted into the beam faces and the sides of the columns. Adhesive bond was increased by sandblasting the surface of the existing frame prior to application of epoxy.

The strengthened frame was subjected to four sets, three cycles each, of reversed cyclic deformation which corresponded to 0.05%, 0.125%, 0.25% and 0.5% drift. The results, presented in hysteresis curves and associated envelopes, are shown in Figures 2.30 and 2.31. The initial lateral stiffness of the strengthened frame was measured from the hysteresis curves as being 1250 K/in, which is more than three times that of the bare frame. The lateral capacity of the strengthened frame was limited by the flexural capacity of the spandrel beams rather than the shear capacity of the columns as in the case of the unstrengthened frame. The observed crack patterns indicated that flexural hinging had developed at all critical beam cross sections. In addition, the flexural reinforcements in the beams were yielding at ultimate and flexural cracks with widths of up to 1/4 in. (6 mm) were noted. In comparison, the columns with added wingwalls exhibited little distress. Maximum stresses in the longitudinal reinforcement in columns were only about one-half of the yield stress at ultimate loads. The measured lateral capacity of the frame with wingwalls indicated that an increase in strength of 5 times were obtained by this strengthening scheme.

2.3 REINFORCED CONCRETE AND MASONRY WALLS:

A limited amount of research on repair and strengthening of reinforced concrete and masonry walls is reported in the literature. Four such studies are presented in this section. The purpose of the study conducted by Corley, Fiorato, Oesterle and Scanlon [2.12] was to evaluate the effectiveness of repair techniques proposed for damaged structural concrete walls. The other studies, conducted by Plecnik, Cousins and O'Conner [2.13], Jabarov, Kozharinov and Lunyov [2.14], and Kahn [2.15], involved repair and strengthening of masonry walls by adding external reinforcement and concrete or mortar overlays.

(1) In the study conducted by <u>Corley, Fiorato, Oesterle and Scanlon</u> [2.12], three reinforced concrete walls with barbell-shaped cross section, named B5, B9 and B11, were loaded laterally as vertical cantilever with forces applied at the top. A constant axial force, corresponding to an axial stress of 545 psi, was maintained on the top of specimen B9. The dimensions of the walls and the reinforcement arrangements are shown in Figure 2.32. All three walls were loaded until significant loss in load carrying capacity occurred. The damaged walls were then repaired using three different techniques and retested to destruction. Detailed description of the repair techniques used for each wall is as follows:

In specimen B5, the damaged web concrete was removed up to the 8 ft-6 in. (2.6 m) level. The criterion used was to remove all concrete that could be taken out easily with hand tools. Reinforcing steel was left intact and no new reinforcement was added. New concrete of the same thickness of 4 in. (10 cm) was then cast in vertical lifts of 3 ft (0.9 m). The gap between the top portion of new concrete and old wall was handpacked with concrete. The repaired specimen was named B5R. Figures 2.33 and 2.34 show the damaged specimen B5R with web concrete removed and the same specimen after completion of repairs.

The same repair procedures were used to repair specimen B9 and the repaired wall was designated as B9R. However, rather than replacing the web at its original thickness of 4 in. (10 cm), a 6 in. (15 cm) web was constructed.

In specimen B11, not only was the damaged web concrete replaced using the same repair procedures as in specimens B5R and B9R, but additional diagonal reinforcement was also added. Prior to replacing the damaged web concrete, #5 diagonal rebars were inserted through the boundary columns at a 45^{0} angle and into predrilled holes in the base block. The bars were then epoxy grouted. Figures 2.35 and 2.36 show the drilling techniques for the diagonal holes and the arrangement of reinforcement in specimen B11R.

The repaired specimens were subjected to the same loading condition as the original specimen. The test results, presented along with the ACI code predictions for flexural and shear strengths of the walls, were summarized in Table 2.6. The hysteresis curves of all 6 specimens were given in Figure 2.37.

Examination of the results of specimens B5 and B5R indicated that replacement of the original damaged web concrete was an effective repair. Both the lateral strength and stiffness were successfully restored. For specimen B9R, replacing the original damaged web by a thickened web resulted in a significant increase in deformation capacity. The original specimen B9 was able to sustained only 1-1/2 inelastic cycles, with a maximum rotation corresponding to five times yield. However, the repaired specimen B9R was able to sustain 8-1/2 inelastic cycles, with a maximum rotation corresponding to six times the yield rotation in specimen B9. The ultimate strength of B9R was about equal to that of B9. For specimen B11R, the addition of diagonal reinforcement resulted in an increase in maximum nominal shear stress of approximately 18%. However, the initial stiffness of all three repaired walls were only approximately 50% of those of the original walls. This reduction in stiffness should be considered in the evaluation of dynamic response of repaired structures.

(2) The possibility for complete restoration of the initial strength and stiffness of damaged masonry walls by adding reinforced mortar layers was investigted by Jabarov,Kozharinov and Lunyov [2.14]. The test specimen was a two-story, one-bay reinforced masonry structure consisting of two parallel masonry walls. The walls were connected by cast-in-place reinforced concrete floor slabs of 3.9 in. (10 cm) thick. Each masonry wall was 23 ft (705 cm) long, 15 in. (38 cm) thick, and 18.4 ft (560 cm) high with two window openings of 4 ft by 5 ft (120 cm by 150 cm) in each story. The masonry blocks had ultimate compressive and bending strengths of 800 psi (5.5 MPa) and 450 psi (3.1 MPa), respectively, and the cement-lime mortar used to joint the masonry blocks had compressive strength of 725 psi (5 MPa). The test specimen was constructed on rigid foundation, its configuration and dimensions are as shown in Figures 2.38 (a) and (b).

The test wall was subjected to both axial and horizontal loads in combination with vibration generated by a vibro-machine placed on top of the wall. The axial load, corresponding to compressive stress in the wall of 20 psi (0.14 MPa) and 9 psi (0.065 MPa), was simulated by putting dead weights onto the floors of the first and second stories. The lateral load was applied incrementally, 13.5 kips (60 kN) per increment, through use of a hydraulic ram. The ram was placed at elevation of 16.3 ft (497 cm), which resulted in a lateral force ratio of 1:2.2 between the floors.

The original, unstrengthened wall was tested until a drop in lateral load capacity was observed. The ultimate lateral load was measured as 204 kips (910 kN). Extensive diagonal cracks appeared in all three piers of the first story at 158 kips (705 kN). In the first story, diagonal crack was observed in the middle pier at ultimate.

The damaged wall was then strengthened using two schemes. In the first scheme, four end piers of the first floor were strengthened by reinforced mortar layers 1 in. (2.5 cm) thick. Diagonal reinforcements were provided in groups of three 5 mm bars as shown in Figure 2.38 (c). The added reinforcement was attached to the masonry surface with the aid of 8 mm N-shaped steel bars which were anchored into the mortar joints. The second strengthening scheme was used only after the wall strengthened by the first scheme had been tested to failure. This second strengthening scheme involved application of mortar layers, reinforced by welded wire fabric, to the two middle piers of the first story (see Figure 2.38 (c)).

The wall strengthened by the first scheme was subjected to the same loading condition for the original wall and ultimate was 1.3 time the original wall. The wall, restrengthened by adding reinforced mortar layers in the middle piers, was able to sustain 97% of the ultimate lateral load of the original wall. Jabarov, Kozharinov, and Lunyov concluded that successful restoration of the initial strength and stiffness of damaged masonry walls can be achieved using the proposed strengthening techniques.

(3) A similar method of wall strengthening which used reinforced shotcrete as overlay was studied by Kahn [2.15]. In this study, fourteen 3 x 3 ft (1 x 1m) single wythe brick panels were constructed using bricks salvaged from the old Atlanta Civic Center, built in 1928, to simulate old, existing masonry walls. Nine of the fourteen brick panels were coated with a layer of 3.5 in. (89 mm) thick dry-mix shotcrete, and three were coated with a 1.5 in. (38 mm) thick shotcrete layer. The panels with 3.5 in. (89 mm) shotcrete were reinforced with welded wire fabric consisting of W4 wire six inches on center each way and placed 1 in. (25.4 mm) from the brick interface. The 1.5 in. (38 mm) shotcrete layer was reinforced with an expanded metal mesh which is commonly used for plaster surfaces, placed at 1/4 in. (6 mm) from the brick interface. Diagonal load tests, standardized by ASTM in the E519-74 specification,

were selected to determine the in-plane capacity of the strengthened brick panels in this test program.

Four surface conditions of the brick at the time of shotcreting were considered in order to study the surface effect on the brick-shotcrete bond. The surface of panels D1, D2, D3 were left dry; panels W1, W2, W3 and X1, X2, X3 were thoroughly wetted with water, and panels E1, E2, E3 were coated with epoxy adhesive (Sikastix 370 by Sika Chemical Corporation) about 10 minutes prior to shotcreting. Shotcrete was not applied to single wythe panel C1 or to double wythe panel CC1 to facilitate comparisons. The panels description and the test results are summarized in Table 2.7. The hysteresis curves for the strengthened panels are shown in Figure 2.39.

The static ultimate strengths listed in Table 2.7 indicated that shear capacity of brick panels was increased by approximately 17 times when strengthened by a 3.5 in. (89 mm) shotcrete layer (0.19 percent reinforcement each way by welded wire fabric). For panels strengthened with expanded metal mesh and 1.5 in. (38 mm) thick shotcrete, the shear capacity was increased by 6.8 times. It was also observed that composite action was fully developed between the brick and shotcrete regardless of surface condition. However, the epoxied and wetted surface panels displayed significantly greater inelastic deformation capacity than the dry panels. The ultimate load of the strengthened brick panels was dependent on the tensile resistance of the welded wire fabric and the expanded metal mesh. The modest amount of reinforcement used in this testing program was sufficient to permit post-cracking, inelastic deformation to develop in the panels.

(4) Plecnik, Cousins and O'Conner [2.13] proposed a different strengthening method for improving the out-of-plane and in-plane lateral load capacity of multi-wythe, unreinforced brick masonry walls. The proposed method involved coring a 2 in. to 4 in. (5.08-12.7 cm) diameter hole vertically through the wall to the foundation. A reinforcing bar would be placed in the core hole with filler material poured into the hole. The filler material could be epoxy, polyester, and cement grout.

Two experimental programs, one at the Long Beach State University (LBSU) and one at the North Carolina State University (NCSU), were conducted to examine the effectiveness of several strengthening methods and the influence of different parameters which affect the lateral load resistance capacity of unreinforced masonry walls.

In the LBSU testing program, over 70 small scale specimens were built as shown in Figure 2.40 and tested to failure under static shear load. All specimens were built using new, solid 8 in. common smooth face brick, manufactured per ASTM specification C62 for Grade SW brick. The mortar joint was prepared according to BIA-M1-72 except that in some cases the amounts of cement, lime and sand were varied from that specified for Type M mortar. Type M mortar was used in the majority of specimens, but several other types were used to determine the effect of mortar strength on the strength of the specimens. Reinforcement used included #4, #5, and #6 deformed bars of grade 60 steel, and 3/4 in. (19 mm) diameter undeformed fiberglass rods. The core at the center of each specimen was filled with either cement grout, a sand/polyester grout, or a sand/epoxy grout. Reference [2.13] described detailed mixing ratios, properties of filler materials and joint mortar.

The specimens were divided into 5 groups according to the type of core filler and the loading condition. Group A included 18 specimens filled with cement grout, Group B 8 specimens with a sand/polyester grout, and Group C 13 specimens filled with a sand/epoxy grout. Each specimen in Groups D and E was also subjected to an axial force in combination with the static shear load. Specimens in Group D were filled with cement grout and specimens in Group E were filled with sand/polyester grout. The test results of each group in Tables 2.8 (a) to (e).

A comparison of test results obtained from the five groups and of unstrengthened specimens indicated that inplane shear strength of the strengthened specimens can be increased by 55-110%. The test results further showed that the shear strength of brick masonry is significantly affected by the shear strength of the core material as well as the applied normal stress. In general, the specimens made with cement grout were approximately 30% weaker than the specimens made with sand/polyester or sand/epoxy grouts. The greater the resin content in the sand/epoxy grout, the greater the shear strength. Sand/polyester grouts were recommended over sand/epoxy grouts due to higher costs of epoxy adhesives. An optimum sand/polyester volume ratio was determined to be between 1:1 and 2:1.

In the NCSU testing program, three existing buildings were selected for strengthening using the technique evaluated in the LBSU test program. The three buildings numbered 3, 4, and 5, had three-wythes masonry walls and were classified as Type III (1982 Uniform Buildding Code) unreinforced masonry buildings with flexible wood diaphragms at roof and floor levels. For strengthening vertical holes of either 2 in. or 4 in. (5 to 10 cm)in diameter, were cored dry using a target electric drill rig. The holes were located at the center of the wall cross-sections and were filled with either a cement grout or a sand/epoxy grout after a No. 5 deformed bars had been inserted.

After determining the compressive strength of the brick and the shear strength of the mortar joint by in-place shear test (shove test), panels and prisms were cut out of the strengthened walls and transported to the laboratory for testing. All panels were three wythes thick (12 in.), seven courses high (21 in.) and 2-1/2 brick long (20 in.) with varying core size and filler material. The panels were subjected to both cyclic in-plane shear loads and cyclic out-of-plane moment. The results of 5 out-of-plane tests and 7 in-plane tests were summarized in Table 2.9 (a) and (b). Typical failure mode of out-of-plane tests consisted of horizontal cracks forming in the bed joints on the tension face of the specimens. This was followed by crushing of the mortar on the compression face. As the number of load cycles increased, the tension cracks increased in size and number while the mortar continued to crush. Compression failure of the bricks was not observed. For the in-plane shear tests, the first signs of failure of the specimens were cracking in the head and bed joints on either face of the specimens with some cracks going through bricks.

Plecnik, Cousins, and O'conner observed that both the out-of-plane moment capacity and the in-plane shear capacity of a strengthened masonry wall with larger core holes (4 in.) are generally greater than those of wall with smaller cored holes. This was because larger diameter cores allowed greater flow of the filler material into collar joints, therefore resulted in a larger effective area to resist both in-plane shear forces, and out-of-plane moments. Further, as in the LBSU's testing program, sand/epoxy was found to be superior to cement grout as a filler material because of their superior strength and flow characteristics.

2.4 REINFORCED CONCRETE COLUMNS:

Various methods for repair and strengthening have been used primarily to improve shear resistance of reinforced concrete columns to lateral loads. This section reports five major research studies conducted by Hayashi, Niwa and Fukuhara [2.1], Kahn [2.16], Bett, Klingner and Jirsa [2.17], Augusti, Focardi, Geiordano and Manzini [2.18], and Stoppenhagen and Jirsa [2.19]. In most of the above listed studies, more than one strengthening scheme was proposed and evaluated.

(1) In the study conducted by <u>Hayashi, Niwa and Fukuhara</u> [2.1], three of the four 1/2-scale reinforced concrete columns, 18 x 18 in. (45cm x 45cm) and 71 in. in height (180cm) were encased with welded wire fabric and mortar, and subjected to both constant axial stress of 40 kg/cm^2 and alternately reversed lateral loads. Three different strengthening schemes were proposed. In two specimens, designated as C-2 and C-3, the columns were encased with welded wire fabrics and 1.8 in. (4.5cm) thick mortar. In specimen C4, the column was strengthened by a thicker jacket of mortar of 3.5 in. (9.0 cm) along with welded wire fabrics. Specimen C1 was the unstrengthened original column. The objective of this study was to provide quantitative evaluation of the effectiveness of the proposed strengthening techniques for improving the

lateral load carrying capacity and the degree of improvement in ductility of the original column. The test specimens are as shown in Figure 2.41.

The hysteresis curves of each specimen and their envelopes are shown in Figure 2.42 and the experimental results were summarized in Table 2.10 along with the predicted results. It was observed that cracking developed in the original column C1 relatively early stage before the reinforcing bar yielded. Thereafter, as deflection increased, the lateral load capacity decreased rapidly to 53% of the ultimate lateral load at rotation angle of 1/50 rad. All strengthened specimens reached higher lateral load and did not show sign of deterioration in lateral load capacity until tensile reinforcement yielded. No reduction in the lateral load carrying capacity were observed in these specimens until the rotation angle reached 1/50 rad. The specimen strengthened with 3 in. (7.6 cm) thick mortar (C.4) reached its ultimate load capacity without rupture of mortar and welded wire fabric at the rotation angle of 1/25 rad. These results showed that both shear strength and ductility of concrete columns can be enhanced using the proposed techniques.

(2) Three different column strengthening techniques using steel bands, plain steel rods, and U-shaped steel clamps as external reinforcement were proposed by Kahn [2.16]. In this study, four specimens representing flexible columns connected to stiff girders were constructed with square cross-sectional areas of 39 in^2 (254 cm^2) and lengths of 11.5 ft (3.5 m). Specimen 1 was designed as the original specimen. It was tested to failure in shear due to a combination of constant axial load of 81 kips (360 kN) and reversed cyclic lateral deformations corresponding to 4 levels of drift, 0.5 ϑ, ϑ, 2 Δy, and 4 Δy, where Δy was the deflection level at which the main reinforcement yielded in tension. After testing, specimen 1 was repaired by removing damaged concrete and replacing buckled reinforcement. The buckled reinforcement was cut and welded to straight pieces of 0.3 in. (7.9 mm) thick, 2.0 in. by 2.0 in. (51 mm x 51 mm) steel angles. Hoops of 0.4 in. (9.5mm) reinforcing bars spaced at 1.5 in. (38 mm) on center were bent

around the repaired main reinforcement (Figure 2.43). The repaired specimen was designated as 1R.

Specimen 2 was strengthened prior to testing by strapping 2.0 in. wide (50.8 mm) packaging bands around the column. The bands were spaced 4.0 in. (102 mm) on center and secured with pressed clips (Figure 2.44). The 1/4 in. (6 mm) gap beneath the band hoops was packed with non-shrink mortar.

Specimen 3 was strengthened by wrapping a 1/4 in (6mm) plain steel rod around the column to form a rectangular spiral with a 1.1 in. (28mm) pitch as shown in Figure 2.45. The splices in the spiral were made by lap welding the bar. The 0.04 in. (1 mm) spaces beneath the spiral were filled with mortar.

Specimen 4 was strengthened by confining the concrete by U-shaped clamps which were held together by A325-3/4 inch bolts (19mm). These clamps were fabricated by welding 5/16 in x 2 in bar (7.9m x 50.8mm) to 3-1/8 in thick (79.4mm) steel angle (3 in x 5 in x 2.5 in wide) and spaced 4-1/4 in (108mm) on centers (Figure 2.46).

The hysteresis curves of all four specimens are shown in Figure 2.47. For the strengthened specimens, these curves were nearly identical in the elastic region. Further, the strengthened specimens demonstrated ductile response without reduction in shear capacity. The unstrengthened specimen collapsed due to loss of axial load capacity at a deflection of 1.5 in. (38mm), which represented a ductility ratio, $\Delta/\Delta y$, of 1.9. All strengthened specimens were able to achieve a ductility ratio of 4 with little deterioration in load carrying capacity. Kahn concluded that by confining the concrete using the proposed strengthening techniques so that shear force was carried primarily by the concrete, significant increases in ductility can be achieved.

(3) Strengthening of short reinforced concrete columns was studied by Bett, Klingner and Jirsa [2.17]. In their study, different techniques were used for repair and strengthening of three nearly full-scale reinforced concrete square columns. The specimens were numbered sequentially as 1-1, 1-2, 1-3 and the repaired specimen was designated as 1-1R. A constant axial compression of 64.8 kips (corresponding to a compressive stress of 450 psi) and a repeated reversed cycles of lateral deformation were applied to each specimen.

Each specimen in this testing program consisted of a column 36 in. in height (0.92m) and two large concrete end blocks. The original specimen 1-1 had eight #6 longitudinal rebars and special 1/4-in. (6 mm) deformed ties, placed at 8 in. (21 cm) intervals along the column height and covered with 1 in. (2.5 cm) of concrete. The configuration and dimensions of the specimens are shown in Figure 2.48.

In specimens 1-2 and 1-3, the strengthening methods involved sandblasting and encasing columns with a shotcrete jacket reinforced with closely-spaced transverse steel with and without crossties being cemented into the existing columns. Table 2.11 shows the descriptions of test specimens, including a brief summary of the strengthening technique used for each specimen.

For the repaired specimen (specimen 1-1R), the repair technique consisted of two operations. First, all loose concrete cover was removed with a chipping hammer, exposing the longitudinal steel. Holes were drilled through the column, and crossties which were used to anchor additional longitudinal steel, were inserted and cemented with epoxy. Second, closely-spaced ties were placed around the column core, and it was encased with shotcrete. The description of the specimen is given in Table 2.11.

The hysteresis curves for each test specimen and their envelopes are shown in Figures 2.49 and 2.50. The unstrengthened specimen 1-1 behaved elastically up to a story drift of 0.5%. Hysteresis loops remained stable up to 1% of the story drift. Failure in specimen 1-1 appeared to be dominated by shear as indicated by pinching shape of the hysteresis loops and strains of the longitudinal reinforcement which were significantly less than the yield strength. Both strengthened specimens (specimens 1-2 and 1-3) exhibited stable hysteresis behavior for deformations up to 1.5% of the story drift, after which a loss in stiffness became apparent. Failure of the strengthened specimens were primarily due to flexure as evidenced by the development of strains in the original column longitudinal reinforcement in excess of the yield strength. The envelopes of the hysteresis curves showed that the strengthened specimens achieved much greater lateral strength and stiffness than those of the original, unstrengthened specimen. The repaired specimen (1-1R) also exhibited stable hysteresis behavior for deformations up to 1.5% of the story drift. This specimen failed due to a combination of shear and flexure. The repaired specimen also achieved much greater lateral stiffness and strength than the original specimen. Further, the column repaired according to the proposed technique displayed lateral strength and stiffness that were nearly equal to those of an undamaged column strengthened with the shotcrete jacket.

(4) Comparative evaluations of different repair techniques for damaged reinforced concrete columns were conducted by Augusti, Focardi, Giordano and Manzini [2.18]. In this study, 24 full-size reinforced concrete columns were tested to failure under a combination of constant axial load and cyclic lateral deformations simulating strong earthquake induced motion. The specimens were then repaired by first removing the damaged concrete, and either straightening or replacing the buckled reinforcements by using different schemes. Rheoplastic cement mortar was then used to restore the damaged portion of the columns to the original dimensions. The reinforcement was repaired according to different procedures which can generally be described as follows:

1. Straighten bent or buckled longitudinal rebars by flame heating.
2. Straighten bent rebars and add stirrup ties.

3. Cut away portion of bent or buckled rebars and weld in replacements with and without adding stirrups. The added stirrups were either single or double hoop ties.

The different repair schemes are shown in Figure 2.51.

The experimental results showed the effectiveness of rheoplastic mortar for repair of concrete columns. Degradation of lateral stiffness and strength of the repaired columns was greatly reduced by the addition of extra stirrups, which helped to prevent buckling of longitudinal rebars. It was established that lateral strength and ductility of damaged reinforced concrete columns can be successfully restored by using the proposed repair schemes. A typical moment-displacement relationship of a specimen before and after repair is shown in Figure 2.52.

(5) Repaired columns in a concrete frame increase not only the lateral stiffness but also substantially change the failure mode of the frame. This latter aspect was investigated by <u>Stoppenhagen and Jirsa</u> [2.19]. The same 2/3-scale one-bay, two-story concrete frame tested by Jones and Jirsa [2.8] and Roach and Jirsa [2.10] was used in this testing program. The general description of the test specimen and the method of loading have been given in section 2.2.2 [2.8]. The column reinforcement details are as shown in Figure 2.53. Longitudinal reinforcing steel consisted of #8 Grade 60 bars. Details of shear reinforcements are shown in Figure 2.54.

The repair was begun by removing loose concrete from columns and holes were then drilled on each side of the columns and in the spandrel beams to accommodate the added stirrups. The longitudinal reinforcements were then added and tied to the stirrups, and concrete jackets were vertically cast over the existing columns. Bonding between the new and old concrete was improved by roughening the existing columns with an electric concrete hammer. The encased columns increased the depth of the original columns from 12 in. (30.5 cm) to 30 in. (76 cm) and the width from 12 in. (30.5 cm) to 20 in. (51 cm).

The load-drift relationships for the original undamaged frame and the frame with repaired columns are shown in Figure 2.55. Final crack patterns in the encased columns are shown in Figure 2.56. The crack patterns in the frame clearly indicated that the behavior of the frame was governed by a flexural failure mechanism in the spandrel beams. It was also evident from these crack patterns that a point of inflection developed in the columns. The majority of the flexural cracks in the columns were concentrated to the top and bottom of the beam-column joints, while the majority of the diagonal cracks occurred on the interior face of the columns. The strengthening of columns increased the strength of the frame against lateral load about five times that of the original frame, while the initial lateral stiffness of the frame was about the same as that of the original frame.

2.5 REINFORCED CONCRETE BEAMS:

While there are numerous reports on repair methods for damaged beams, a limited number of reports on strengthening of beams are found in open literature. Typically, the strengthening methodology involved the attachment of steel plates or steel bands to existing beams. The studies of Holman and Cook [2.20] and Vanek [2.21] are reported in this section.

(1) <u>Holman and Cook</u> [2.20] evaluated the effectiveness of externally bonded steel plates on flexural and torsional capacities of reinforced concrete beams. The test beams were designed to simulate a single bay spandrel beam with intermediate floor beams framing in at the third points as shown in Figure 2.57. This type of beam was usually subjected to a combination of flexural, shear and torsional loads. The test beams were 4 in. (100 mm) wide and 8 in. (200 mm) deep with an overall length of 10 ft-6 in. (3.2 m). To simulate floor beams framing into the beams, two beam stubs were also built into the beams at the third points. The beams were reinforced with two #6 deformed bars at top and bottom, and smooth wire of 3/16 in. diameter (5 mm) stirrups were used as web reinforcements. The beam cross-section is shown in Figure 2.58.

The test beams were divided into three groups. Group A beams, the control group, were tested to failure, and then the damaged beams were repaired by bonding steel plates to one side of each beam. This repaired set of beams was designated as Group C. Group B beams had plates attached prior to testing. Loads were applied through the stubs at locations 8 in. (20 cm) from the centerline of the beam to produce torsional load.

For attachment of the steel plates, the concrete surface was first brushed to remove all loose particles using a stiff bristle brush, and the steel plate surface was sanded to remove all mill scale. The plates, 8 in. (200 mm) by 36 in. (0.9 m) by 0.25 in. (6.4 mm) thick, were attached to one side of the beams (see Figure 2.57) using a two-part gel type epoxy that satisfied the requirements of ASTM C882 Type I.

The test results, presented in Figure 2.59 and in Table 2.12, indicate that both the strength and stiffness of the beams were enhanced by this strengthening technique. The repaired beams of Group C exhibited a gain in ultimate load of 25-40%, while an increase in strength of 35-45% was observed for the Group B beams. Further, this technique appeared to have altered the failure mode of these torsionally weak beams. The observed cracking patterns indicate that all repaired and strengthened beams (Groups B and C) appeared to have failed in flexure rather than torsion as in the cases of the Group A's beams. Strong structural bonding between the steel plate and the beam was noted.

(2) Vanek [2.21] investigated a method of improving the shear strength of reinforced concrete beams by using externally bonded steel plates. All specimens had cross sectional dimensions of 4 in. by 6 in. (100 mm by 150 mm) and had an overall length of 5 ft (1.5 m). The specimens initially had insufficient shear reinforcement. They were divided into three groups.

Group One was comprised of control specimens. Group Two consisted of beams strengthened by four glued-on steel bands on each side of the specimen. The bands were placed at an angle with the beam axis as shown in Figure 2.60. Structural bonding betwen the steel band and the concrete was accomplished by a combination of surface coating with epoxy adhesive and post-installed dowels. Two steel mandrels of 0.3 in. (8 mm) diameter, embedded in predrilled holes in the beams and grouted with either cement mortar or epoxy concrete, served as dowels for each steel band. Structural bonding between the steel band and the concrete in the Group three's beams were provided solely by epoxy adhesive.

All beams were centrally loaded by a static concentrated load. Crack patterns and load-deflection curves are shown in Figures 2.61 and 2.62. The test results show that the shear capacity of concrete beams can be increased by the bonded external band reinforcement. The strengthened beams with dowels, Group Two, and the strengthened beams without dowels, Group Three, showed increased shear capacity of 1.52 and 1.46 times of unstrengthened beams, Group One. A small increase in load carrying capacity of 5.6% was observed in beams where dowels were used in addition to surface coating with epoxy adhesive.

3. CASE HISTORIES OF REPAIR AND STRENGTHENING TECHNIQUES

3.1 INTRODUCTION:

As described in chapter 2, numerous techniques have been proposed for repair and strengthening of damaged and undamaged reinforced concrete and masonry structures. These techniques, while different in procedure, have common objectives:

1. to increase or restore strength, and
2. to increase or restore ductility of existing structural members or frames.

In general, reinforced concrete and masonry structural members are strengthened by adding new load carrying members to the existing structures. In this chapter, case histories where strengthening of existing structures or members was done using some of the techniques described in chapter 2 are presented. Reinforced concrete beams or concrete beam/column joints have been repaired and strengthened by epoxy-injection or by bonding external reinforcing steel bands or plates onto the concrete surfaces. Reinforced concrete frames have been strengthened either with infilled concrete or masonry walls, or with external steel braces or steel frames. Damaged or understrength reinforced concrete columns have been strengthened by replacing damaged rebars or adding new reinforcement and then encasing columns with concrete or mortar jacket. Reinforced concrete beams or concrete beam/column joints have been repaired and strengthened by epoxy-injection or by bonding external reinforcing steel bands or plates onto the concrete surfaces.

3.2 COMMON TECHNIQUES

3.2.1 Infill Walls:

Various techniques for infilling open reinforced concrete frames have been investigated [2.1,2.2,2.3,2.4,2.5,2.6]. In many cases, infilled walls are selected primarily to increase lateral stiffness and ductility of a frame for seismic strengthening. The infilled walls can either be cast-in-place concrete walls, precast concrete walls, steel panels, or concrete block walls. They are usually connected to the existing concrete frame through the use of metal shear connectors such as anchor bolts or, in some cases, epoxy-bonded concrete shear keys. These joining elements are mostly post-installed anchors and dowels inserted in pre-drilled holes with epoxy adhesives or epoxy mortar for grouting. When the number of joining elements are adequately provided, the lateral load carrying capacity of a frame with infilled wall can approach that of a monolithic frame-wall system. The general concepts of the infilling wall technique are illustrated in Figure 3.1.

Infilled walls were used to strengthen the Izumi High School after the 1978 Miyagi-Ken-Oki earthquake [3.1]. This three-story reinforced concrete frame building with a narrow rectangular-shaped plan did not have any shear walls in the transverse direction, and sustained severe damage to exterior columns. After repairing all damaged columns, shear walls were added to provide stiffness in the transverse direction. The following step-by step procedure was used to place cast-in-place infill walls:

1. Substrate mortar of columns on the side adjacent to walls was chipped off.
2. Deformed bars, used as dowels, were installed and grouted by epoxy adhesive into columns and beams above and below the walls (see Figure 3.2).
3. Wall reinforcement was placed and spliced to dowels and followed by placement of concrete.
4. Non-shrink mortar was pumped into the gaps between the walls and beams above.

A simple numerical analysis of the building estimated that the lateral load carrying capacity of the Izumi High School building after strengthening increased to 0.75 times of the weight of the building, 1.5 times higher than the original undamaged strength.

3.2.2 Steel Braces:

In addition to increasing lateral stiffness, steel braces are effective in increasing the ductility of concrete frames. Various schemes such as external steel frame bracing, in-plane compression cross bracing, and in-plane tension cross bracing have been investigated [2.2,2.3,2.7,2.8].

The main buildings of the Tohoku Institute of Technology in Sendai, Japan [3.2] were strengthened using specially designed steel braces after it sustained severe damage from the 1978 Miyagi-Ken-Oki earthquake. Steel cross braces were installed on both faces along the longitudinal direction of the eight-story reinforced concrete frame building. The braces were installed with an eccentricity from the surface of the exterior wall as shown in Figure 3.3.

This eight-story concrete frame building sustained shear and shear-bending failure of columns on the north side of the building due to lateral forces in the longitudinal direction. The column failures were caused by the weakness in the lateral stiffness of the building in the longitudinal direction and the influence of the infill spandrel walls. The cast-in-place infill spandrel walls (see Figure 3.4) increased the stiffness of the frame, thereby attracting large shear forces to the columns.

Strengthening of this damaged building consisted of repairing cracked columns, beams, walls and slabs, and adding new shear walls in the transverse direction. To reduce the adverse effect on the columns due to the infill spandrel walls, holes were drilled in these spandrel walls to weakened the walls. In the longitudinal direction, steel cross braces were attached to both faces of the building from the outside. The brace members were H-sections of weathering steel and were painted with a rust stablizing agent. The steel braces system was selected because:

1. It would not interupt natural lighting through windows,
2. Installation of braces would not disrupt use of the building,

3. Uniform distribution of braces could be selected to eliminate undesired concentration of shear forces.

The joining elements that provide the brace-to-frame connections were specially designed. Brace members were fastened by friction bolts to steel bases which were set against the reinforced concrete beam face. After filling the gap between the base and the concrete beam with cement mortar, the steel base was post-tensioned by prestressing steel rods inserted through bored holes in the concrete beams (see Figure 3.5).

3.2.3 WingWalls:

Adding wingwalls adjacent to existing columns is another technique of seismic strengthening of existing building. The wingwalls can either be cast-in-place or precast concrete walls. The cast-in-place wingwalls are monolithically cast over existing columns or frames. Surface coating of epoxy adhesives, in combination with dowels installed in the existing frame, can be used to provide composite action between the existing frame and the newly cast wingwall. Precast concrete wingwalls can be attached to the existing frame with dowel connectors. With adequate connections, cast-in-place wingwalls can provide as much lateral strength as the monolithic construction. The addition of precast wingwalls usually results in less strength but more ductility. The concepts of adding wingwalls are shown in Figure 3.6.

An example of the application of this technique for retrofitting a residential building in Tokyo is described in detail in Ref 3.3. A four-story building was strengthened to improve both the strength and stiffness of the longitudinal frames and to improve ductility of all columns. Cast-in-place wing walls were attached to columns by means of mechanical anchors (see Figure 3.7).

3.2.4 Encased Columns:

Encasing a column with a layer of reinforcement and concrete has been the most common method of strengthening reinforced concrete columns. This technique has been proven by laboratory experiments to be an effective method to increase strength and ductility of columns [2.1,2.16,2.17,2.19]. Additional reinforcement can be either vertical rebars with stirrups, welded wire fabric, or steel straps wound around the column. The column is then encased by a jacket of either shotcrete, in-situ concrete, or non-shrink mortar. A coat of epoxy adhesive can be applied on the surface of the existing column to provide better bonding.

Work by Nene [3.4] applied this technique to repair and strengthen damaged columns. The damaged columns were first stripped of all finishing materials and thoroughly washed with a powerful water jet. Vertical rebars were welded to the beam reinforcement connected to the column, and new #6 spiral reinforcement was then tied around the column. A coat of suitable epoxy grade was then applied prior to encasing each column with a 3-in (76-mm) thick jacket of in-situ concrete. The epoxy coat served two purposes: one was to provide structural bonding between the new and the old concrete; the second was to prevent the trapped moisture in the existing concrete from attacking the new concrete from inside. Figure 3.8 shows the details of the typical column repaired and strengthened by this technique.

3.2.5 Epoxy Bonded Reinforcement:

The steel-epoxy-concrete system has been applied for repair and strengthening of reinforced concrete beams. Experimental studies of using epoxy bonded steel plates for strengthening in shear and torsion have been presented in section 2.5. Insertion of conventional rebars in pre-drilled holes that intersect diagonal crack planes in beams with epoxy injection has proven to be successful.

The Kansas Department of Transportation has carried out shear strengthening of cracked concrete bridge girders using this technique [3.5]. The technique, known as "post-reinforcement", consists of the following steps:

1. Sealing of the surface of cracks using silicone sealant;
2. Vacuum drilling dust-free holes 6 in. apart and 45^{o} to the deck surface, thereby crossing crack planes at 90^{o} angle;
3. Filling the holes and crack plane with epoxy pumped under low pressure; and,
4. Inserting the reinforcing bars into the holes to span the crack by at least 18 in. (46 cm). Epoxy bonds the bar to the wall of the hole and fills the crack plane. The 6 in (15 cm) spacing between holes was selected to ensure that cracks would be intersected by at least one post-installed rebar.

This technique for repair and strengthening of concrete bridge deck beams and girders has been used to repair and strengthen over 20 bridges in Kansas since 1981.

An apartment building in Brussels, Belgium was damaged in 1982 due to gas explosion [3.5,3.6], and subsequently repaired using epoxy bonded steel plates. During the first phase of the explosion, the ceiling concrete slab was loaded upward by the overpressure, and during the next phase, it was loaded downward by underpressure. As a result, the concrete ceiling slab sustained large deflections, the maximum deflection was 2 in (51 mm). In addition, the concrete was severely damaged by fire.

For repair and strengthening, the slab was first lifted to the horizontal position. The damaged concrete was then removed to expose the clean, sound, concrete surface. Cracks were then filled by pressure injected epoxy resin prior to replacing the removed unsound concrete by epoxy mortar. Steel plates of 0.2 in. by 9.8 in. (0.5 cm by 25 cm) were bonded to the underside of the slab at approximately 2.5 ft (0.8 m) on centers. Epoxy mortar was used to cover the steel plates. The temporary support was removed after 7 days of epoxy curing, at which time the slab deflected about 0.2 to 0.3 in. (5 to 7 mm). These deflections corresponded to the calculated values.

Two possible drawbacks should be pointed out regarding the application of epoxy resin adhesives. Fire exposure tests of about 200 concrete beams [3.7], cracked by static concentrated load and repaired by epoxy injection, showed that both strength and stiffness of epoxy repaired beams reduced rapidly at uniform temperatures exceeding about 250^{o}F. This is because epoxy adhesives are organic thermosetting resin systems and thus are highly susceptible to softening and pyrolysis at elevated temperatures. Exposure tests of beams externally reinforced by epoxy-bonded steel reinforcement and left exposed to the environment for 2 years [3.8] showed that long term exposure in natural condition could lead to eventual reduction in strength of the exposed beams. The small overall reduction in strength of the two-years beams was attributed to the significant amount of corrosion of the steel plate at the steel/resin interface, which resulted in deterioration of bond between concrete and the external steel reinforcement. Corrosion appears to be due to migration of moisture from the concrete through the resin. Microcracking of the resin has also been observed in samples of broken pieces of resin extracted from the tested beams.

4. SUMMARY AND AREAS OF NEEDED RESEARCH

4.1 SUMMARY:

Numerous research studies on the repair and strengthening of different structural elements and frames, including reinforced concrete and steel frames, concrete and masonry walls, and concrete columns and beams, are reviewed in this report. The majority of these studies dealt exclusively with restoring or improving seismic resistance of concrete structures. For each type of structural elements, unique repair and strengthening techniques were proposed.

Strengthening of reinforced concrete or steel portal frames usually involves one of the following three techniques: infilled shear walls, steel bracing, or adding wingwalls. Increased lateral stiffness, ductility, and resistance to lateral loads are attained by these techniques. In terms of improving lateral load resistance, infilling the frame with walls was found to be most effective. The lateral load resistance of frame infilled with either cast-in-place or precast concrete walls could attain that of the monolithic wall/frame system when adequate connections between the existing frame and the infilled wall are provided.

The steel bracing technique can provide the concrete frames with a moderate increase in lateral stiffness and load resistance when compared with the infilling technique. However, the steel bracing technique is most suitable if the improvement in ductility is the primary concern. The effectiveness of the steel bracing technique, when represented in terms of the antiseismic cabability index C_E as defined in references, can be as large as that of monolithic wall/frame system.

Adding wingwalls provides the least increase in lateral stiffness and force capacity as observed in Higashi, Endo and Shimizu's study. However, this method can be successfully employed in altering the failure mechanism in a strong beam-weak column frame. The added lateral stiffness provided to the columns by cast-in-place wingwalls effectively changed the failure mechanism of a frame from shear failure in columns to flexural failure in spandrel beams.

Reinforced concrete or masonry walls can be strengthened by adding reinforced concrete or mortar overlays to the walls. The in-plane shear capacity of brick wall panels with reinforced shotcrete overlay can be increased substantially even with a moderate amount of reinforcement.

Both the out-of-plane and the in-plane lateral force capacities of masonry walls can be increased by filling vertical cored holes with rebars grouted with different types of filling materials. Large cored holes (4 in.-diameter) with sand/epoxy as filler material proved to be superior in providing masonry walls with increased lateral force capacity.

For reinforced concrete columns, many different encasing techniques were proven to be effective in providing concrete columns with added strength and ductility. Significant increase in ductility may be obtained using welded wire fabrics, while shear strength can be increased by confining the concrete core using steel bands or steel straps.

Strengthening of reinforced concrete beams usually requires bonding of external steel reinforcements such as steel plates, steel angles or deformed reinforcing rods to the existing beams. Both the shear and torsional strengths of reinforced concrete beams can be greatly increased using this technique. Repairing cracked concrete beams with reinforcing bars secured in concrete beam by epoxy-injection have shown to be effective in increasing the shear capacity.

4.2 AREAS OF NEEDED RESEARCH:

The success of strengthening concrete structures is critically dependent on the interaction between the new and existing elements. This interaction is provided by different types of post-installed shear connectors and/or epoxy adhesives. For frame or wall strengthening, shear connectors range from precast concrete shear keys to conventional mechanical or epoxy-grouted anchors.

Experiments have shown that when a sufficient number of connectors are used, the full capacity of the reinforcing elements can be mobilized [2.8]. On the other hand, when an insufficient number of connectors are used, a significant drop in the load carrying capacity of strengthened members was observed while no damage occurred to the reinforcing elements. This was due primarily to pullout or shear failure of connectors. These observations clearly demonstrate the importance of the relationship between the capacity of the joining elements and the overall capacity of the strengthened structures. For example, Sugano and Fujimura showed that the connection between the top beam and the infilled wall must have the shear strength of at least 10 kg/cm^2 in order to develop the lateral load capacity equal to 60 percent of that of the equivalent monolithic wall/frame system. To develop 100 percent, the connection must have the shear strength of 20 kg/cm^2. It is apparent that understanding this relationship is crucial both in estimating the capacity of a structure after strengthening and in developing an optimum strengthening scheme. A more thorough and systematic examination of this relationship is needed to develop guidelines for the design of strengthening methodologies. To develop such guidelines, knowledge on the following is essential:

1. Strength and deformation behavior of joining elements, including mechanical anchors, epoxy-grouted dowels and epoxy adhesives, in cracked and uncracked concrete. This can only be achieved through a comprehensive experimental program.
2. The relationship between the strength of anchors and the overall capacity of strengthened structures. The theoretical model should be developed with substantiation of data obtained from testing of anchors.
3. A quantitative assessment of the effectiveness of various strengthening techniques that are deemed practical for field application in improving the load carrying capacity and

ductility of reinforced concrete members and frames. Such assessment can be acquired through a comprehensive experimental program in strengthening methodologies. This assessment, coupled with knowledge on behavior of anchors, can be used to establish an empirical approach which will enable the designer to accurately predict the strength and behavior of a strengthened structure.

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List Of References By Topics:

o. Beam Strengthening :	2.20, 2.21, 3.5, 3.7, 3.8.
o. Column Strengthening:	2.1, 2.16, 2.18, 2.19, 3.4.
o. Wall Strengthening:	2.12, 2.13, 2.14, 2.15.
o. Frame Strengthening:	2.1, 2.2, 2.3, 2.4, 2.5, 2.6, 2.7, 2.8, 2.9, 2.10, 2.22, 2.23, 3.1, 3.2, 3.3.

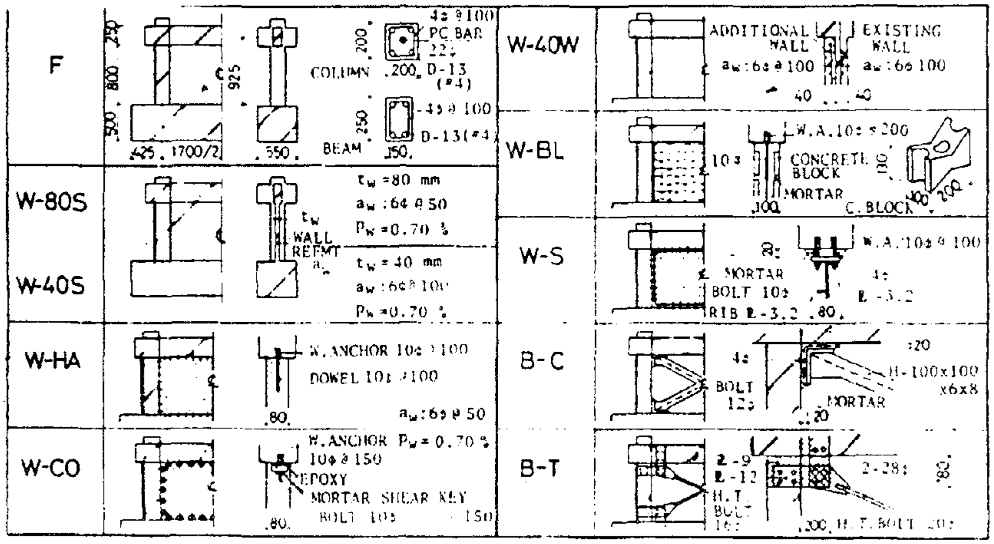


Table 2.1 Sugano and Fujimura's test specimens [2.2].

Specimen	Initial Stiffness *1	Load Capacity Qu *3 *1	*2	Displacement at Qu (x 10^{-3} rad.)	Ultimate Displacement (x 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.

Table 2.2 Summary of Sugano and Fujimura's test results [2.2].

1977 TEST SERIES	No. 1-F1	Bare, unstrengthened one-bay, one-story concrete frame.
	No. 2-PW	One-bay, one-story frame strengthened by cast-in-place infilled walls, connected by 9mm. dia. wedge anchors, spaced at 120mm. intervals along the top and bottom beams.
	No. 3-C3	Infilled by three precast concrete panels, connected to top and bottom beam by wedge anchors.
	No. 4-C3C	Infilled by three precast concrete panels with shear cotter, connected to top and bottom beam by wedge anchors.
1978 TEST SERIES	No. 1-F2	Unstrengthened one-bay, one-story reinforced concrete frame identical to specimen 77-1 (F1).
	No. 2-SP	Both columns were reinforced by adding steel plates on the two sides of the columns that are parallel to the frame plane. The steel plates were epoxy-bonded and bolted to columns by pairs of wedge anchors spaced at 90mm. intervals along the column height.
	No. 3-C2A	Partially infilled frame with two precast concrete walls, attached to frame at locations next to columns by wedge anchors.
	No. 4-C2B	Partially infilled frame with two precast concrete walls at the center, leaving two openings next to columns.
	No. 5-C4	Completely infilled frame with 4 precast concrete panels, connected to frame by wedge anchors.,
	No. 6-C40	Completely infilled frame with 4 precast concrete slit walls, also connected by wedge anchors.
	No. 7-SB	Strengthened by steel bracing with brace members bolted to steel sections that are anchored into frame by wedge anchors.
	No. 8-SF	Strengthened by steel frame
	No. 9-ST	Strengthened by steel truss placed at center of frame and anchored into top and bottom beam using wedge anchors.
	No. 10-FW	Monolithic wall/frame system.

Table 2.3 Descriptions of Higashi, Endo and Shimizu's frames [2.3,2.7].

1979 TEST SERIES	No. 1-3F	Three-story, one-bay reinforced mortar frame, unstrengthened.
	No. 2-3PW	Strengthened by infilling all three stories with cast-in-place concrete walls, 50mm. thick, and connected to frame using pairs of 6mm.-dia. wedge anchors placed at 40mm. intervals at top and bottom beams.
	No. 3-3C2A	Strengthened by partially infilling each story by two precast concrete panels at locations next to columns.
	No. 4-3C4	Strengthened by complete infilling each story by four precast concrete walls, anchored to top and bottom beams using pairs of 6mm. diameter.
	No. 5-3C40	Strengthened by infilling each story with four precast concrete slit walls, anchored to top and bottom beams by wedge anchors.
	No. 6-3SB	Strengthened by adding steel bracing into all three stories.
	No. 7-3SF	Strengthened by adding steel frame into all three stories.
	No. 8-3FW	Monolithic wall/frame system.
1981 .TEST SERIES	No. 1-3F2	Unstrengthened three-story, two-bay reinforced mortar frame.
	No. 2-3PW2	Strengthened by infilling all stories and all bays with 50mm. thick cast-in-place concrete walls. 6mm. dia. wedge anchors, spaced at 40mm. intervals were used in connecting the infilled walls to top and bottom beams of each story.
	No. 3-3C2A2	Strengthened by partial infilling each story and each bay by cast-in-place wing walls, connected to frame by 6mm. dia. wedge anchors, spaced at 40mm. intervals.
	No. 4-3FW2	Monolithic three-story, three-bay wall/frame system, with wall reinforcement anchored into frame.

SPECIMEN		77 series				78 series									
		No. 1	No. 2	No. 3	No. 4	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6	No. 7	No. 8	No. 9	No.10
INITIAL STIFFNESS $\times 10^5$ N/cm ($\frac{ton}{cm}$)		2.18 (22.2)	44.6 (445.0)	13.3 (136.0)	18.8 (192.0)	2.53 (25.8)	2.99 (30.5)	13.9 (141.3)	7.82 (79.7)	48.3 (492.5)	14.1 (144.1)	7.27 (74.1)	4.29 (43.7)	3.68 (37.5)	58.8 (600.0)
YIELDING POINT	load $\times 10^4$ N Q_y (ton)	9.4 (9.6)	39.2 (40.0)	19.6 (20.0)	43.1 (44.0)	9.1 (9.3)	9.8 (10.0)	12.6 (12.8)	9.3 (9.5)	39.0 (39.8)	9.6 (9.8)	17.3 (17.6)	16.1 (16.4)	14.5 (14.8)	49.0 (50.0)
	deflection δ_y (cm)	1.15	0.47	0.45	0.86	0.96	1.00	0.50	0.50	0.50	0.20	0.48	0.76	0.77	0.51
MAXIMUM	load $\times 10^4$ N Q_{max} (ton)	10.5 (10.7)	39.2 (40.0)	32.4 (33.0)	45.1 (46.0)	10.9 (11.1)	11.5 (11.7)	15.4 (15.7)	14.2 (14.5)	39.2 (40.0)	15.7 (16.0)	25.6 (26.1)	25.7 (26.2)	18.2 (18.6)	56.9 (58.0)
	deflection δ_m (cm)	1.65	0.47	1.89	1.03	1.95	2.00	2.00	2.00	0.73	2.00	3.54	4.00	1.97	0.87
ULTIMATE	load $\times 10^4$ N Q_u (ton)	10.5 (10.7)	35.3 (36.0)	32.4 (33.0)	45.1 (46.0)	10.9 (11.1)	11.4 (11.6)	13.8 (14.1)	12.9 (13.2)	39.2 (40.0)	15.1 (15.4)	25.6 (26.1)	24.9 (25.4)	18.2 (18.6)	56.9 (58.0)
	deflection δ_u (cm)	1.65	0.73	1.89	1.03	1.95	3.52	3.35	2.56	0.73	4.00	3.54	5.50	1.97	0.87
C_E		0.61	2.18	3.74	2.26	0.81	1.19	2.07	1.67	2.31	4.01	4.03	3.88	1.57	3.75

(a)

Specimen		79 Series							
		No. 1	No. 2	No. 3	No. 4	No. 5	No. 6	No. 7	No. 8
Initial Stiffness $\times 10^5$ N/cm (ton/cm)		0.59 (6)	14.71 (150)	1.27 (13)	4.41 (45)	5.39 (55)	4.41 (45)	3.43 (35)	15.20 (155)
YIELDING POINT	load $\times 10^4$ N Q_y (ton)	1.03 (1.05)	7.44 (7.59)	1.44 (1.47)	4.18 (4.26)	3.38 (3.45)	6.03 (6.15)	4.71 (4.80)	7.44 (7.59)
	deflection δ_y mm	2.2	2.3	2.1	5.2	2.0	4.4	4.4	2.9
MAXIMUM	load $\times 10^4$ N Q_{max} (ton)	1.91 (1.95)	8.47 (8.64)	2.97 (3.03)	5.74 (5.85)	5.12 (5.22)	7.80 (7.95)	5.97 (6.09)	8.47 (8.64)
	deflection δ_m mm	9.4	7.0	16.8	21.2	14.9	12.0	18.8	7.0
ULTIMATE	load $\times 10^4$ N Q_u (ton)	1.91 (1.95)	8.15 (8.31)	--	5.74 (5.85)	5.00 (5.10)	7.65 (7.80)	--	8.12 (8.28)
	deflection δ_u mm	9.4	9.8	20.0	21.2	17.8	15.1	20.0	9.0
C_E		0.89	3.80	2.15	2.61	3.48	3.15	2.89	3.15

(b)

Table 2.4 Summary of Higashi, Endo and Shimizu's test results [2.3,2.7].
a) 1977,78 series.
b) 1979 series.
c) 1981 series.

Specimen	81-No.1		79	81-No.2		79	81-No.3		79	81-No.4		79
	P	N	No.1	P	N	No.2	P	N	No.3	P	N	No.6
Initial Stiffness Ke (ton/cm)	37		6	250		150	63		13	273		155
Yield load Py (ton)	2.6		1.1	12.8		7.6	5.2		1.5	13.6		7.6
Yield deflection δy (mm)	3.5		2.2	3.5		2.3	4.0		2.1	3.5		2.9
Maximum load Pm (ton)	3.4	3.4	2.0	12.8	11.1	8.6	6.7	6.4	3.0	14.0	11.2	8.6
Maximum deflection δm (mm)	15.1	7.0	9.4	5.3	6.8	7.0	11.1	7.0	16.8	5.3	3.5	7.0
Ultimate load Pu (ton)	-			12.0		8.3	6.4		-	13.7		8.3
Ultimate deflection δu (mm)	-			7.0		9.8	12.6		-	6.9		9.0

N.B. P ; positive loading
N ; negative loading

(c)

Specimens	Q_i (ton)	R_i ($\times 10^{-3}$)	Q_u (ton)	R_u ($\times 10^{-3}$)
A0				
A1	10.6	1.34	44.4	13
A2	12.4	1.03	34.9	13
B0				
B1	17.8	2.10	33.0	12
B2	15.0	1.60	31.4	30

Q_i = initial crack load
Q_u = ultimate load
R_i = lateral displacement to height ratio at initial crack load
R_u = lateral displacement to height ratio at ultimate load

Table 2.5 Summary of Makino's test results [2.5].

Specimen	Confined Boundary Element	Axial Load psi	ACI Design				Full Yield Load					Maximum Load						Failure Mode(4)
			Flexure		Shear		Calculated(2)		Observed		Obs./Calc.	Calculated(2)		Observed		Obs./Calc.	Obs./ACI(3)	
			kips	$\sqrt{f'_c}$ (1)	kips	$\sqrt{f'_c}$	kips	$\sqrt{f'_c}$	kips	$\sqrt{f'_c}$		kips	$\sqrt{f'_c}$	kips	$\sqrt{f'_c}$			
B5	Yes	—	129	6.6	127	6.5	123.1	6.3	138.0	7.1	1.12	213.7	11.0	171.3	8.8	0.80	1.33	WC
B5R	Yes	—	129	6.8	127	6.7	123.1	6.5	—	—	—	213.7	11.5	167.8	8.9	0.79	1.30	WC
B9	Yes	545	173	9.0	148	7.7	165.6	8.6	186.4	9.7	1.13	241.6	12.6	219.6	11.4	0.91	1.48	WC
B9R	Yes	450	173	5.6	162	5.2	165.6	5.3	—	—	—	241.6	7.7	218.7	7.0	0.91	1.35	WC
B11	Yes	—	129	6.1	127	6.0	122.4	5.8	141.7	6.7	1.16	210.1	9.9	163.3	7.7	0.78	1.29	WC
B11R	Yes	—	129	6.8	180	9.5	122.4	6.5	—	—	—	210.1	11.1	171.0	9.1	0.81	1.33	WC

(1) Lateral load in terms of nominal shear stress $v = \frac{V}{0.8 \ell_w b \sqrt{f'_c}}$ (psi)
(2) Calculated monotonic flexural strength from analysis based on strain compatibility using measured material properties including strain hardening *of reinforcement.*
(3) ACI taken as the lower of flexure or shear design strength with capacity reduction factor $\phi = 1.0$.
(4) WC = Web Crushing
1 kip = 4.448 kN, $10\sqrt{f'_c}$ (psi) = $0.08304\sqrt{f'_c}$ (MPa)

Table 2.6 Summary of Corley et al's test results [2.12].

Specimen	Surface condition	Shotcrete thickness (in.)*	Reinforcement	Cracking load on diagonal† (kips)*	Ultimate load on diagonal† (kips)*
C1	—	—	—	6.2 0	6.2 0
CC1	—	—	—	18.0 0	18.0 0
D1	dry	3.1	wwf‡	87.4 60.2	121.8 84.2
D2	dry	3.8	wwf	83.9 61.7	147.6 82.2
D3	dry	3.4	wwf	92.3 72.8	131.9 111.6
E1	epoxy	3.1	wwf	76.6 58.8	121.8 103.5
E2	epoxy	3.8	wwf	75.7 87.4	133.9 104.5
E3	epoxy	3.8	wwf	79.1 45.5	138.0 100.4
W1	wet	3.4	wwf	76.5 83.2	142.0 85.3
W2	wet	3.9	wwf	82.2 38.1	148.1 106.5
W3	wet	3.5	wwf	87.8 58.1	150.2 87.3
X1	wet	1.5	exp. met.§	65.0 22.5	67.0 69.0
X2	wet	1.5	exp. met	59.0 39.5	61.0 47.5
X3	wet	1.4	exp. met.	50.0 22.0	52.0 41.0

*1 in. = 25 mm, 1 kip = 4.5 kN.
†Top value indicates load across original diagonal. Lower value is the reversed cycle load across second diagonal.
‡WWF 6 x 6 — W4 x W4.
§Expanded metal.

Table 2.7 Kahn's brick masonry test results [2.15].

Sample number (1)	Type of grout[a] (2)	Type of mortar[a] (3)	Shear stress over gross area (lbs/sq in.) (4)	Average for group (lbs/sq in.) (5)
1	1, 1/10, 3	1, 1/4, 3	171.1	—
2	1, 1/10, 3	1, 1/4, 3	153.1	—
3	1, 1/10, 3	1, 1/4, 3	142.2	155.7
4	1, 1/4, 3	1, 1/4, 3	93.8	—
5	1, 1/4, 3	1, 1/4, 3	78.1	—
6	1, 1/4, 3	1, 1/4, 3	185.9	—
7	1, 1/4, 3	1, 1/4, 3	150.8	—
8	1, 1/4, 3	1, 1/4, 3	107.0	123.1
9	1, 1/8, 1.5	1, 1/4, 3	160.9	—
10	1, 1/8, 1.5	1, 1/4, 3	56.3	—
11	1, 1/8, 1.5	1, 1/4, 3	56.3	—
12	1, 1/8, 1.5	1, 1/4, 3	127.3	—
13	1, 1/8, 1.5	1, 1/4, 3	213.3	122.8
14	1, 1/8, 1	1, 1/4, 3	160.9	—
15	1, 1/8, 1	1, 1/4, 3	160.9	—
16	1, 1/8, 1	1, 1/4, 3	112.5	—
17	1, 1/8, 1	1, 1/4, 3	150.8	—
18	1, 1/8, 1	1, 1/4, 3	246.1	166.2

[a]Cement, lime, sand proportions by volume.
Note: 1 psi = 6.89 kN/m².

Table 2.8a Shear strength of cement grout specimen [2.13].

Sample number (1)	Sand/polyester proportions[a] (2)	Type of mortar[b] (3)	Shear stress over gross area (lbs/sq in.) (4)	Average for group (lbs/sq in.) (5)
1	2/1	1/6, 1/4, 3	350.0	—
2	2/1	1/6, 1/4, 3	262.5	—
3	2/1	1/6, 1/4, 3	306.3	—
4	2/1	1/6, 1/4, 3	181.3	275.0
5	2/1	1, 1/4, 3	160.9	—
6	2/1	1, 1/4, 3	204.7	—
7	2/1	1, 1/4, 3	160.9	—
8	2/1	1, 1/4, 3	204.7	182.8

[a]Polyester/sand proportions by volume.
[b]Cement, lime, sand proportions by volume.
Note: 1 psi = 6.89 kN/m².

Table 2.8b Shear strength of sand/polyester grout specimens [2.13].

Sample number (1)	Type of epoxy (2)	Type of mortar[a] (3)	Shear stress over gross area (lbs/sq in.) (4)	Average for group (lbs/sq in.) (5)
1	I	1, 1/4, 3	191.4	—
2	I	1, 1/4, 3	246.1	218.8
3	II	1, 1/4, 3	196.3	—
4	II	1, 1/4, 3	171.1	183.7
5	III	1/9, 1/12, 3	256.3	—
6	III	1/6, 1/4, 3	150.8	—
7	III	1/6, 1/4, 3	246.1	—
8	III	1/6, 1/4, 3	246.1	—
9	III	1/3, 1/4, 3	171.1	214.1
10	IV	1/9, 1/12, 3	246.1	246.1
11	V	1/9, 1/12, 3	204.7	—
12	V	1/9, 1/12, 3	204.7	—
13	V	1/9, 1/12, 3	137.5	182.3

[a]Cement, lime, sand proportions by volume.
Note: 1 psi = 6.89 kN/m².

Table 2.8c Shear strength of sand/epoxy specimens [2.13].

Sample number (1)	Type of grout[a] (2)	Type of mortar[a] (3)	Shear stress over gross area (lbs/sq in.) (4)	Average for group (lbs/sq in.) (5)	Normal stress (lbs/sq in.) (6)
1	1, 1/8, 3	1, 1/4, 3	223.4	—	29.9
2	1, 1/8, 3	1, 1/4, 3	500.0	—	62.5
3	1, 1/8, 3	1, 1/4, 3	350.00	—	43.8
4	1, 1/8, 3	1, 1/4, 3	350.0	—	43.8
5	1, 1/8, 3	1, 1/4, 3	306.3	—	38.3
6	1, 1/8, 3	1, 1/4, 3	350.0	—	43.8
7	1, 1/8, 3	1, 1/4, 3	350.0	—	43.8
8	1, 1/8, 3	1, 1/4, 3	350.0	—	43.8
9	1, 1/8, 3	1, 1/4, 3	262.5	—	32.8
10	1, 1/8, 3	1, 1/4, 3	350.0	—	43.8
11	1, 1/8, 3	1, 1/4, 3	262.5	—	32.8
12	1, 1/8, 3	1, 1/4, 3	434.4	—	54.3
13	1, 1/8, 3	1, 1/4, 3	306.3	—	38.3
14	1, 1/8, 3	1, 1/4, 3	284.4	284.3	35.6

[a]Cement, lime, sand ratio by volume.
Note: 1 psi = 6.89 kN/m².

Table 2.8d Shear strength of specimens with cement grout and axial force [2.13].

Sample number (1)	Sand/polyester proportion[a] (2)	Type of mortar[b] (3)	Shear stress over gross area (lbs/sq in.) (4)	Average for group (lbs/sq in.) (5)	Normal stress (lbs/sq in.) (6)
1	3/1	1, 1/4, 3	331.3	—	21.9
2	3/1	1, 1/4, 3	331.3	—	39.1
3	3/1	1, 1/4, 3	296.9	—	39.1
4	3/1	1, 1/4, 3	331.3	—	39.1
5	3/1	1, 1/4, 3	243.8	306.9	
6	2/1	1, 1/4, 3	362.5	—	30.0
7	2/1	1, 1/4, 3	362.5	—	54.7
8	2/1	1, 1/4, 3	453.1	—	51.6
9	2/1	1, 1/4, 3	465.6	—	51.6
10	2/1	1, 1/4, 3	351.6	—	51.6
11	2/1	1, 1/4, 3	304.7	—	51.6
12	2/1	1, 1/4, 3	267.2	—	51.6
13	2/1	1, 1/4, 3	414.1	—	54.7
14	2/1	1, 1/4, 3	414.1	—	54.7
15	2/1	1, 1/4, 3	455.5	—	54.7
16	2/1	1, 1/4, 3	414.1	387.7	54.7

[a]Sand/polyester ratio by volume.
[b]Cement, lime, sand ratio by volume.
Note: 1 psi = 6.89 kN/m².

Table 2.8e Shear strength of specimens with sand/polyester grout and axial force [2.13].

Building number and specimen number (1)	Core size and type (in.) (2)	Gross cross-sectional area (sq in.) (3)	Maximum load (lbs) (4)	Maximum moment (in./lb) (5)	Cycle of maximum load (6)
4D	2-Grout	300	2,260	70,100	1
5M	2-Grout	273	2,800	85,400	1
5D	2-Grout	260	2,400	68,400	1
3E	4-Epoxy	273	2,450	66,200	4
4A	2-Epoxy	260	2,500	123,800	6

Note: 1 in. = 2.54 cm; 1 lb = 0.453 kg.

Table 2.9a Results of out-of-plane tests [2.13].

Building number and specimen number (1)	Core size and type (2)	Gross cross-sectional area (sq in.) (3)	Maximum load (lbs) (4)	Maximum shear stress (lbs/sq in.) (5)	Cycle of maximum load (6)
4B	2-Grout	308	7,895	25.6	1
3E	4-Epoxy	252	27,600	109.5	1
5C	2-Grout	246	25,000	101.6	1
5L	2-Epoxy	252	12,500	49.6	1
3C	4-Grout	240	20,400	85.0	1
3M	4-Grout	252	15,100	59.9	1
4H	2-Epoxy	280	13,200	47.1	5

Note: 1 psi = 6.89 kN/m^2; 1 lb = 0.453 kg; and 1 in. = 2.54 cm.

Table 2.9b Results of in-plane tests [2.13].

	initial stiffness (1) TON/cm²		bending cracking load (2) TON		shear cracking load (3) TON		yield strength of F function (4) TON	ultimate strength of F function (5) TON	ultimate shear strength TON	ultimate strength TON
	(1)	(2)	(1)	(2)	(1)	(2)			(1) (6)	(2)
C-1	164.5	69.9	12.25	15.0	25.8	25.8	33.82	36.16	30.9	30.5
C-2	278.0	109.7	11.90	15.0	33.1	30.8	33.35	35.36	46.7	41.6
C-3	278.0	100.0	11.90	15.0	33.1	31.5	33.35	35.36	46.7	42.6
C-4	497.4	93.8	12.35	15.0	42.0	32.5	34.05	36.11	57.5	39.1
C-5	344.8	79.5	12.35	15.0	32.6	—	34.05	36.11	43.8	37.1

Calculated values(1) and experimental results(2)

Note:

(1) $K_1 = 1/\left\{2\int_{0}^{a}\left(\frac{x^2}{cEI_e}+\frac{K}{GA}\right)dx + 2\int_{a_1}^{a_2}\left(\frac{x^2}{cEI_e}+\frac{K}{GA}\right)dx\right\}$

(2) $Q_{bc} = \left(1.8\,F_c \cdot Z_e + \frac{ND}{6}\right)/\left(\frac{l}{2}\right)$

(3) $Q_{sc} = 0.055 + 0.15N/3D \cdot {}_c\sigma_b$

(4).(5) e-Function Method

(6) $Q_{su} = \left\{\frac{0.053 p_t^{0.23}(180+F_c)}{M/QD+0.12} + 2.7\sqrt{p_w \cdot {}_s\sigma_{wy}} + 0.1\sigma_0\right\} \times b \cdot j$

Fc: concrete strength

Table 2.10 Summary of test results [2.1]
(1) Predicted results.
(2) Experimentally obtained results.

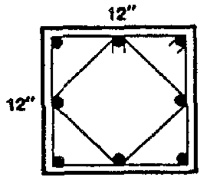
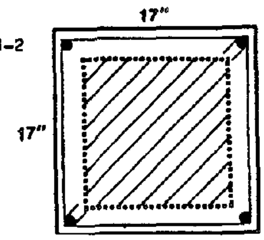
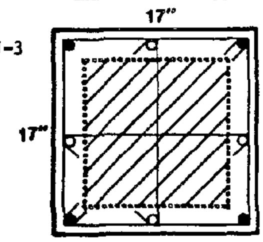
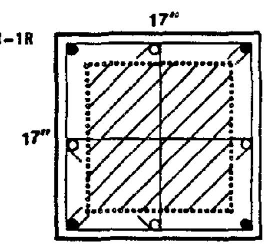
Test No.	Specimen No.		Column Cross Section	Test Description
1	1-1		Long. steel: #6's Ties: 6mm @ 8" Identical 12" x 12" cores used for all specimens.	Description: original specimen Test: reversed unidirectional loading Deflection limit: 2% drift
2	1-2		Long. steel: #3's Ties: 6mm @ 2-1/2" 2-1/2" shotcrete shell	Description: sandblast, add 4-#3 longitudinal bars and 6mm ties at 2-1/2". Shotcrete to 17" x 17". Test: reversed unidirectional loading. Deflection limit: 2.5% drift
3	1-3		Long: #3's, #6's Ties: 6mm @ 2-1/2" Crossties: #3's @ 9" 2-1/2" shotcrete shell	Description: sandblast, add 4-#3 corner bars and 4-#6 midface bars. Anchor midface bars w/#3 crossties, secured with epoxy. 6mm ties @ 2-1/2". Shotcrete to 17" x 17". Test: reversed unidirectional loading. Deflection limit: 2.5% drift.
4	1-1R		Long: #3's, #6's Ties: 6mm @ 2-1/2" Crossties: #3's @ 9" 2-1/2" shotcrete shell	Description: Remove all loose cover. Add 4-#3 corner bars and 4-#6 midface bars. Anchor midface bars w/#3 crossties, secured with epoxy. 6mm ties @ 2-1/2". Shotcrete to 17" x 17". Test: reversed unidirectional loading. Deflection limit: 2.5% drift.

Table 2.11 Descriptions of Bett, Klingner and Jirsa's test specimen [2.17].

Beam (1)	Load, in kips (kilonewtons) (2)	Average load, in kips (kilonewtons) (3)	Calculated load, in kips (kilonewtons) (4)
A1	7.0 (31.2)	7.0 (31.2)	6.0 (26.7)
A2	7.0 (31.2)		
A3	7.0 (31.2)		
B1	9.5 (42.3)	10.0 (44.5)	8.3 (36.8)
B2	10.0 (44.5)		
B3	10.5 (46.7)		
C1	8.0 (35.6)	9.3 (41.3)	8.3 (36.8)
C2	11.0 (49.0)		
C3	8.8 (39.2)		

Table 2.12 Holman and Cook's test results.

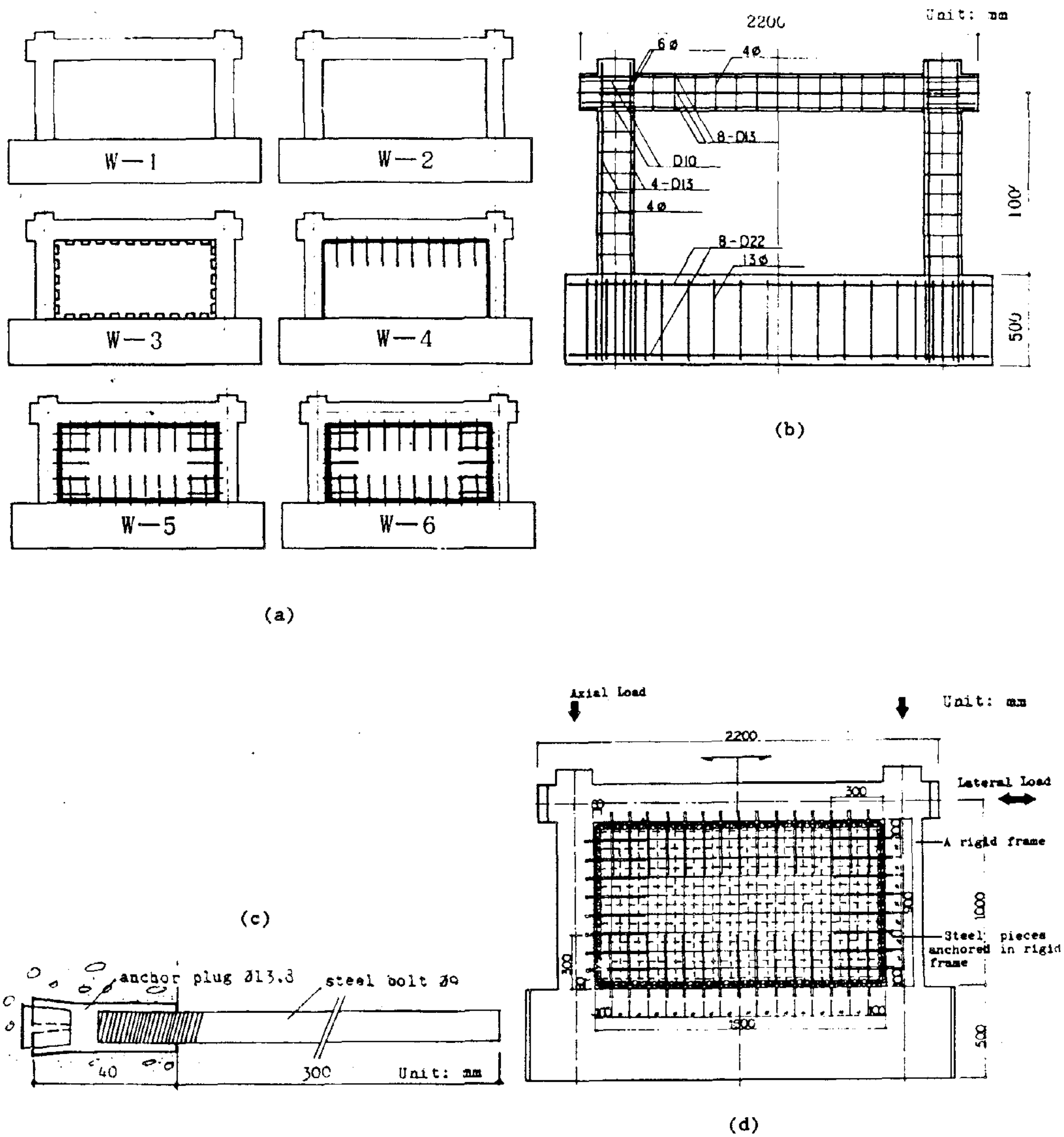


Figure 2.1 Hayashi, Niwa and Fukuhara's frame specimens [2.1].
a) Specimen configuration and strengthening concepts.
b) Reinforcement arrangement.
c) Sleeve expansion anchor used as joining element.
d) Reinforcing details of infilled wall.

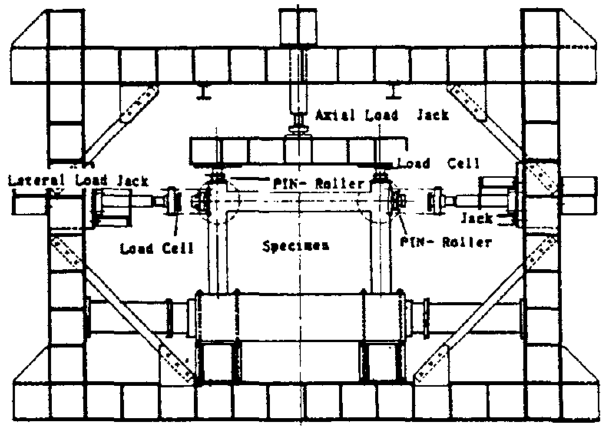
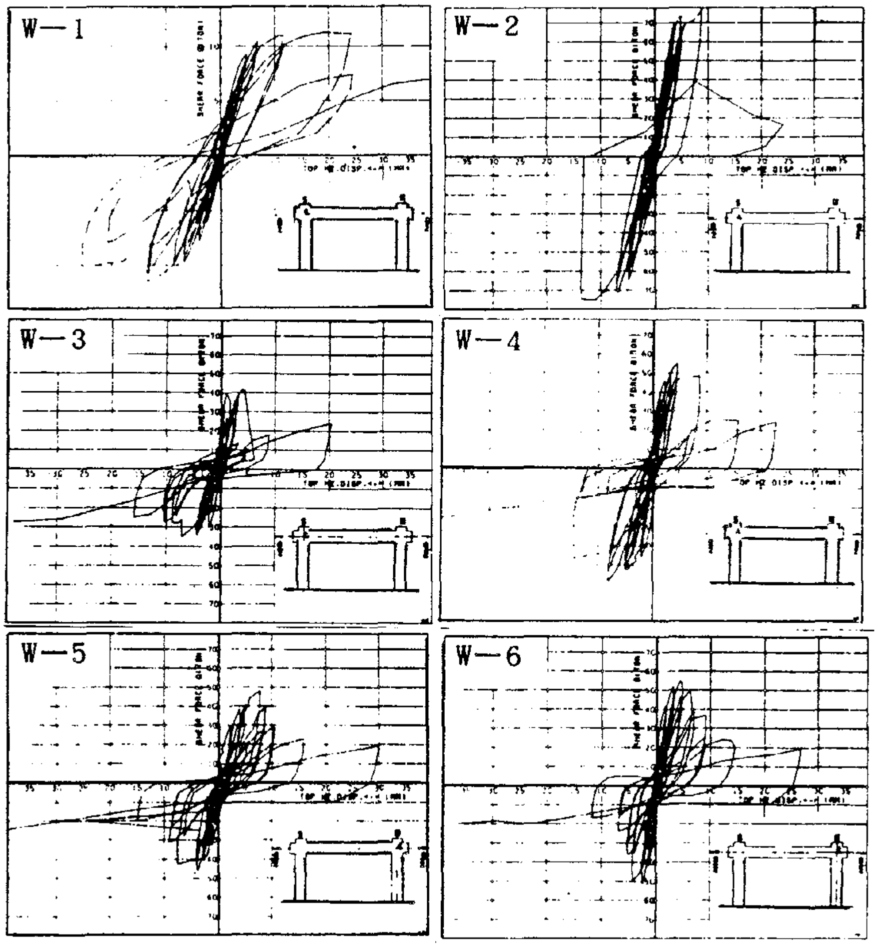
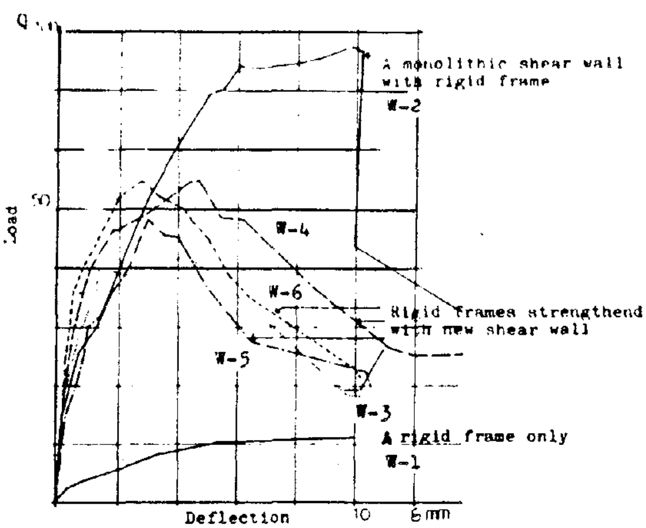


Figure 2.2 Test setup [2.1].



(a)



(b)

Figure 2.3 Hayashi et al's test results [2.1]
a) Hysteresis curves.
b) Envelopes of hysteresis curves.

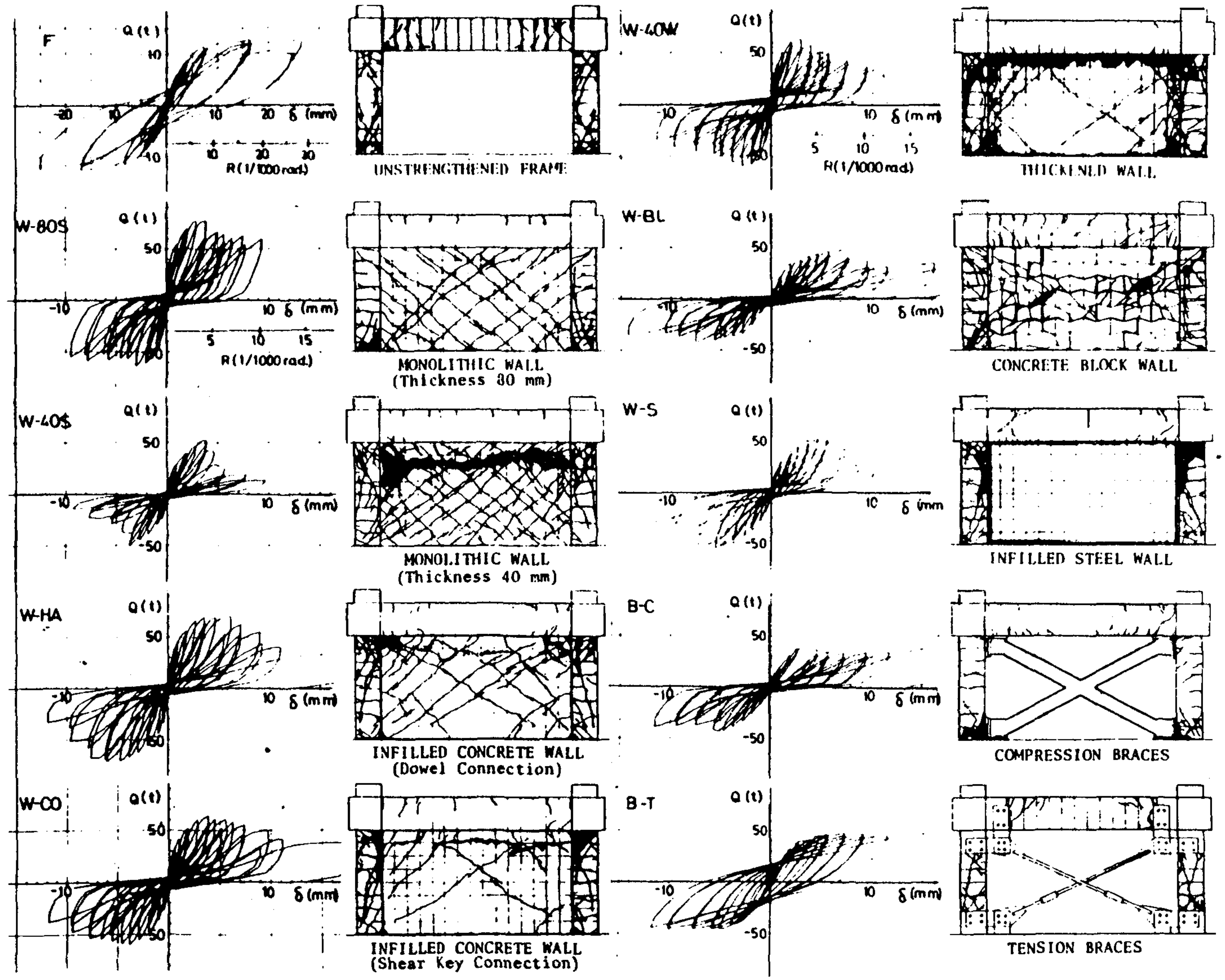
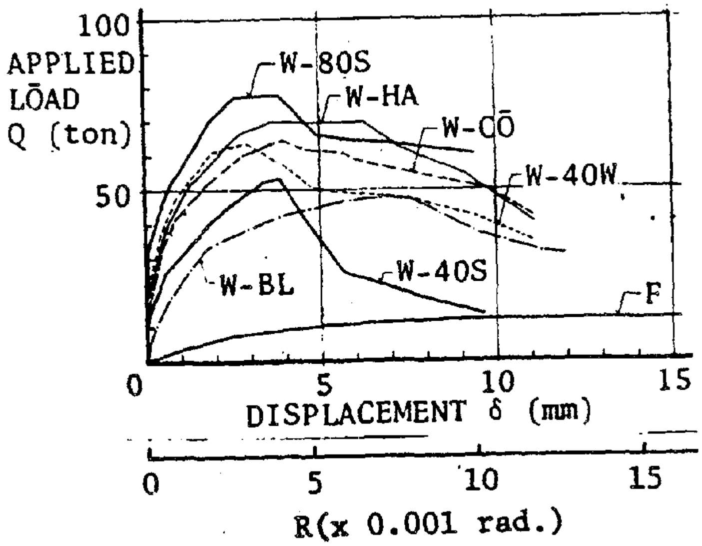
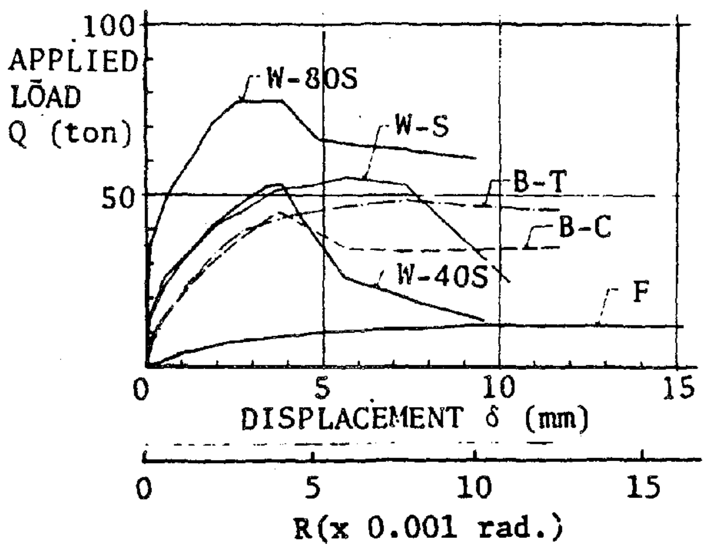


Figure 2.4 Hysteresis curves and crack patterns [2.2].



(a)



(b)

Figure 2.5 Envelopes of hysteresis curves [2.2].
a) Infilled frames.
b) Steel braced frames.

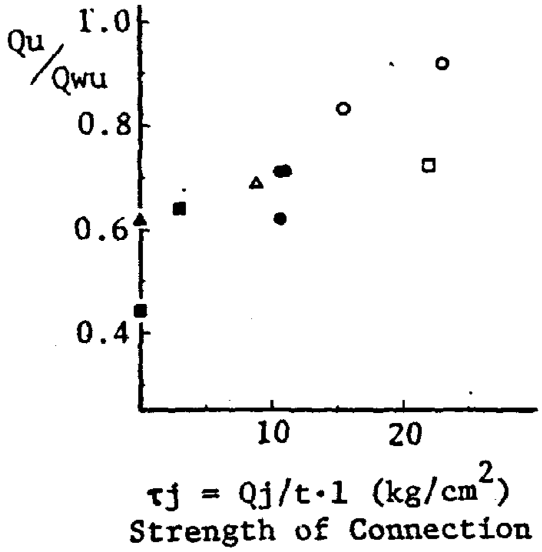
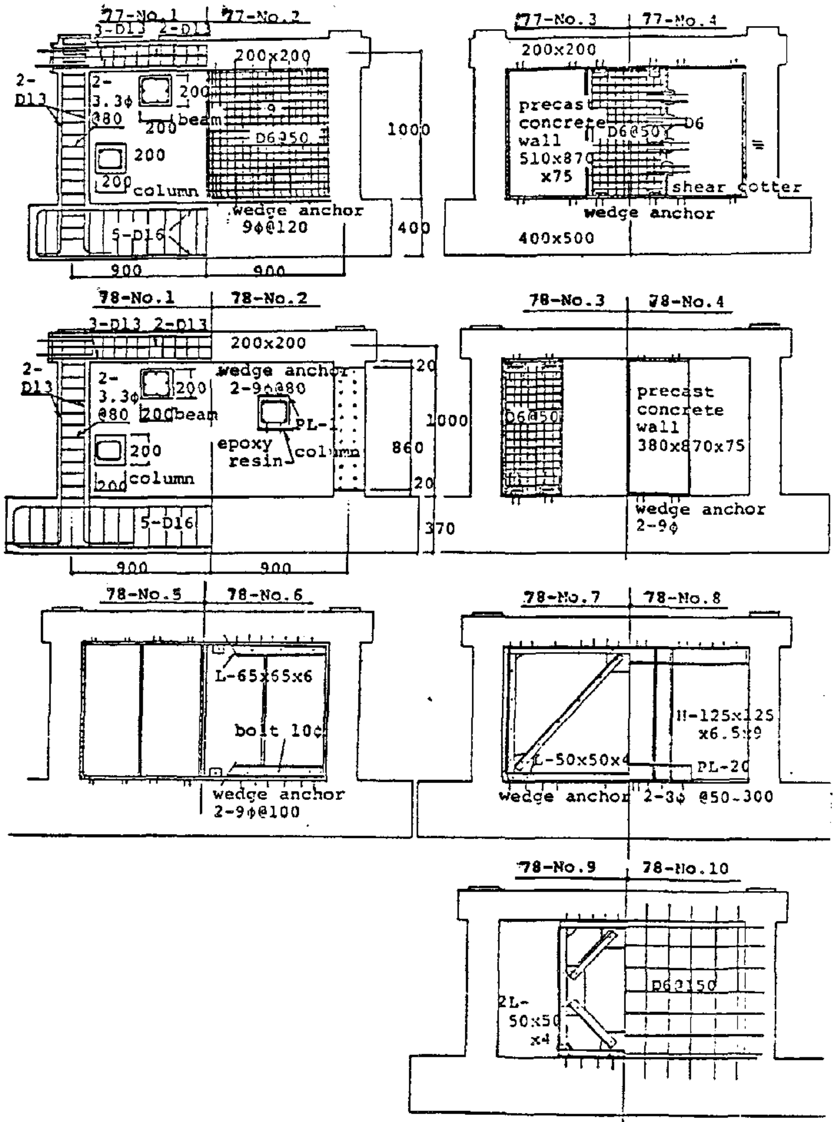


Figure 2.6 Strength of connection vs. frame's lateral strength [2.2].



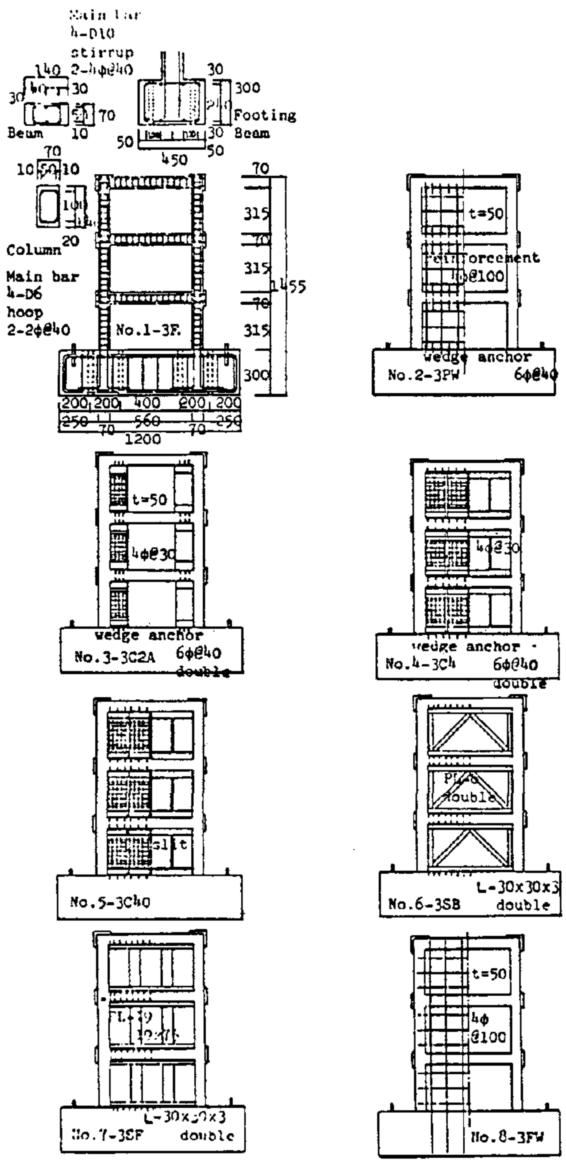
(a)

Figure 2.7 Details of Higashi et al's specimens [2.3,2.7].

a) Specimens in 1977,78 series.

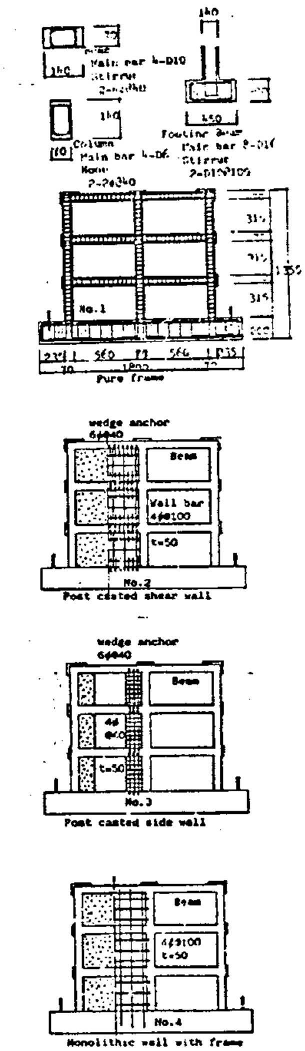
b) Specimens in 1979 series.

c) Specimens in 1981 series.



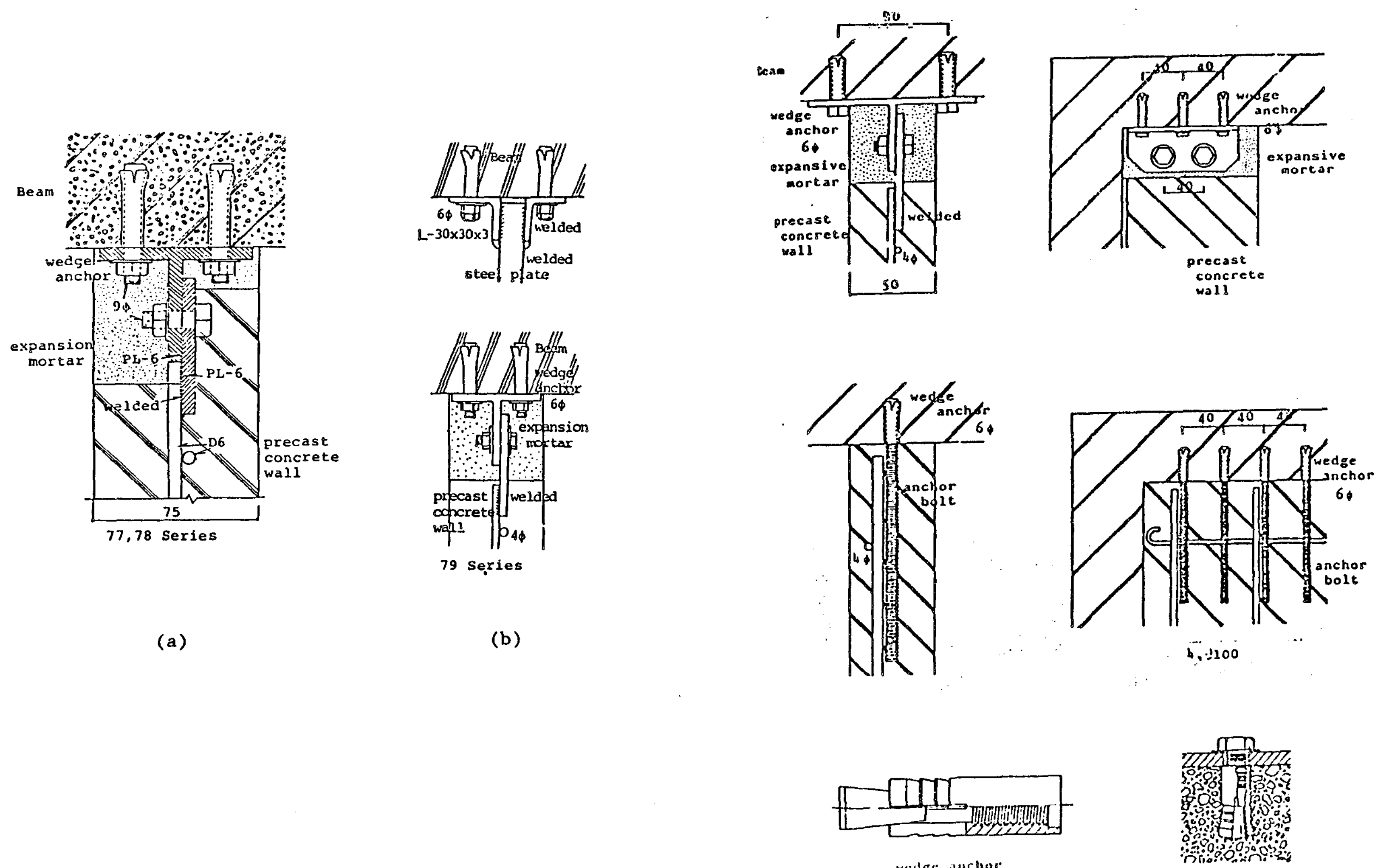
Footing Beam
Beam
Column
Main bar
4-D6
hoop
2-2φ@40
No.1-3F
t=50
wedge anchor
No.2-3PW
6φ@40
t=50
4φ@30
No.3-3C2A
6φ@40
double
No.4-3C4
6φ@40
double
slit
No.5-3C40
No.6-3SB
L-30x30x3
double
t=50
4φ
@100
No.8-3FW
double

(b)



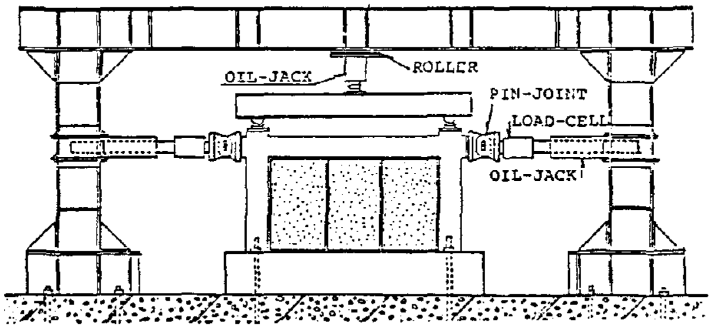
No.1
Pure frame
wedge anchor
6φ@40
Beam
Wall bar
4φ@100
t=50
No.2
Post casted shear wall
wedge anchor
6φ@40
Beam
t=50
No.3
Post casted side wall
Beam
4φ@100
t=50
No.4
Monolithic wall with frame

(c)

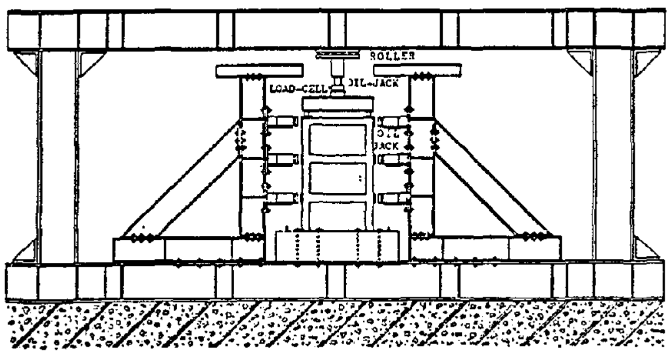


(c)

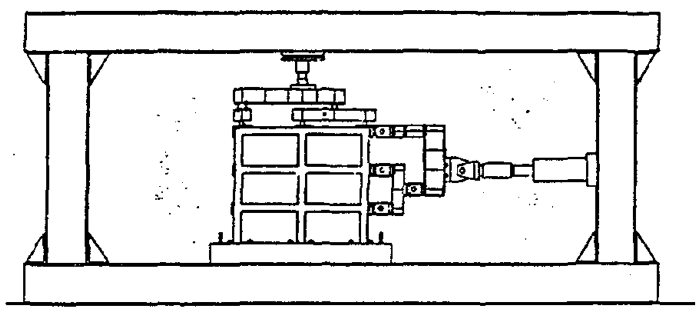
Figure 2.8 Details of the connection between infill wall and frame [2.3,2.7].
a) 1977,78 series.
b) 1979 series.
c) 1981 series.



(a)



(b)



(c)

Figure 2.9 Higashi et al's test setups [2.3,2.7].
a) One-bay, one-story frame.
b) One-bay, three-story frame.
c) Two-bay, three-story frame.

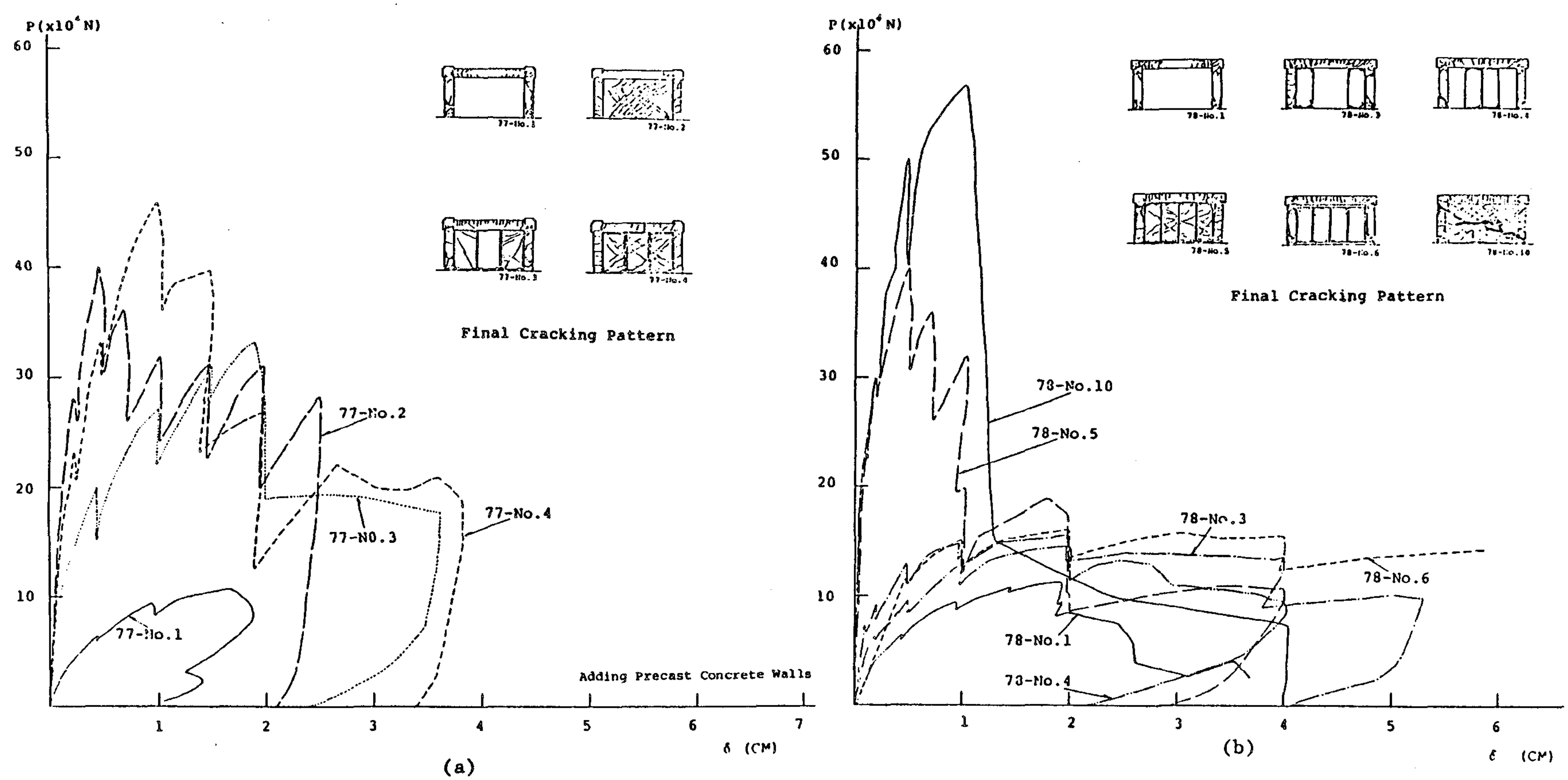
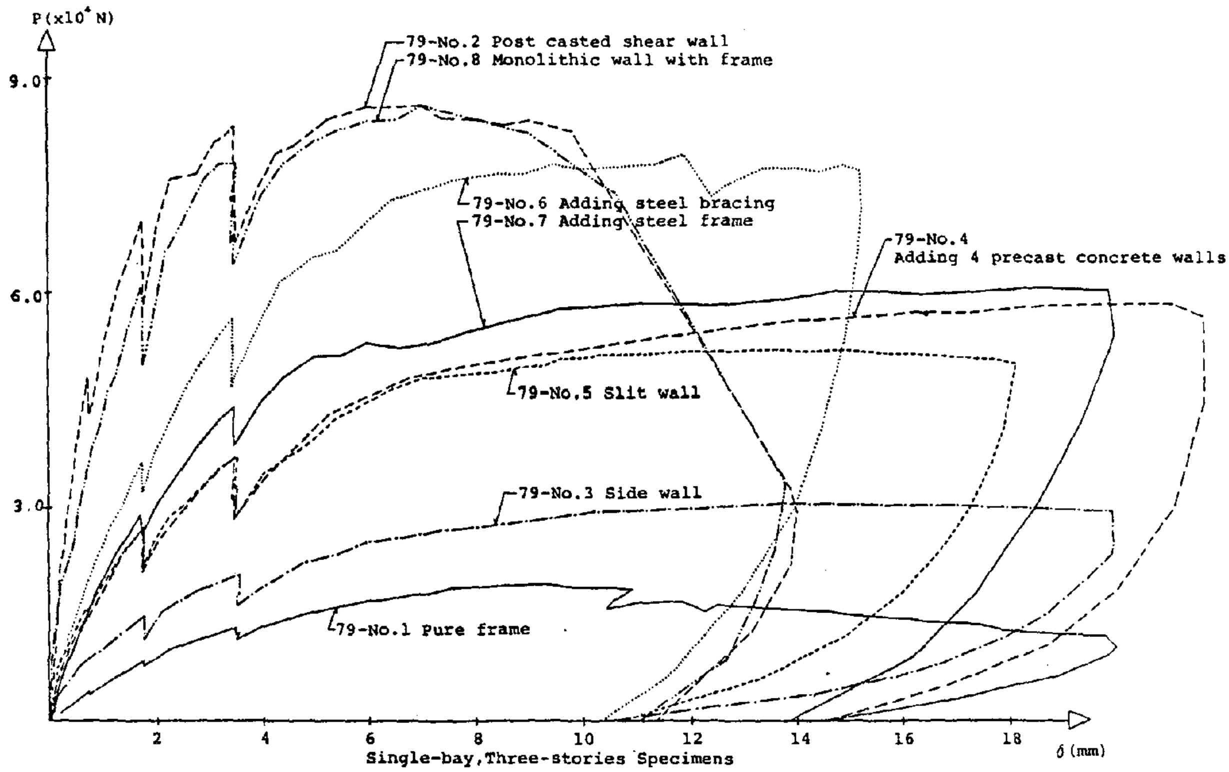
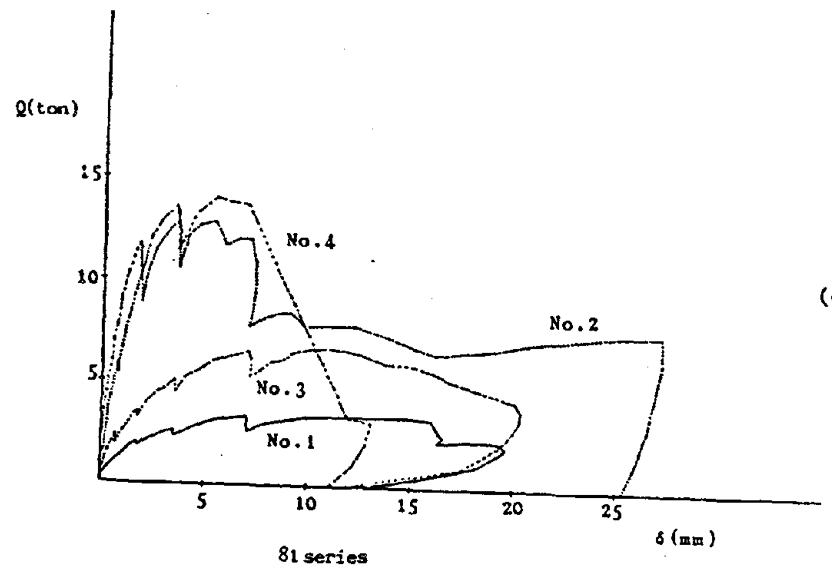


Figure 2.10 Envelopes of hysteresis curves [2.3,2.7]
a) 1977 series.
b) 1978 series.
c) 1979 series.
d) 1981 series.

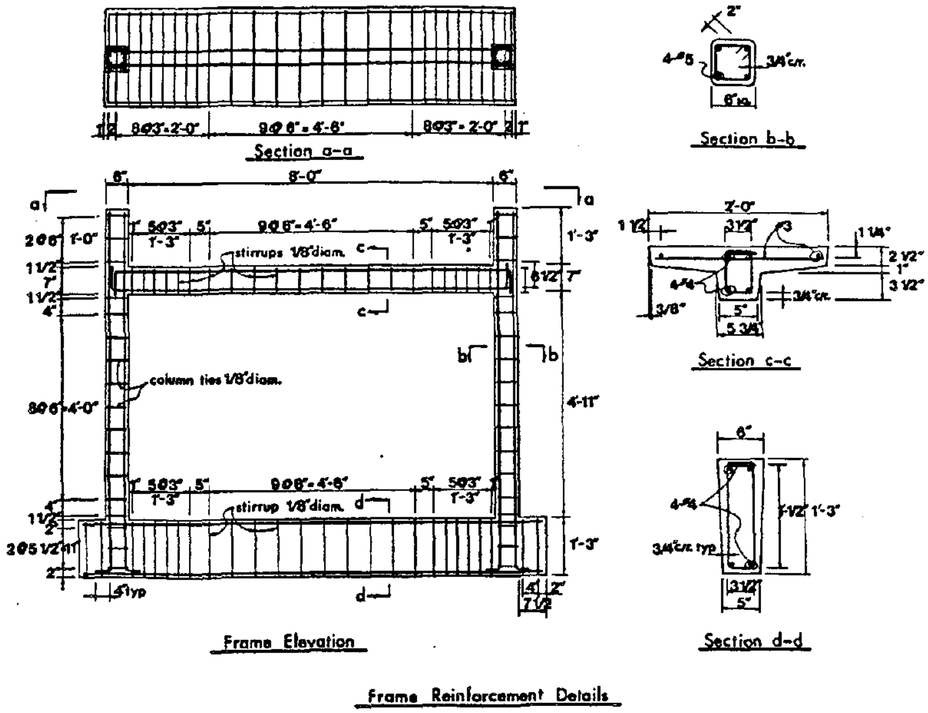


(c)

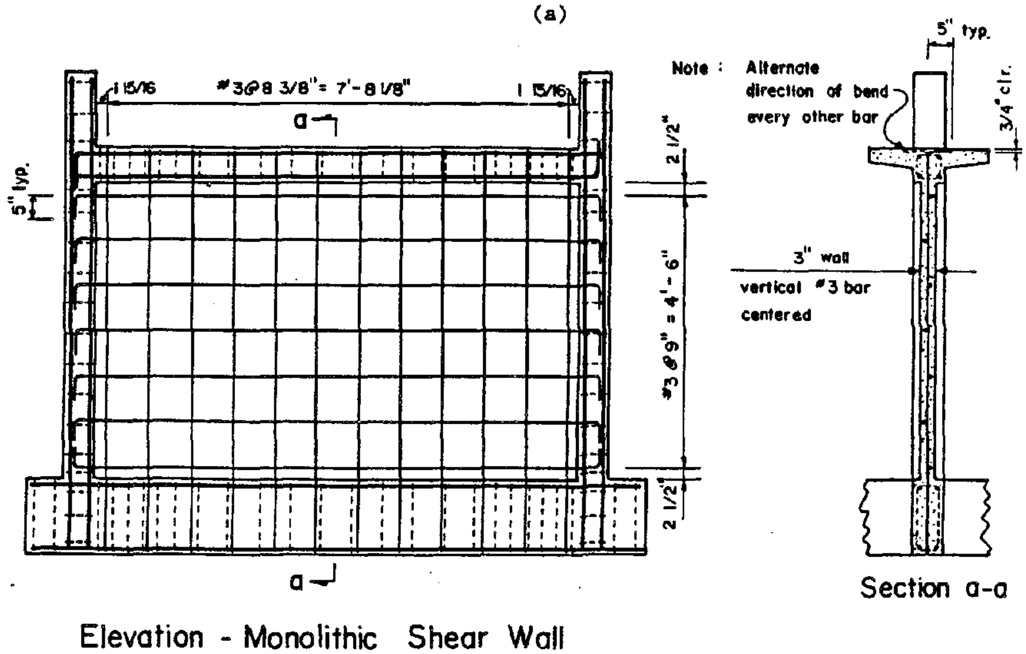


(d)

Figure 2.10 (Continued)

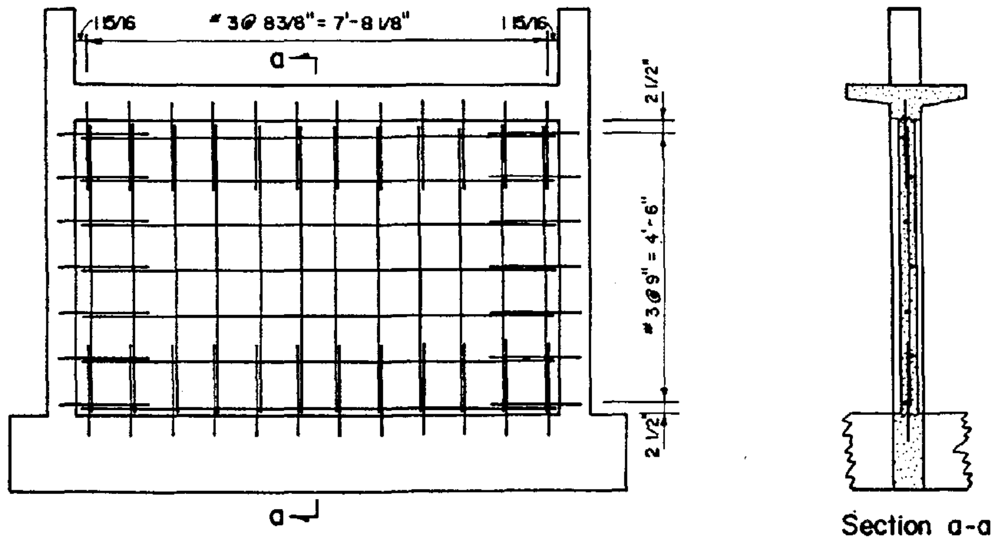


(a)



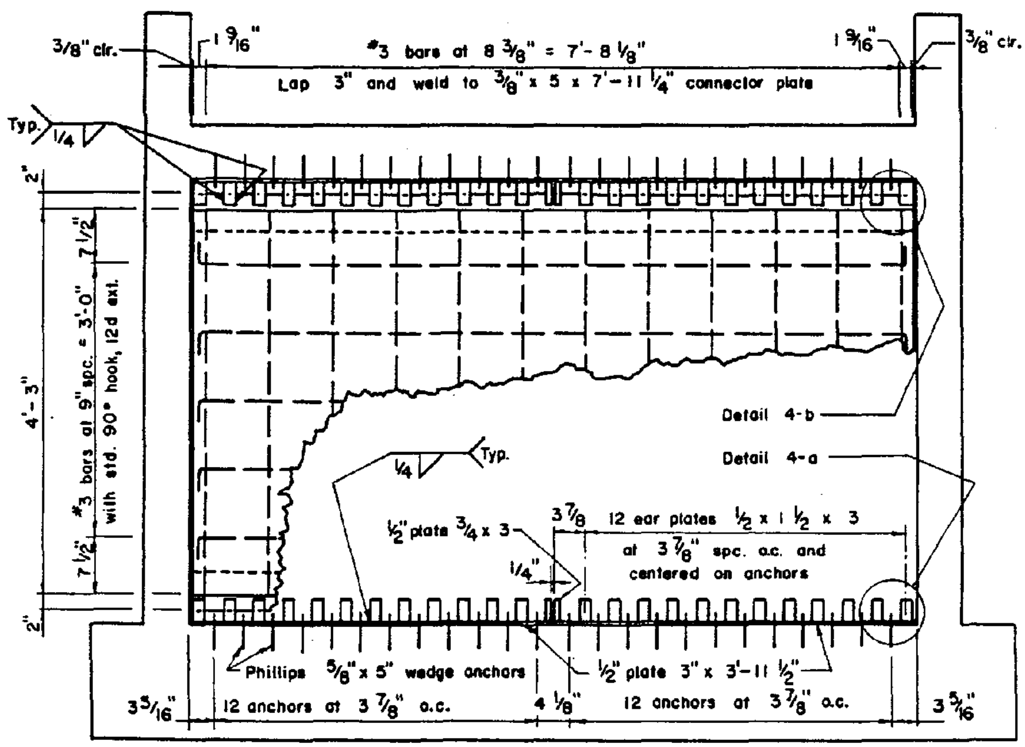
(b)

Figure 2.11 Reinforcement arrangement in Kahn's specimens [2.4]
a) Bare frame reinforcement details (specimen 2).
b) Monolithic shear wall reinforcing detail (specimen 1).
c) Cast-in-place infilled frame (specimen 3).
d) Single precast infill panel (specimen 4).
e) Multiple precast infill panels (specimen 5).

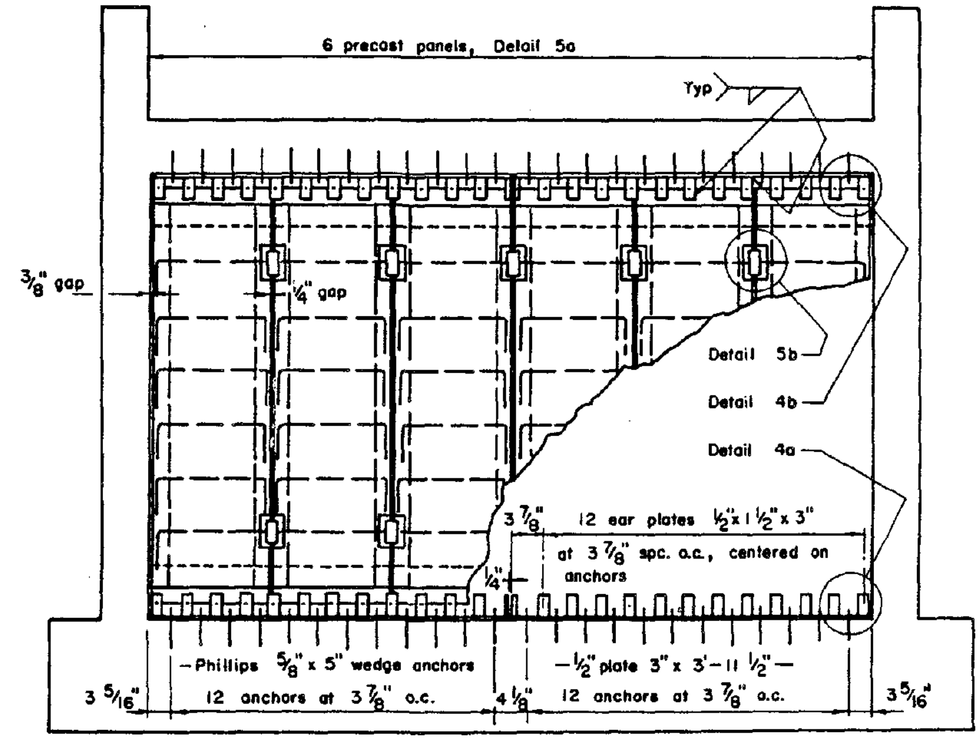


Elevation Cast-in-Place Shear Wall

(c)



(d)


6 precast panels, Detail 5a
Typ
3/8" gap
1/4" gap
Detail 5b
Detail 4b
Detail 4a
3 7/8"
12 ear plates 1/2" x 1 1/2" x 3"
at 3 7/8" spc. o.c., centered on anchors
1/4"
Phillips 5/8" x 5" wedge anchors
1/2" plate 3" x 3'-11 1/2"
3 5/16"
12 anchors at 3 7/8" o.c.
4 1/8"
12 anchors at 3 7/8" o.c.
3 5/16"

(e)

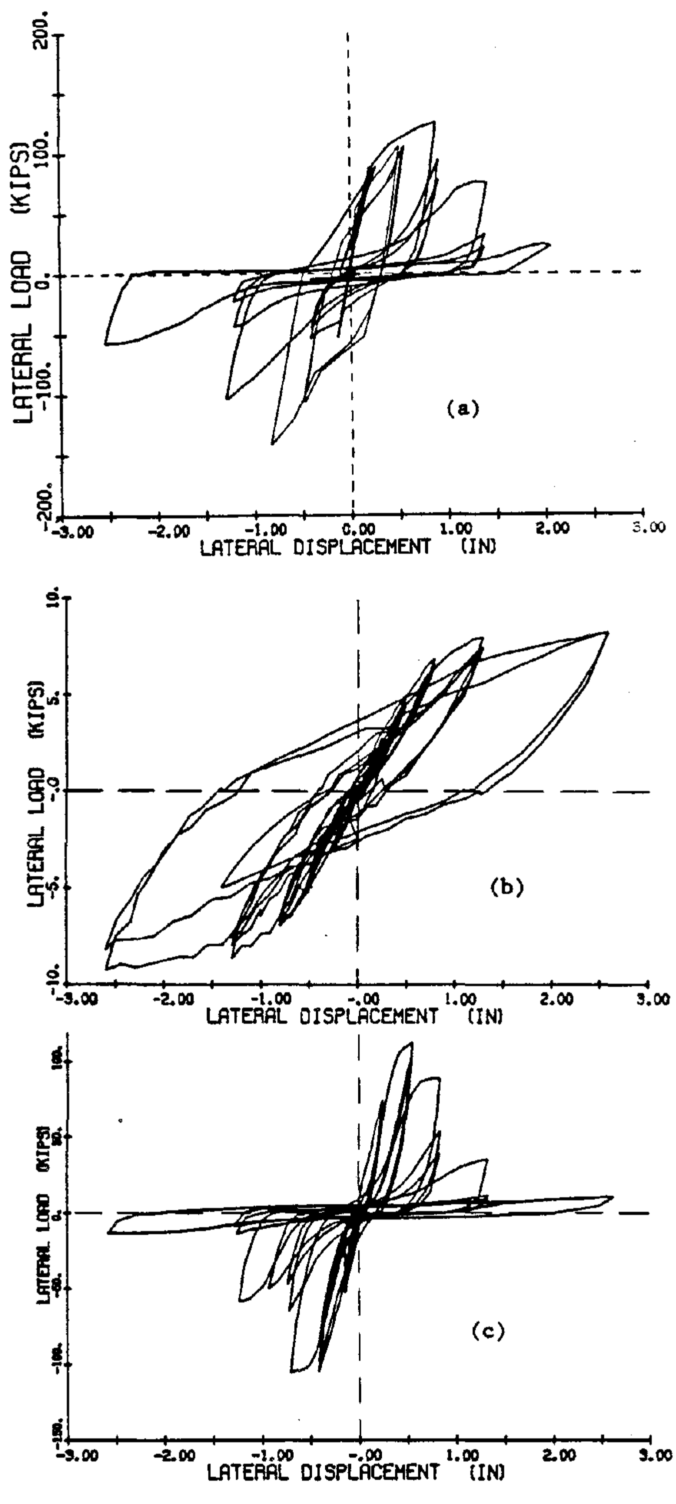
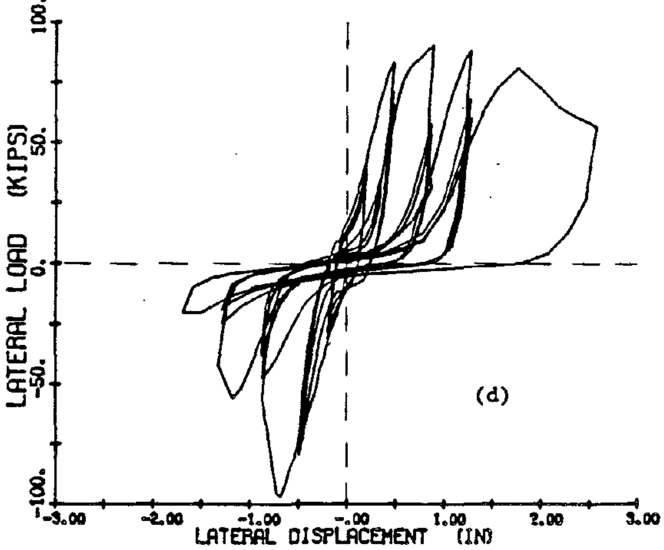
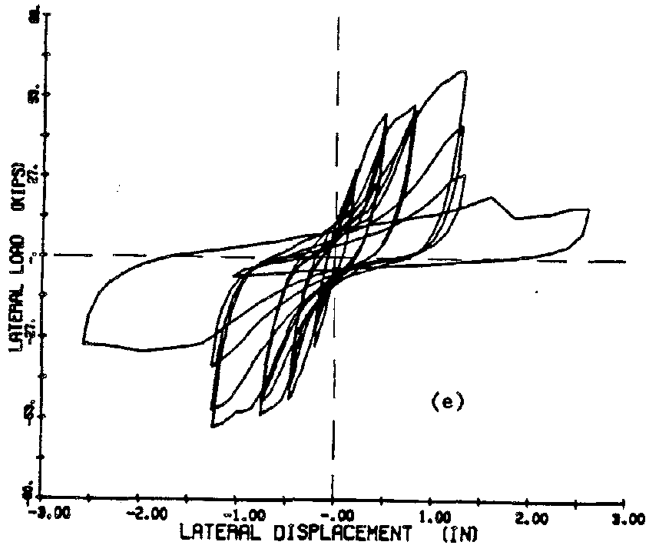


Figure 2.12 Hysteresis behavior of Kahn's specimens [2.4]
a) Specimen 1.
b) Specimen 2.
c) Specimen 3.
d) Specimen 4.
e) Specimen 5.


LATERAL LOAD (KIPS)
100.
50.
0.
-50.
-100.
-3.00
-2.00
-1.00
-.00
1.00
2.00
3.00
LATERAL DISPLACEMENT (IN)
(d)


LATERAL LOAD (KIPS)
LATERAL DISPLACEMENT (IN)
-3.00
-2.00
-1.00
-.00
1.00
2.00
3.00
(e)

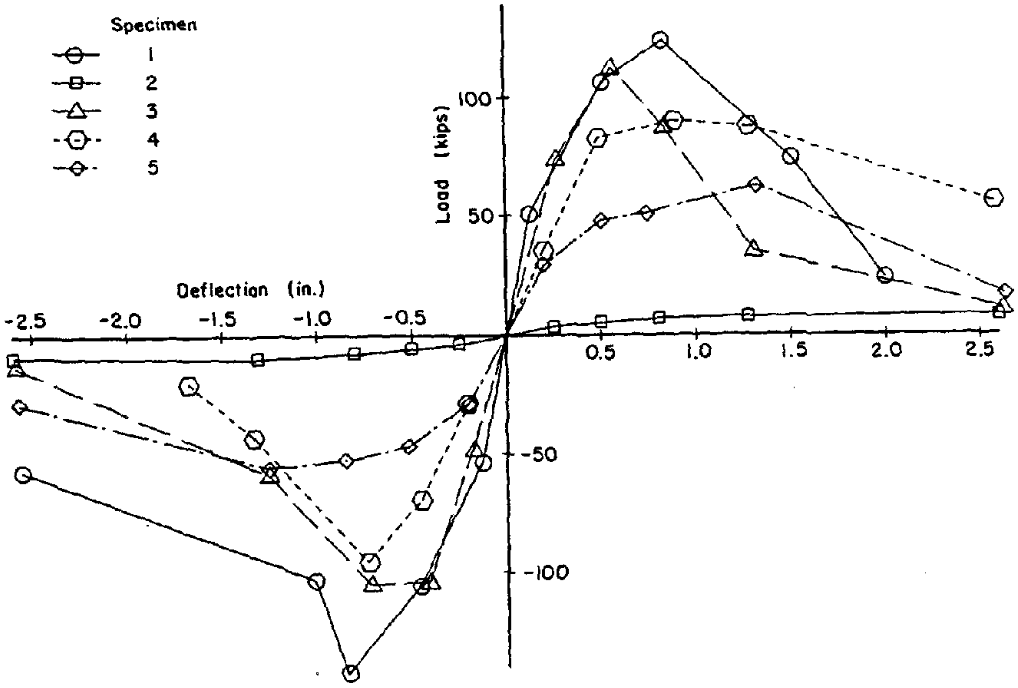


Figure 2.13 Envelopes of hysteresis curves of Kahn's specimen [2.4]

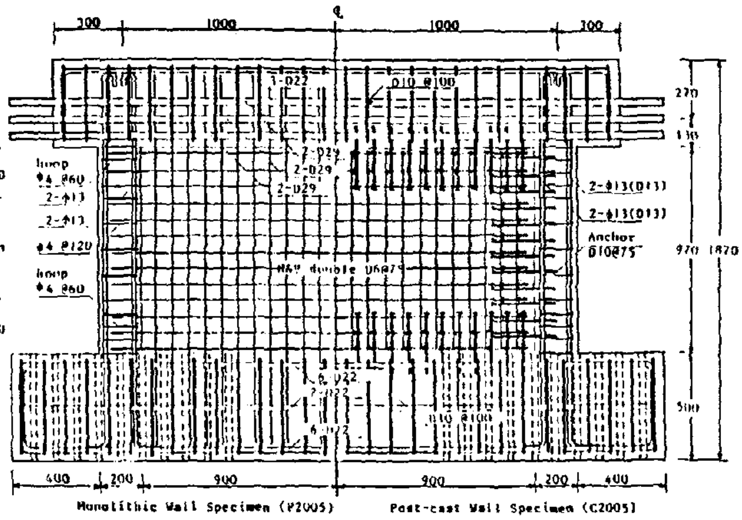


Figure 2.14 Dimensions and reinforcement arrangement of Shiohara et al.'s specimens [2.22]

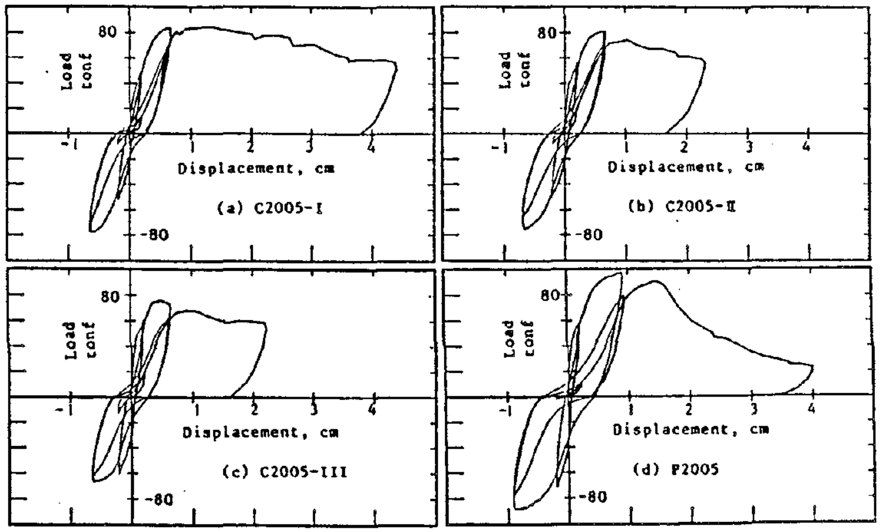


Figure 2.15 Hysteresis curves of Shiohara et al.'s specimens [2.22]

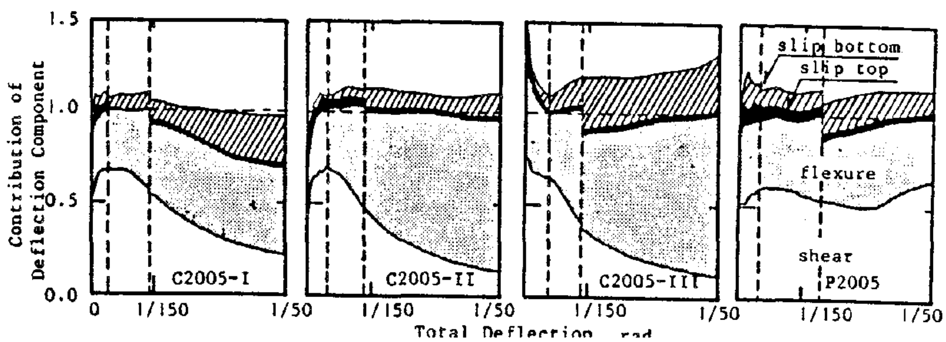
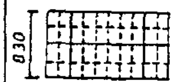
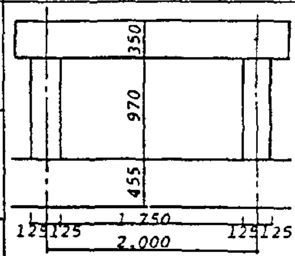
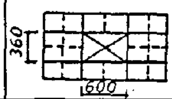
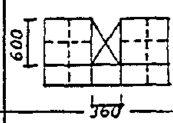
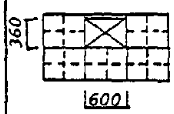
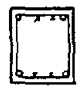
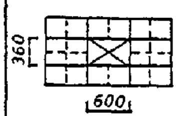
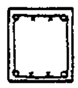


Figure 2.16 Measured displacement components of Shiohara et al.'s specimens [2.22]

Specimens	Steel Panel		Stud n	Headed Anchor n	RC Frame
P-1-0		FLANGE: PL-6x80 WEB: PL-4.5	UG·LG 14 RC LC 7	UG LG 13 RC LC 8	
P-1-S		FLANGE: PL-6x80 WEB: PL-4.5	UG LG 20 RC LC 21	UG LG 21 RC LC 20	
P-2-C		FLANGE: PL-6x80 FLANGE: PL-9x80 WEB: PL-4.5	UG 20 LG 26 RC·LC 13	UG 22 LG 27 RC LC 14	 bXD=200x250 A_g: 6-D16 Hoop 2-4Φ @120
P-2-G		FLANGE: PL-12x80 WEB: PL-9 FLANGE: PL-6x80 WEB: PL-4.5	UG LG 26 RC LC 13	UG LG 27 RC LC 14	 bXD=300x350 A_t: 2-D16,2-D13 St: 2-6Φ @120
P-1-N		FLANGE: PL-6x80 WEB: PL-4.5	UG LG 26 RC LC 13	UG LG 27 RC LG 14	 bXD=200x200 A_g: 6-D19 Hoop 2-4Φ @120  bXD=300x350 A_t: 2-D19,2-D13 St: 2-6Φ @120

notes: PL-6x80 was used as steel rim, and PL-4.5x35 as stiffners. Signatures UG and LG mean the upper and lower girders, and RC and LC mean the right and left columns.

Figure 2.17 Description of Aoyama et al.'s test specimens [2.23]

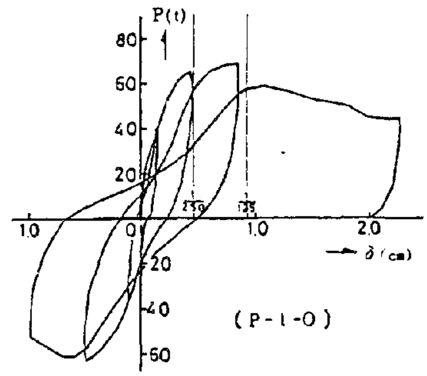


Figure 2.18 Hysteresis behavior of
Aoyama et al.'s specimen P-1-0 [2.23].

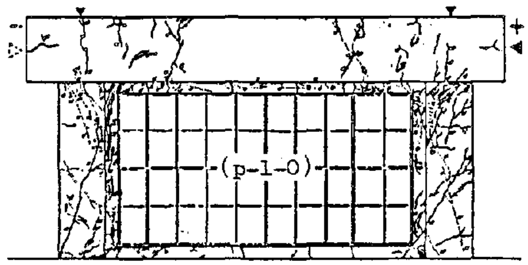


Figure 2.19 Crack patterns in Aoyama et al.' test
specimen P-1-0 [2.23].

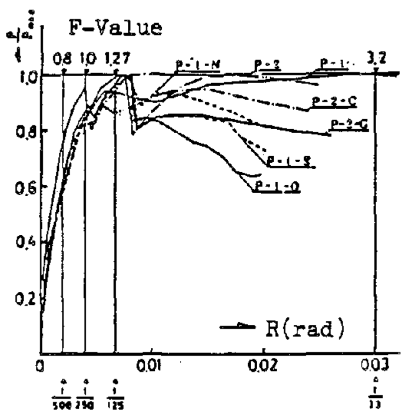


Figure 2.20 Deflection envelopes of
Aoyama et al.'s test specimens [2.23].

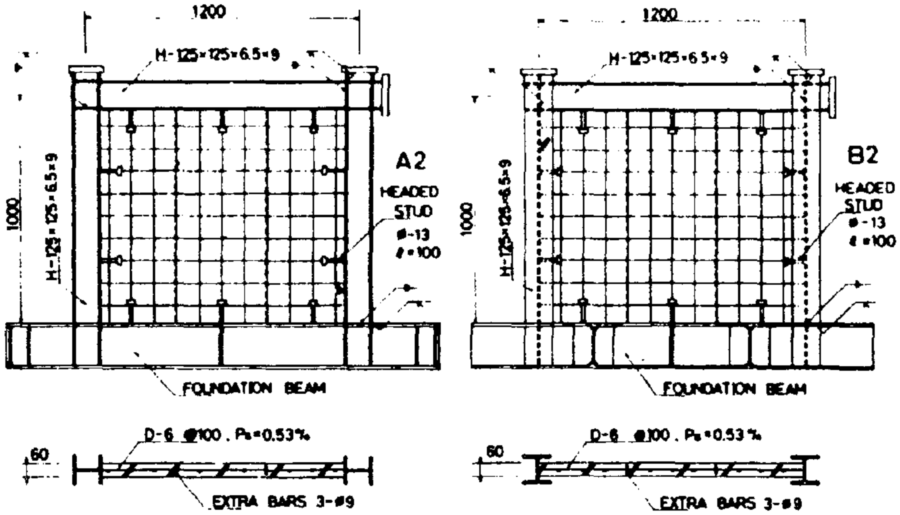


Figure 2.21 Makino et al.'s steel frame specimens A2 and B2 [2.5].

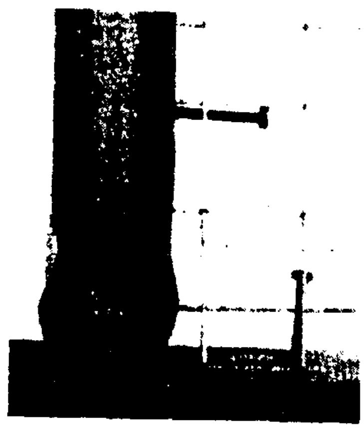


Figure 2.22 Local buckling in steel column [2.5].

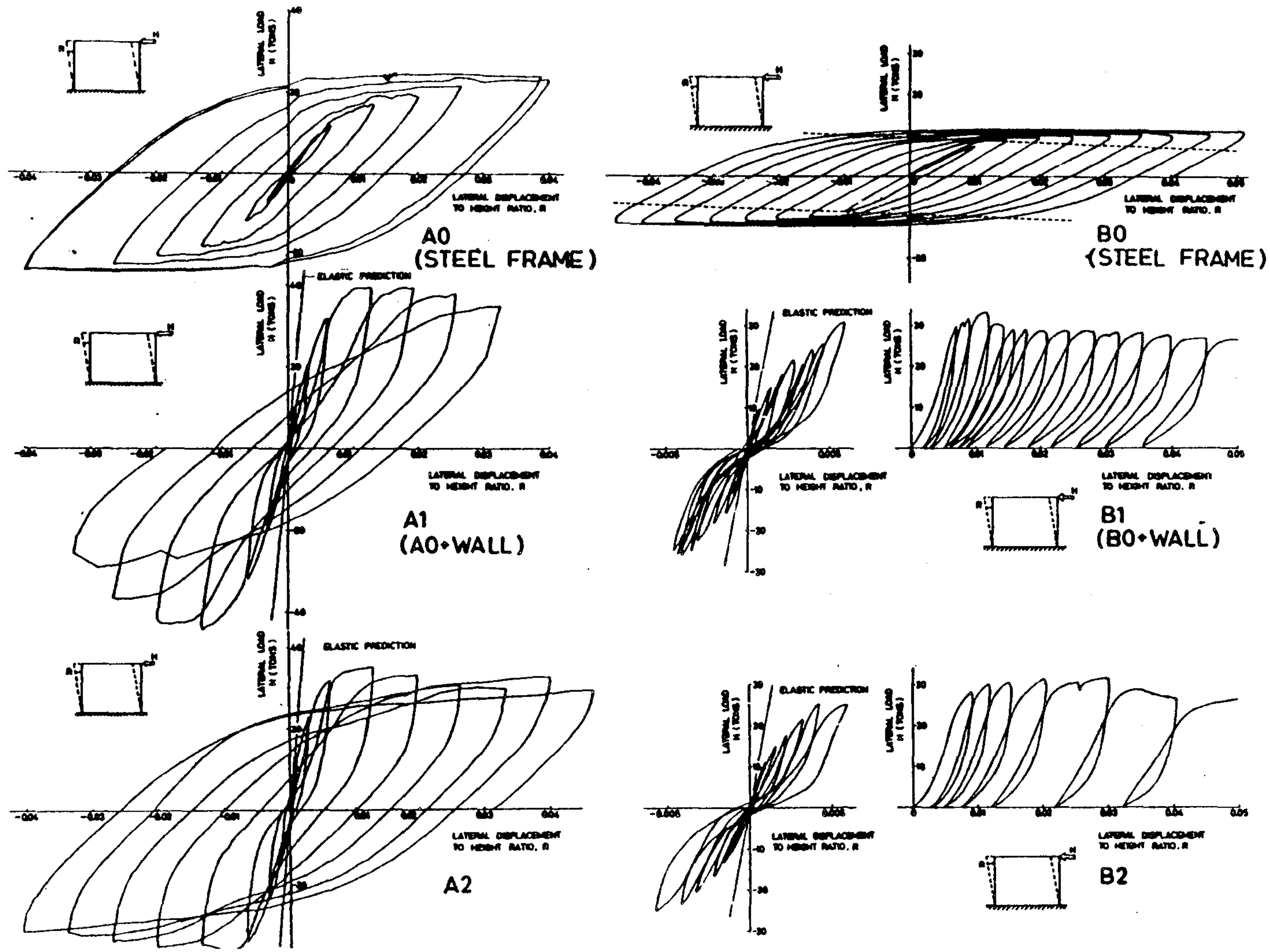


Figure 2.23 Hysteresis curves of original and strengthened frames [2.5].

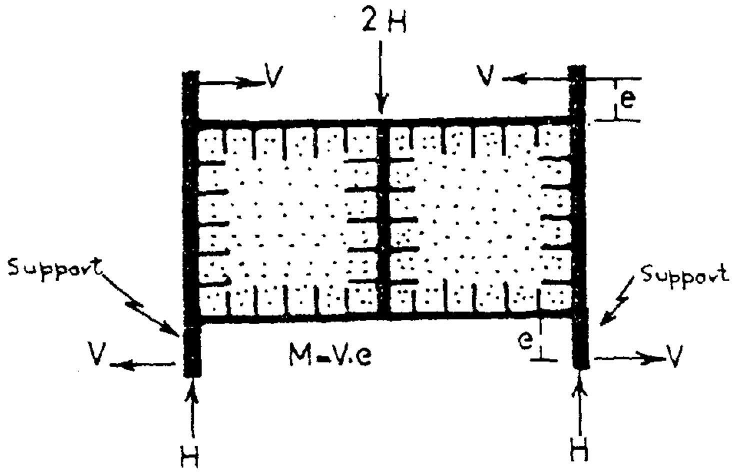


Figure 2.24 Mallick's test frames [2.6].

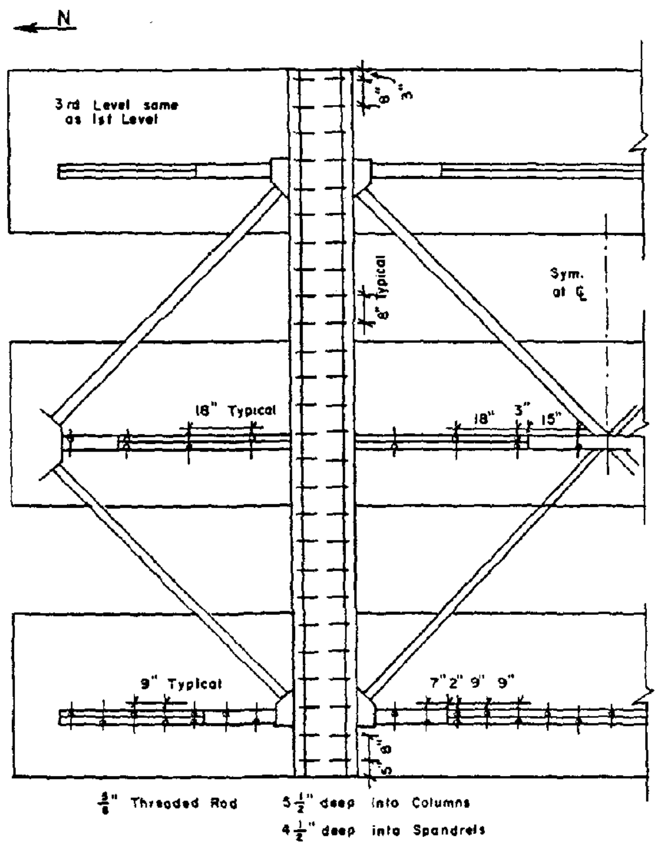
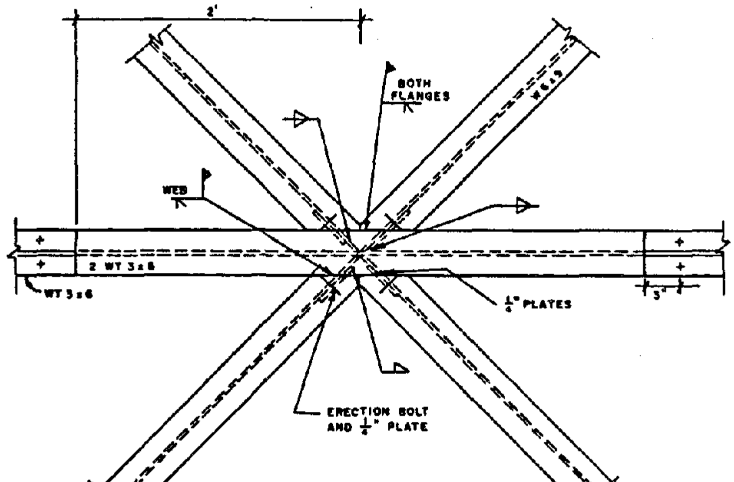
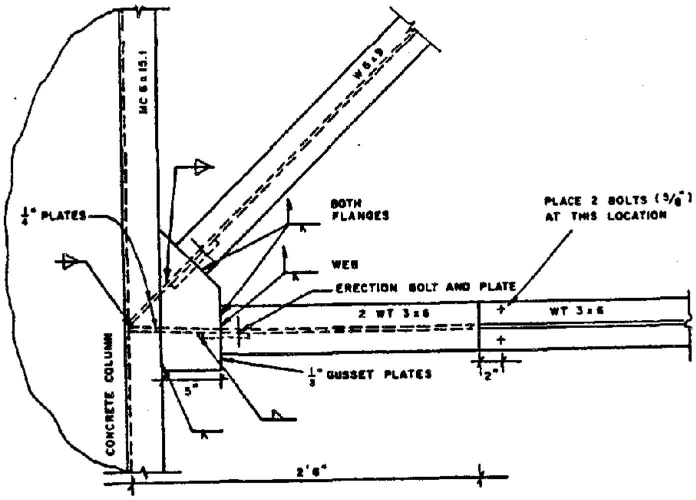


Figure 2.25 Dowel layout for Jones and Jirsa's steel bracing scheme [2.8,2.9].



(a)



(b)

Figure 2.26 Connection details [2.8,2.9].
a) Brace-to-collector connection.
b) Brace-to-channel connection.

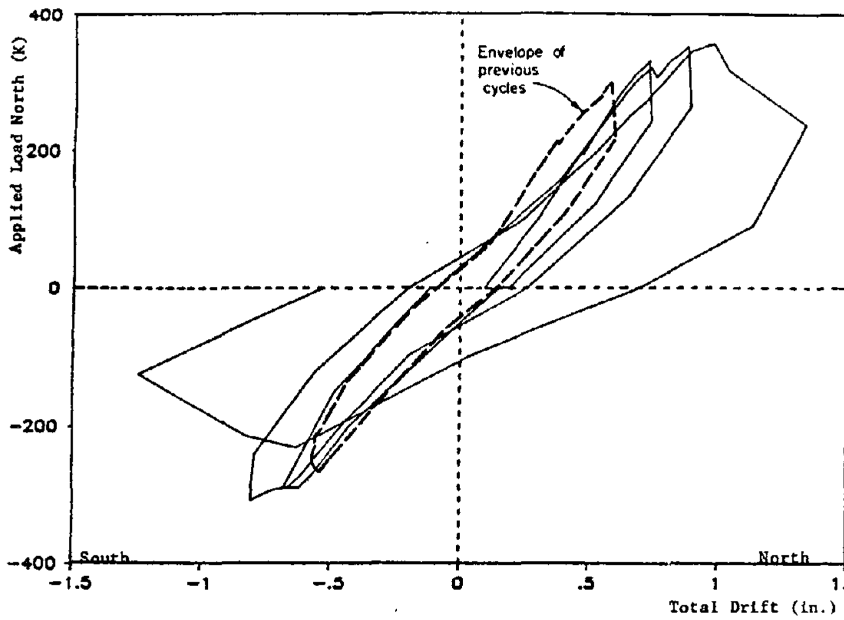


Figure 2.27 Load-drift relationship of the steel braced frame [2.8,2.9].

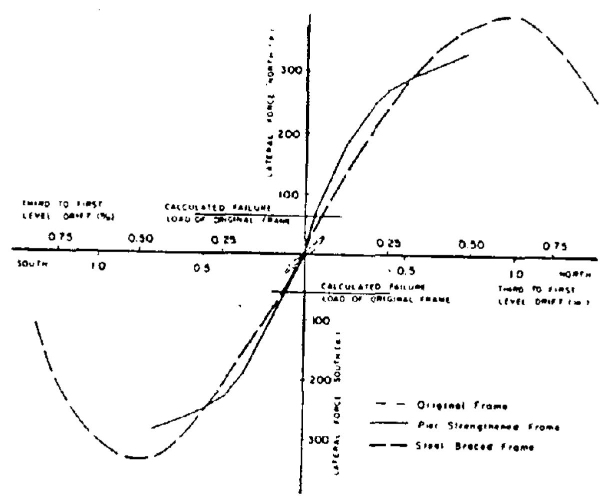


Figure 2.28 Deflection envelopes [2.8,2.9].

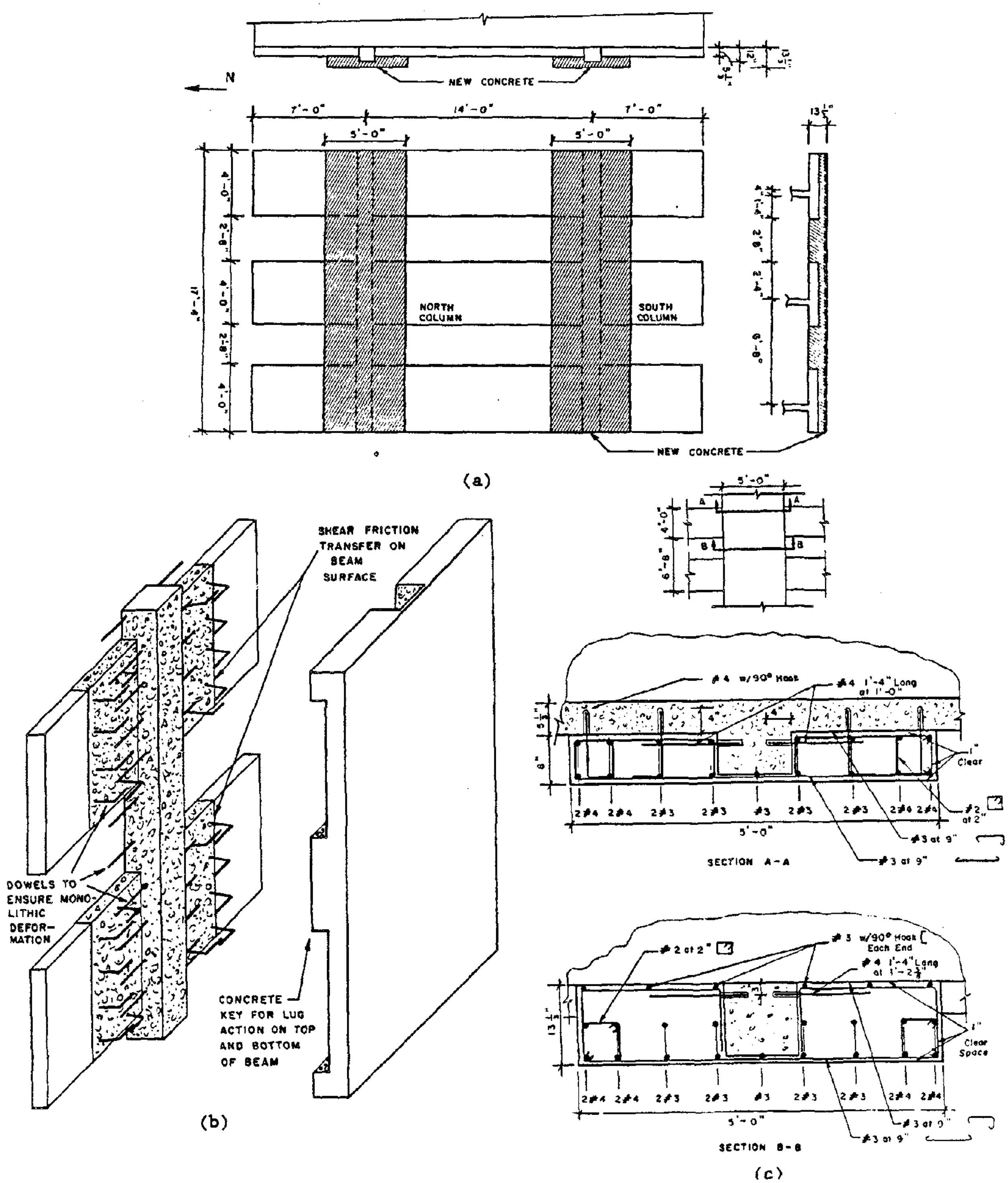


Figure 2.29 Roach and Jirsa's strengthening scheme [2.9,2.10].
a) Strengthening concept.
b) Connection mechanism between old and new concrete.
c) Reinforcing details.

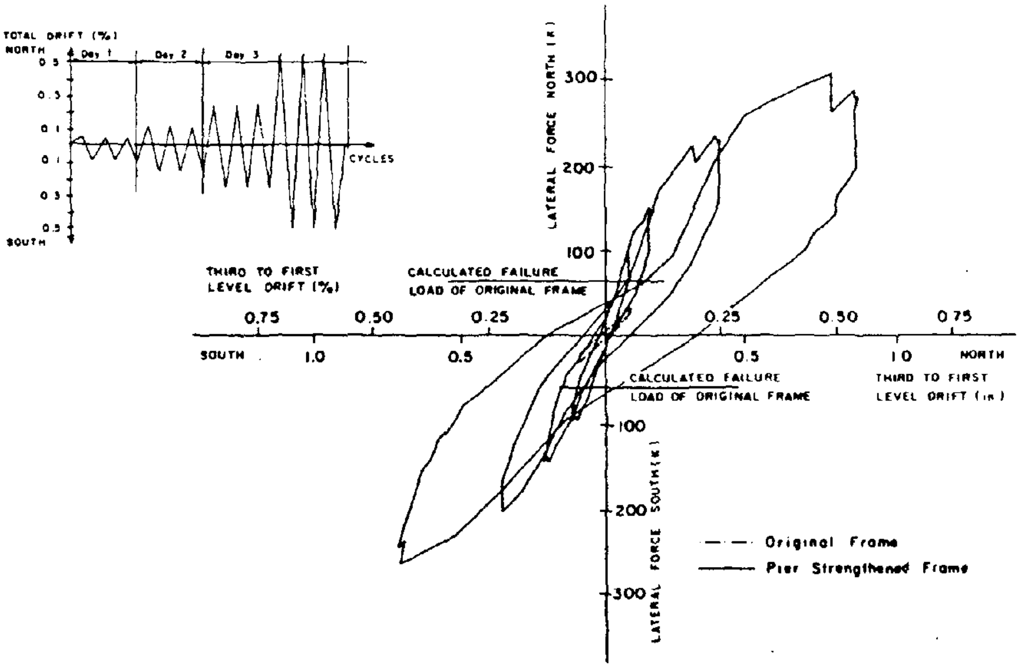


Figure 2.30 Hysteresis curves [2.9,2.10].

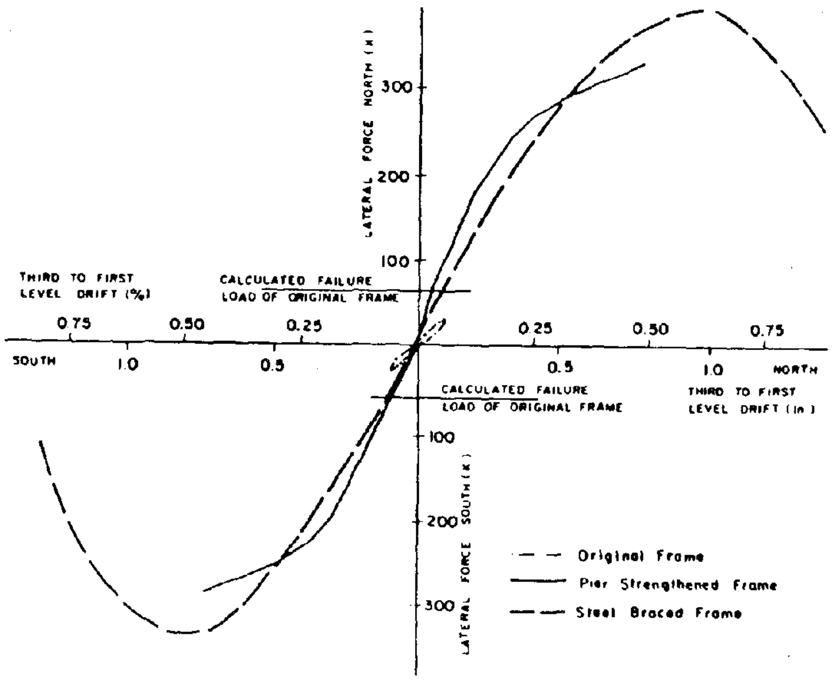


Figure 2.31 Envelopes of hysteresis curves of frames strengthened by adding wingwall and steel bracing [2.8,2.9,2.10].

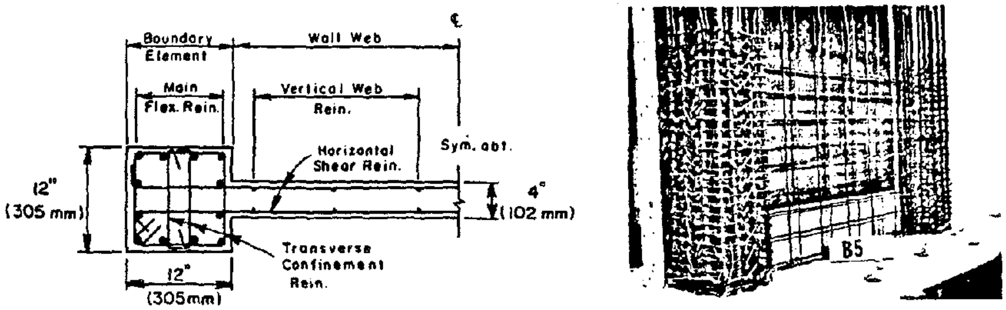


Figure 2.32 Dimensions and reinforcement arrangement of Corley, Fiorato, Oesterle and Scanlon's walls [2.12].

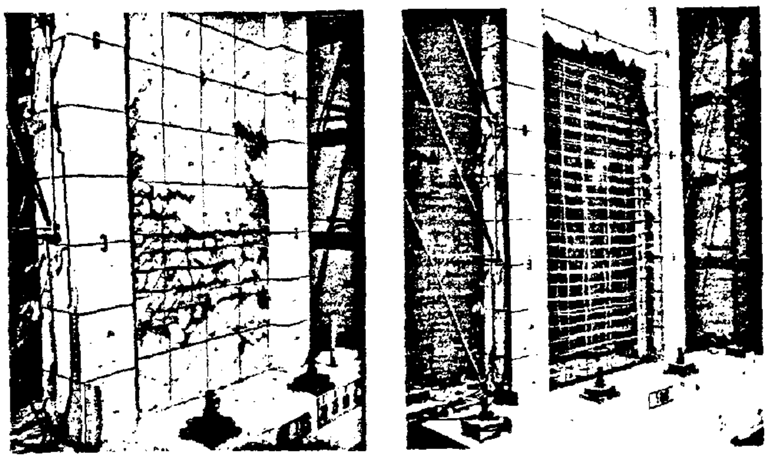


Figure 2.33 Specimen B5 with damaged web and with web concrete removed.

Figure 2.34 Specimen B5R after repair.

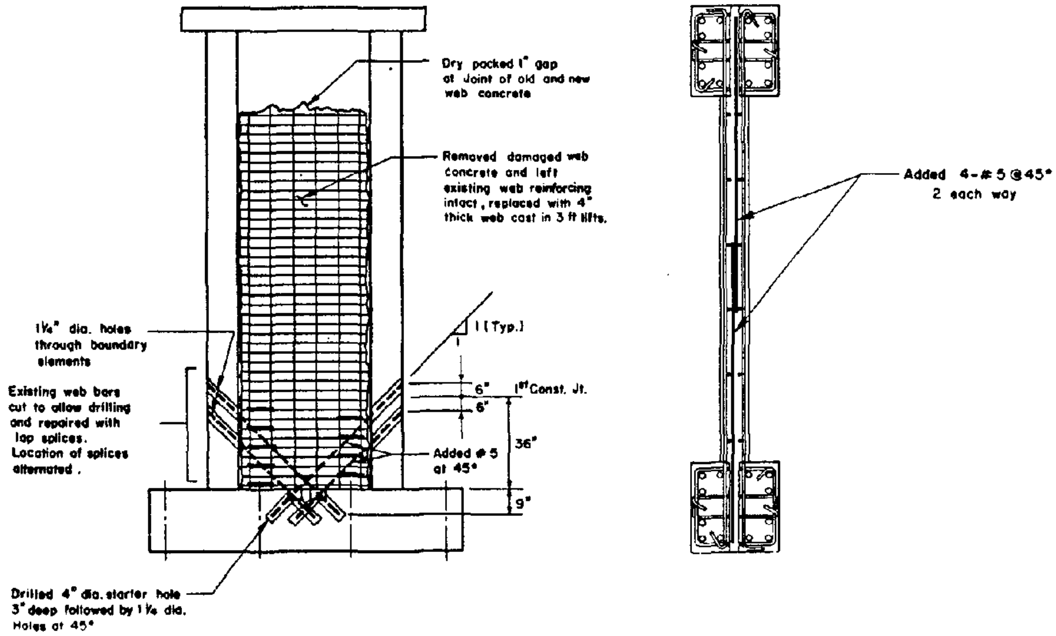
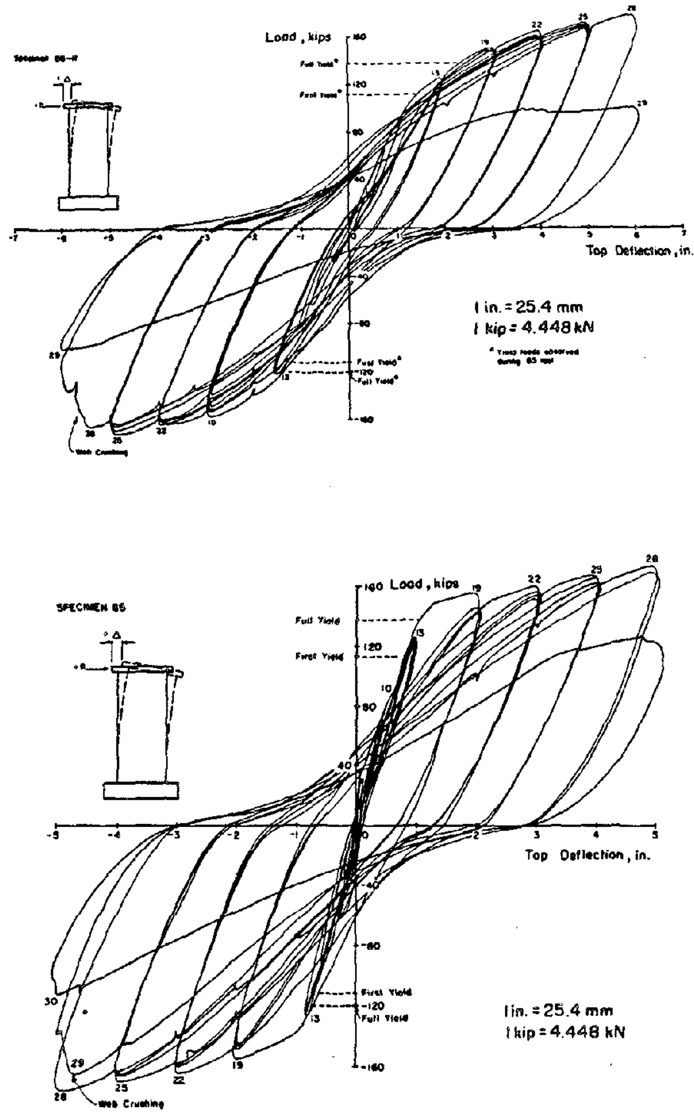


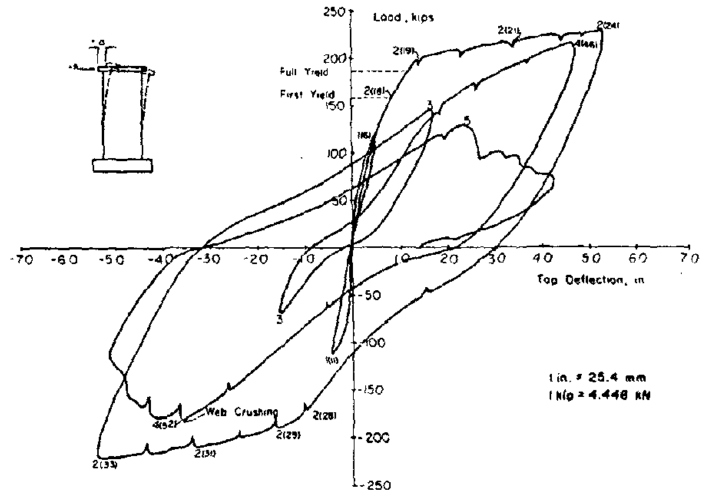
Figure 2.35 Reinforcement arrangement in specimen B11R.

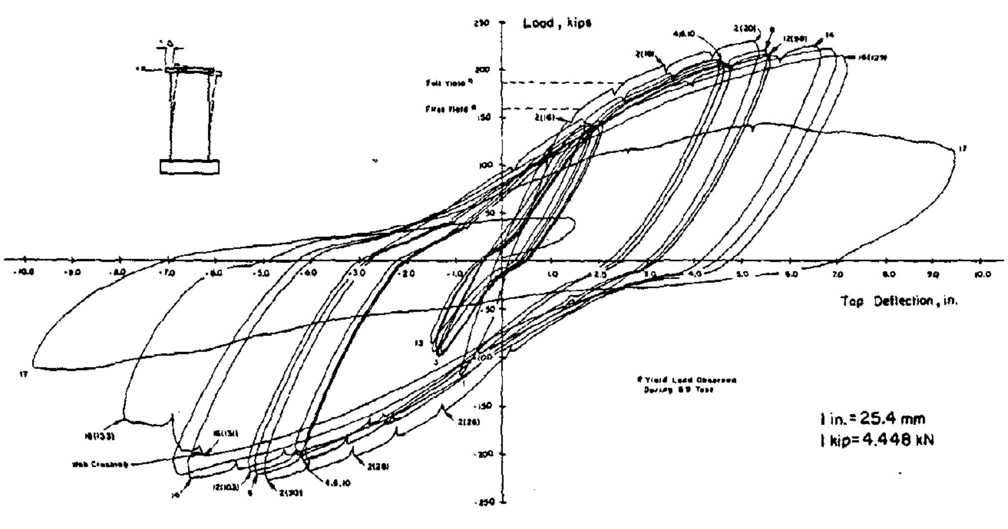
Figure 2.36 Drilling holes for diagonal reinforcement in B11R.



(a)

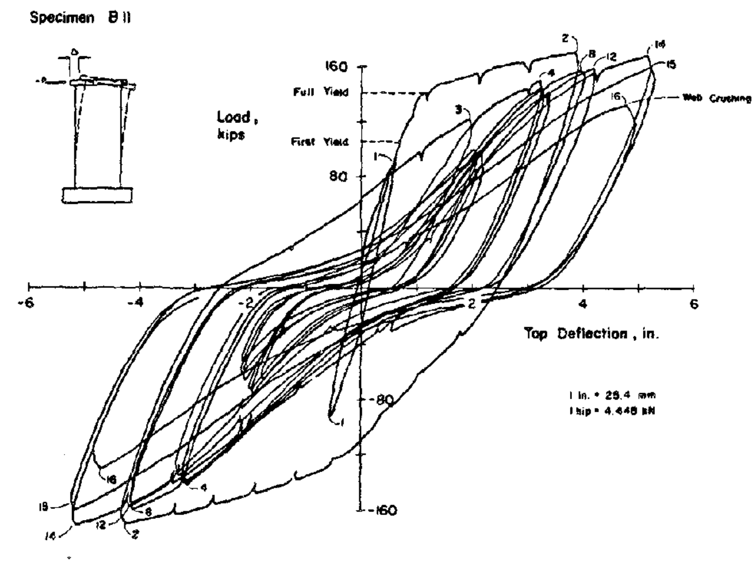
Figure 2.37 Hysteresis curves of Corley et al's original and repaired specimens [2.12].
a) Specimens B5 and B5R.
b) Specimens B9 and B9R.
c) Specimens B11 and B11R.

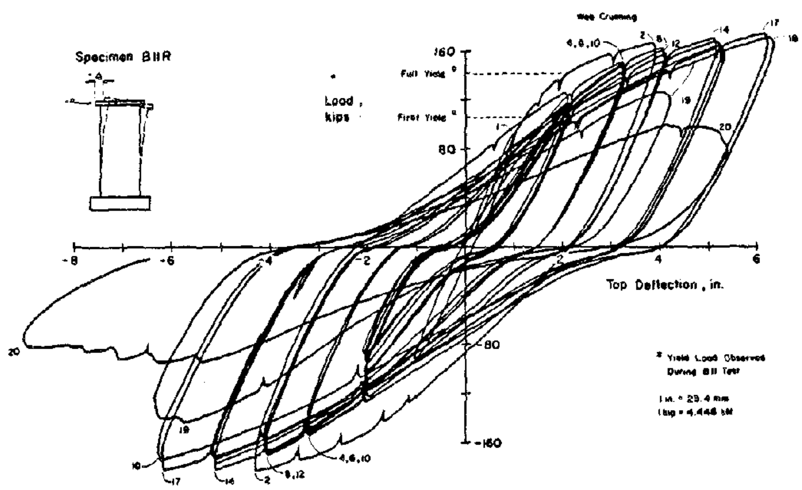




(b)

Figure 2.37 (continued).





(c)

Figure 2.37 (continued)

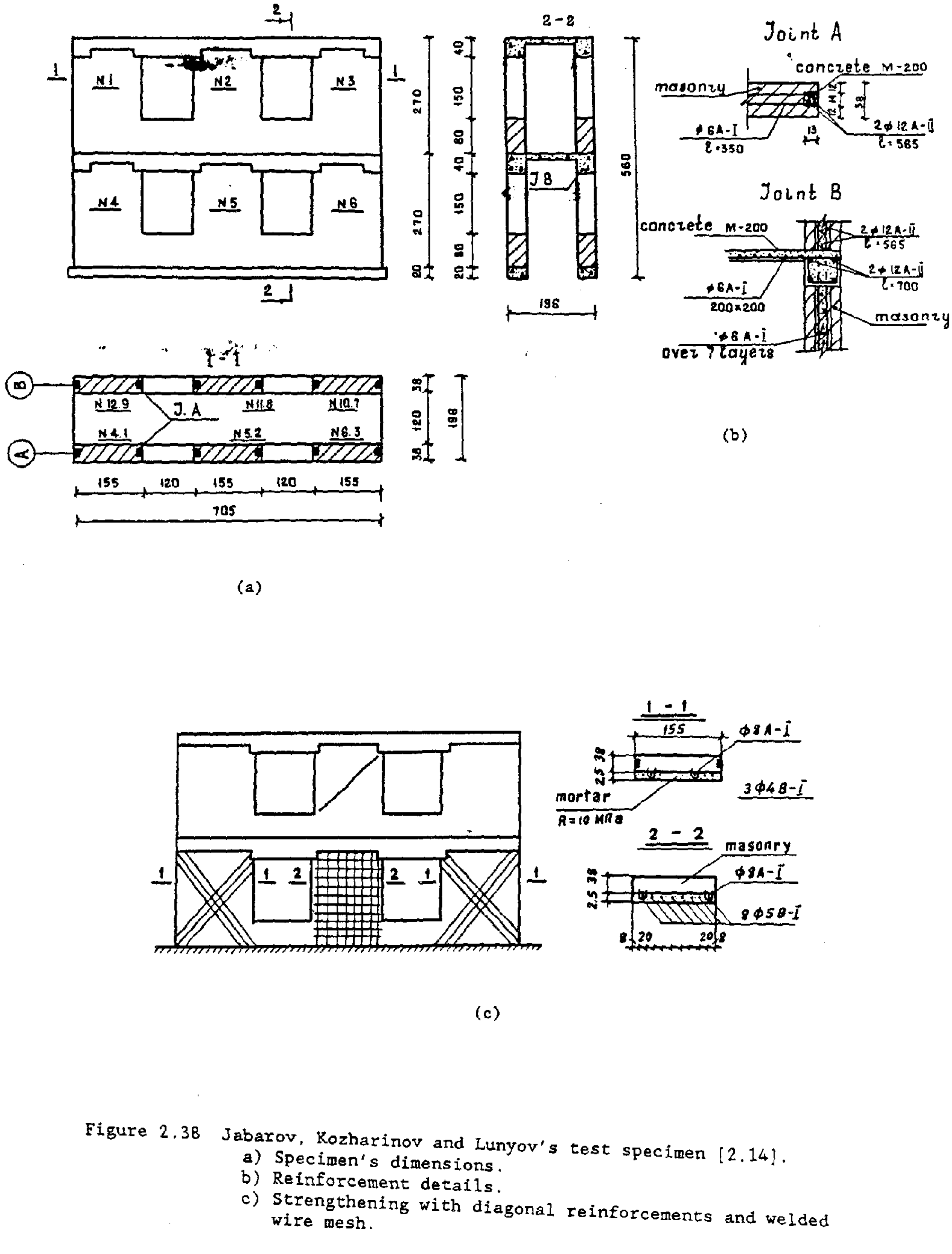


Figure 2.38 Jabarov, Kozharinov and Lunyov's test specimen [2.14].
a) Specimen's dimensions.
b) Reinforcement details.
c) Strengthening with diagonal reinforcements and welded wire mesh.

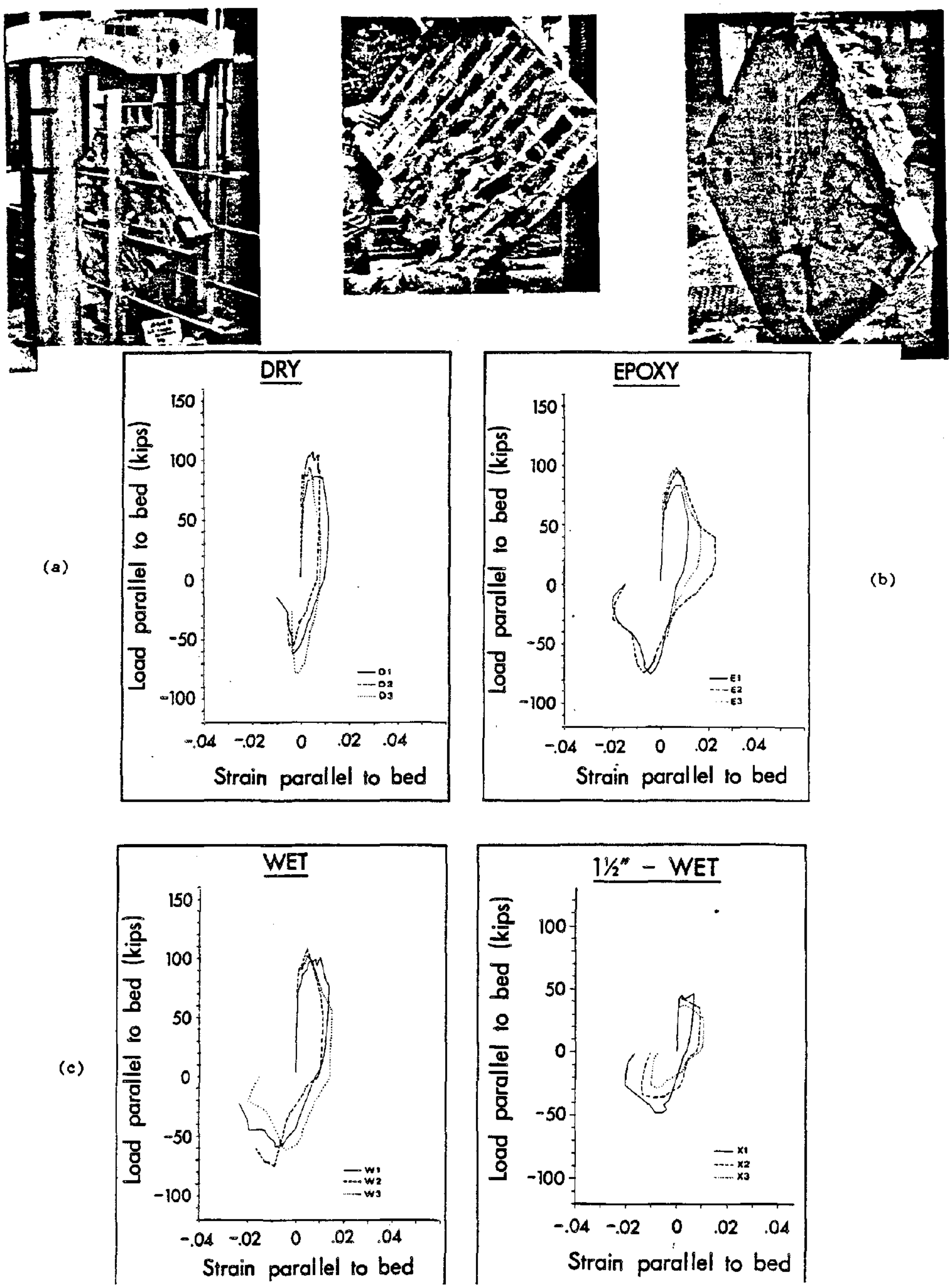
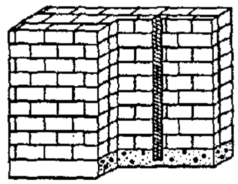
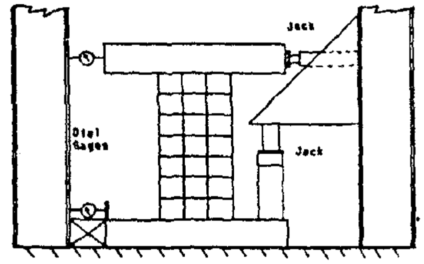


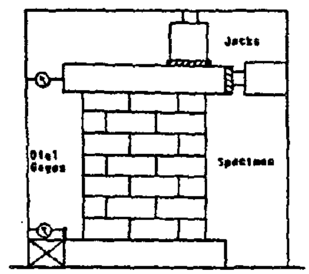
Figure 2.39 Load parallel to bed joints vs. strain parallel [2.15] to bed joints relationships for Kahn's wall panels.
a) Dry surface condition.
b) Epoxied surface condition.
c) Wet surface condition.



Cut-out view of NCSU's test specimen.



Out-of-plane test setup.



in-plane test setup.

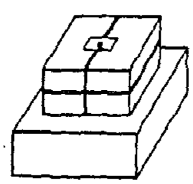


Figure 2.40 Plecnik, Cousins and O'Conner's static shear specimens (LBSU's testing program) [2.13].

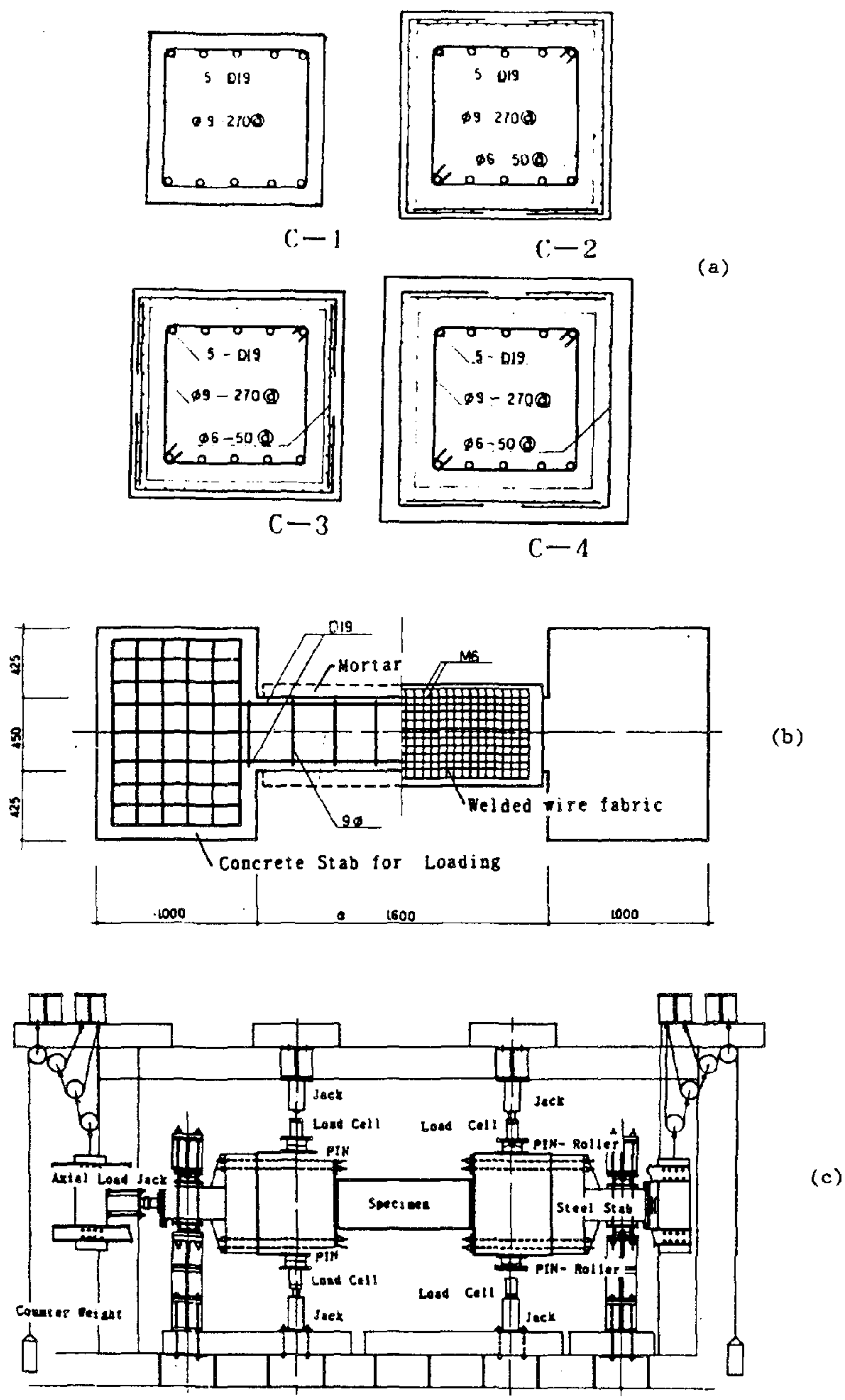
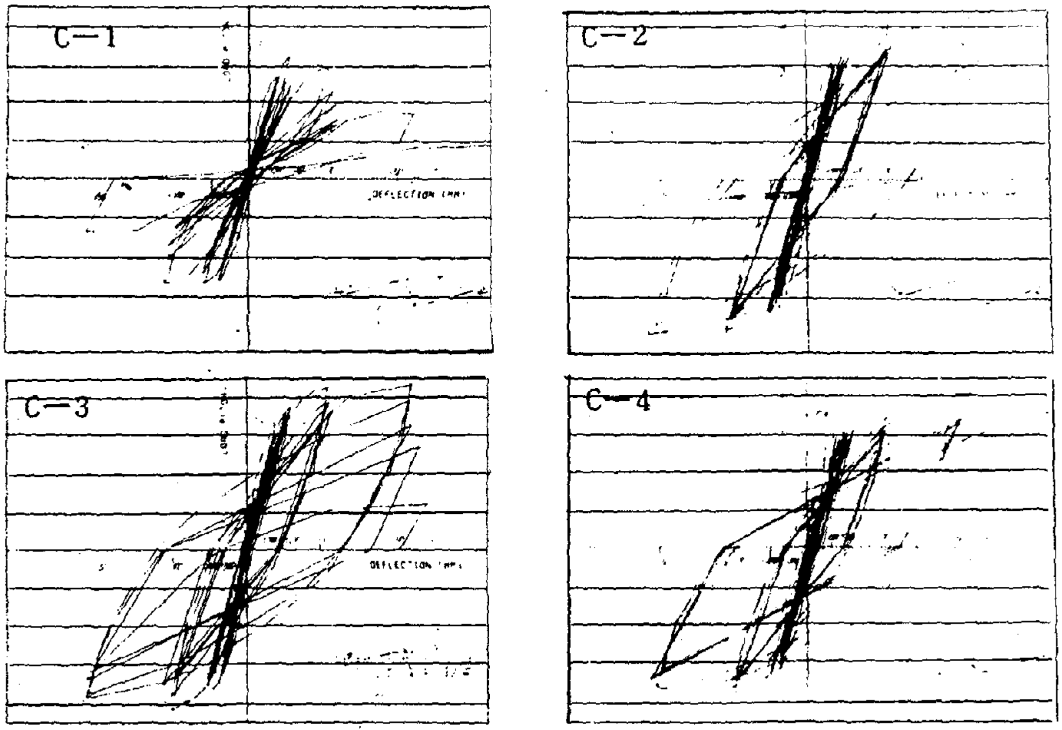
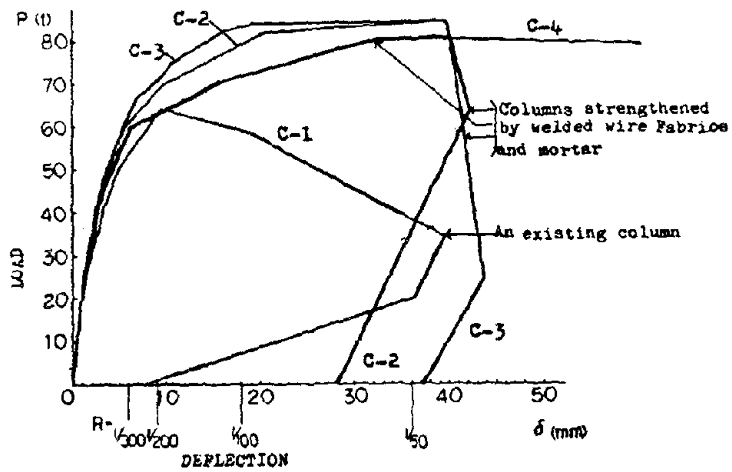


Figure 2.41 Hayashi, Niwa and Fukuhara's column specimens [2.1].
a) Column cross-section.
b) Reinforcement arrangement.
c) Test setup.



(a)



(b)

Figure 2.42 Hayashi, Niwa and Fukuhara's test results [2.1].
a) Hysteresis curves.
b) Envelopes of hysteresis curves.

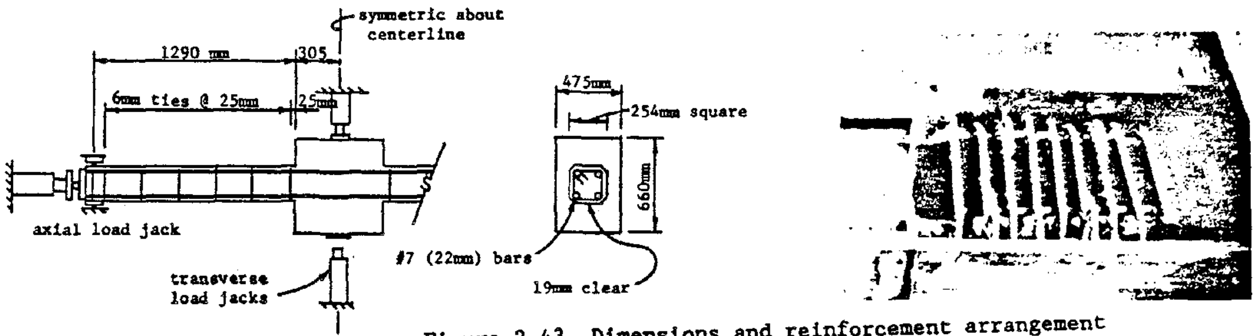


Figure 2.43 Dimensions and reinforcement arrangement of Kahn's column specimens [2.16].

Figure 2.44 Strengthening using steel bands (specimen 2).

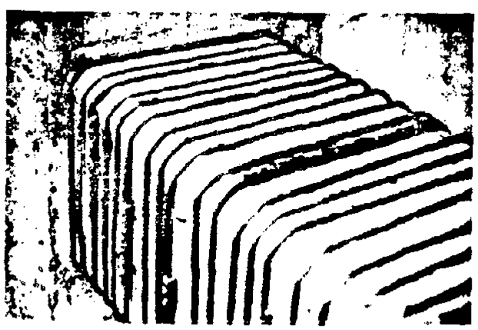


Figure 2.45 Strengthening using plain steel rod (specimen 3).

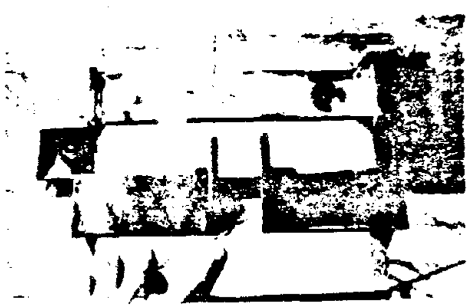


Figure 2.46 Strengthening using U-shaped clamps (specimen 4).

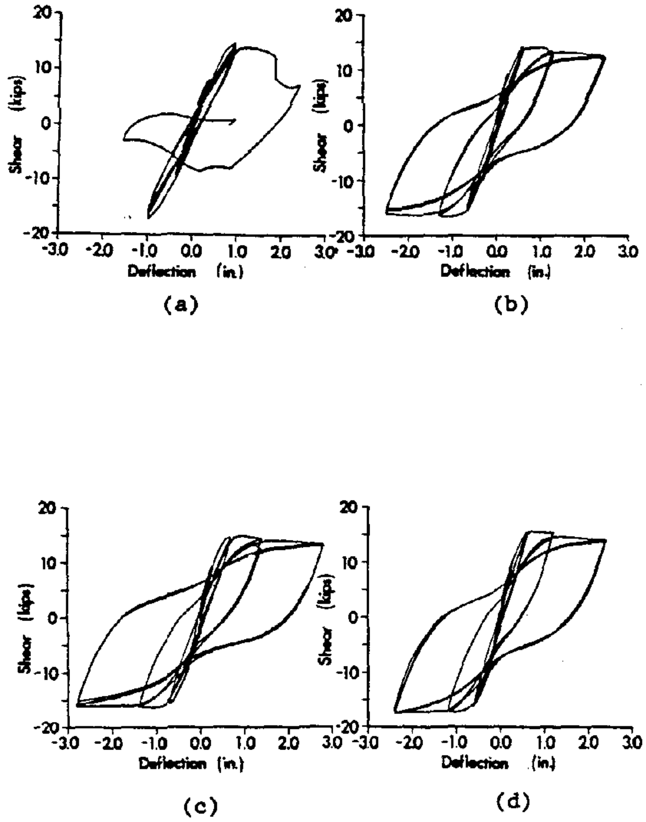


Figure 2.47 Hysteresis curves of Kahn's columns [6].
a) Specimen 1, unstrengthened.
b) Specimen 2, steel bands.
c) Specimen 3, 6 mm plain rod.
d) Specimen 4, U-shaped clamps.

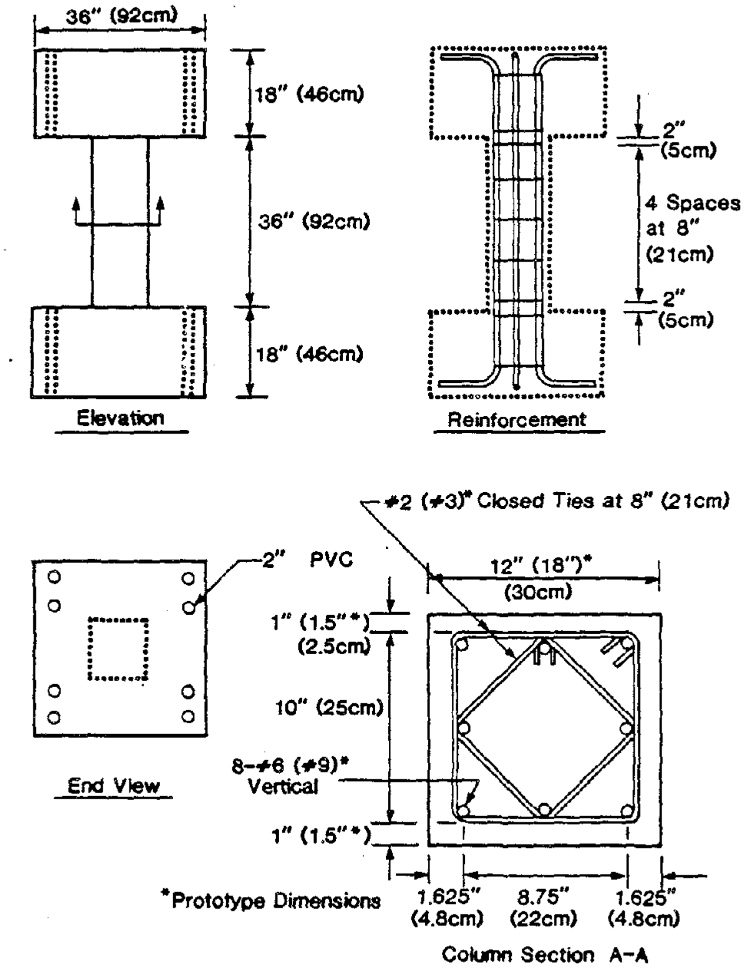


Figure 2.48 Details of Bett, Klingner and Jirsa's columns [2.17].

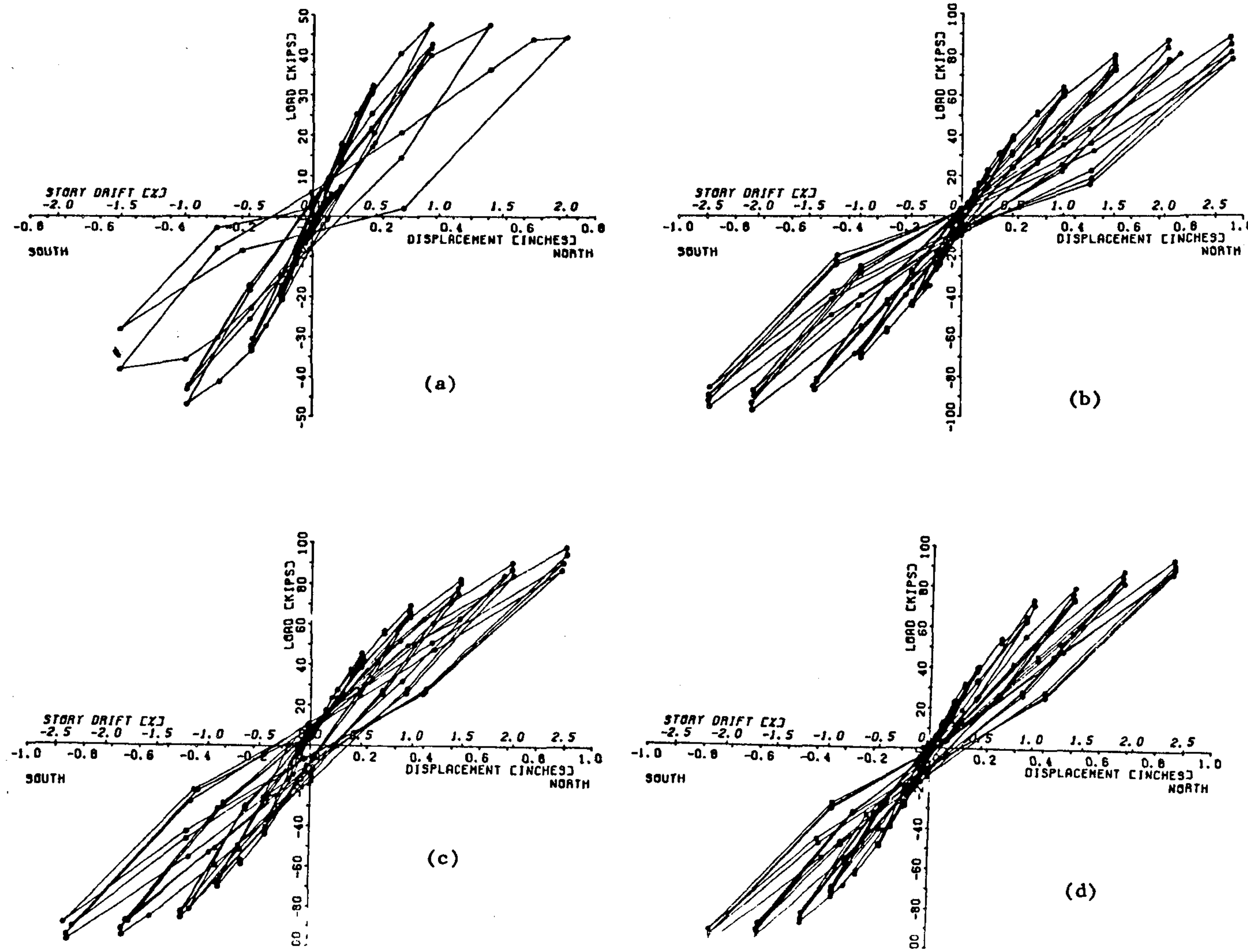


Figure 2.49 Hysteresis curves of Bett, Klingner and Jirsa test specimens [2.17].
a) Specimen 1-1.
b) Specimen 1-1R.
c) Specimen 1-2.
d) Specimen 1-3.

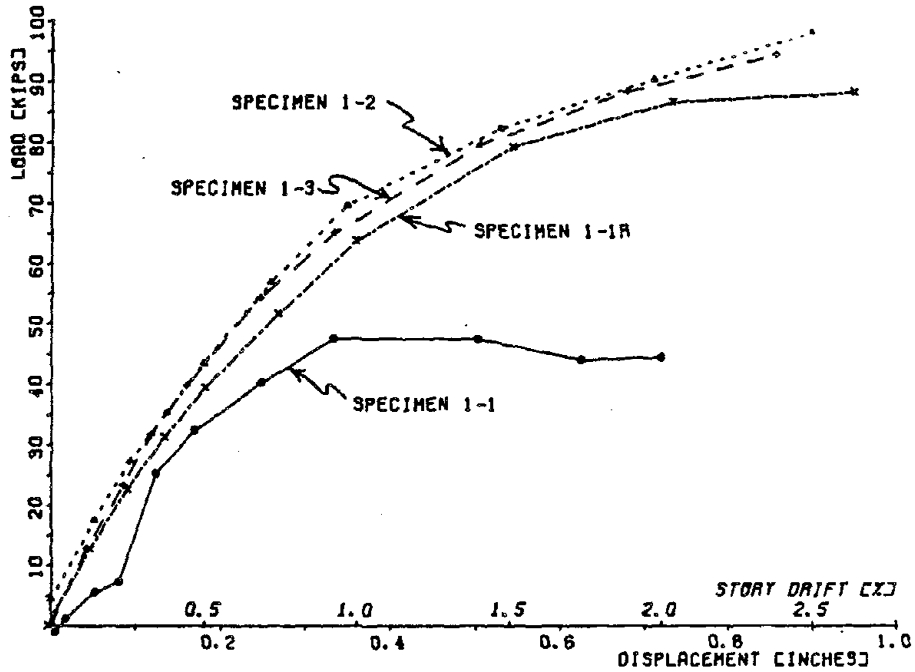


Figure 2.50 Envelopes of Bett, Klingner and Jirsa's hysteresis curves [2.17].

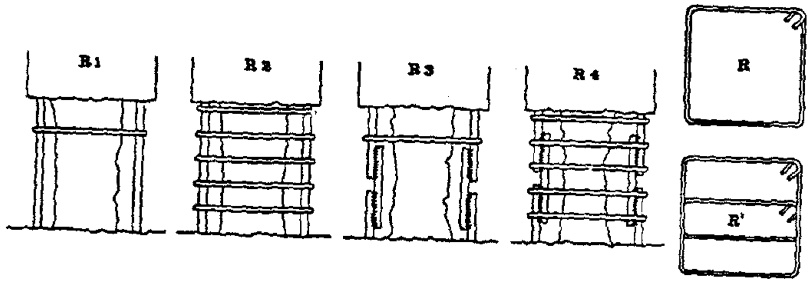


Figure 2.51 Reinforcement repair schemes in Augusti, Focardi, Giordano and Manzini's test program [2.18].

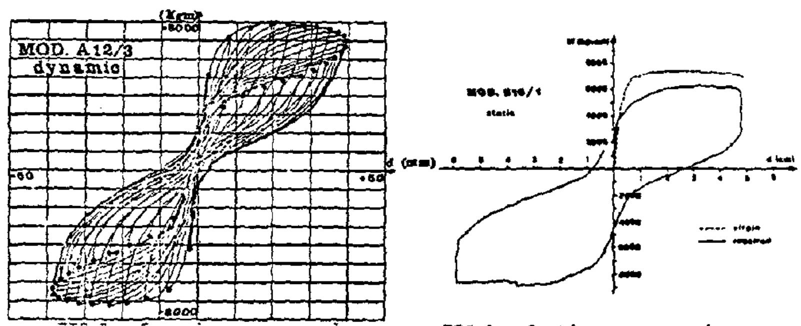


Figure 2.52 Typical moment-displacement relationship [2.18].

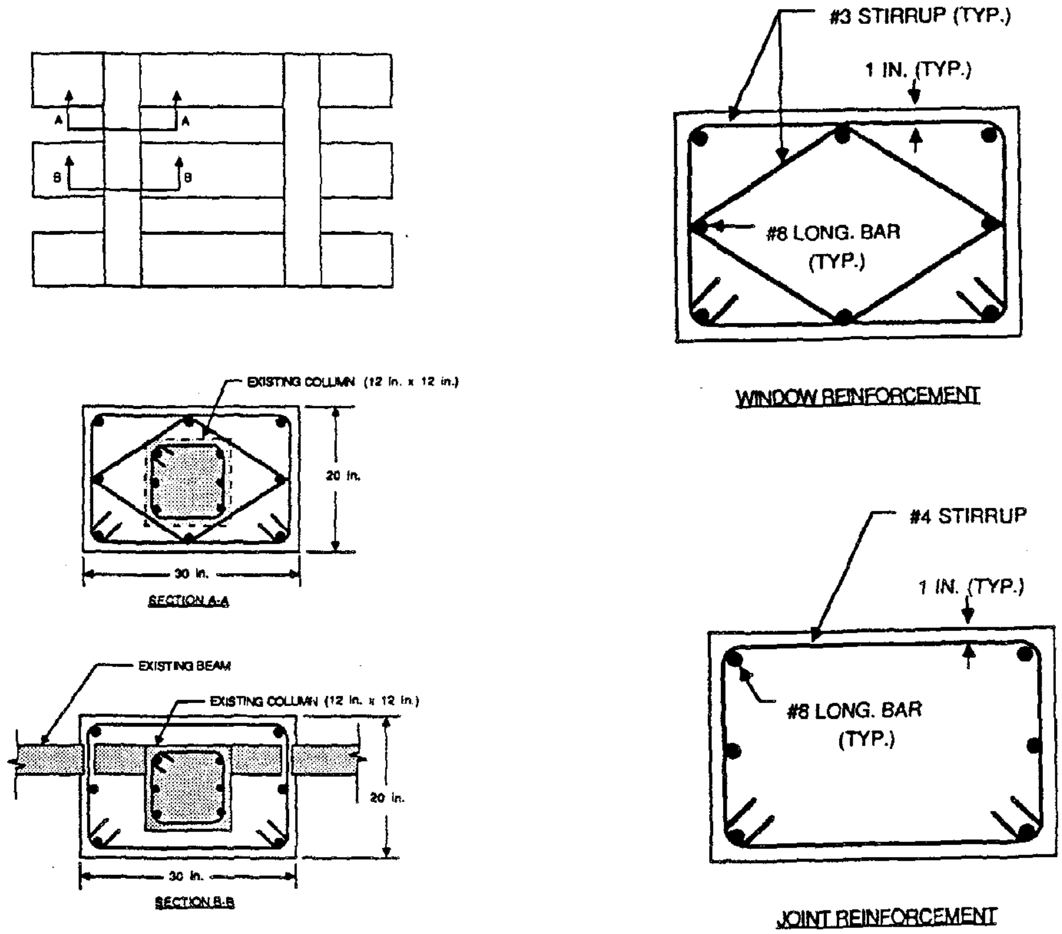


Figure 2.53 Reinforcement details of Stoppenhagen and Jirsa's encased columns [2.19].

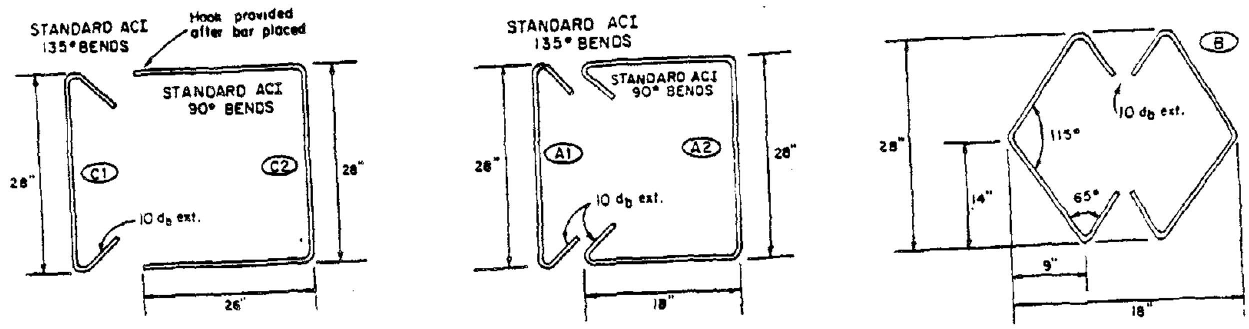
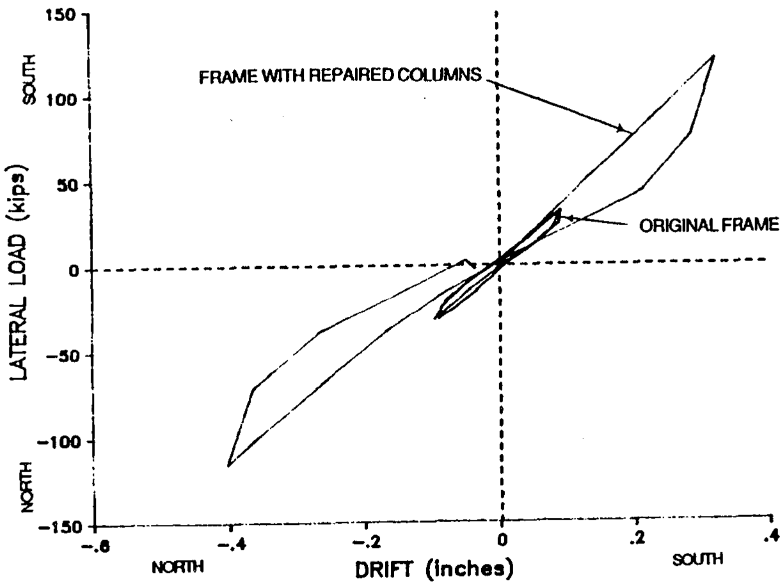


Figure 2.54 Shear reinforcements [2.19].
a) Bent #4 bars in beam-column joints.
b) Bent #3 bars in window regions.



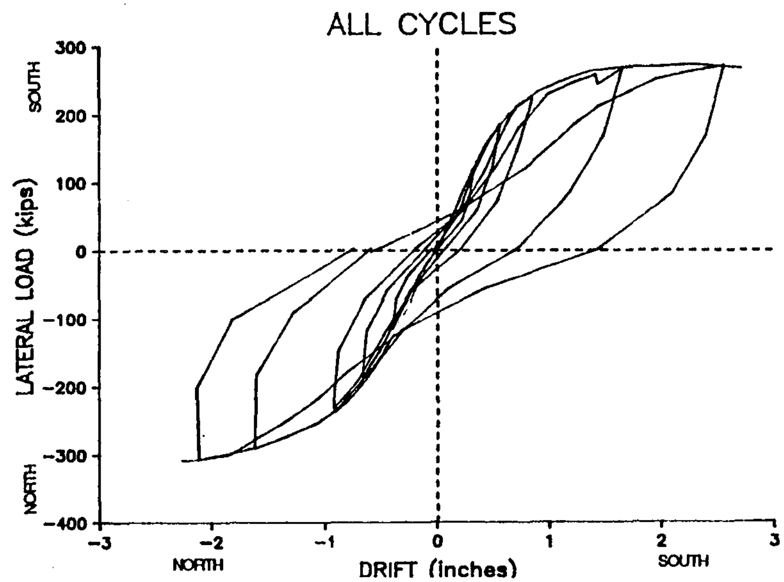


Figure 2.55 Load-drift relationships [2.19].

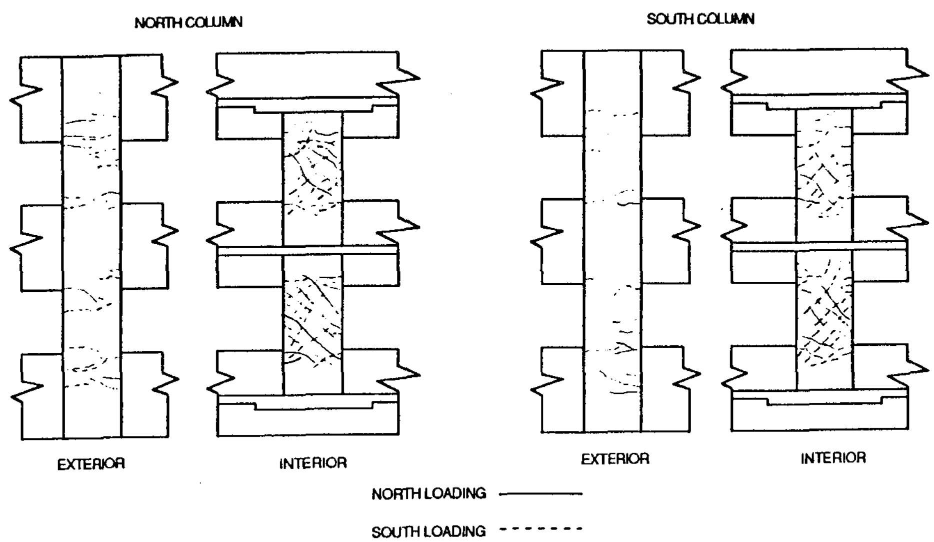


Figure 2.56 Final crack patterns [2.19].

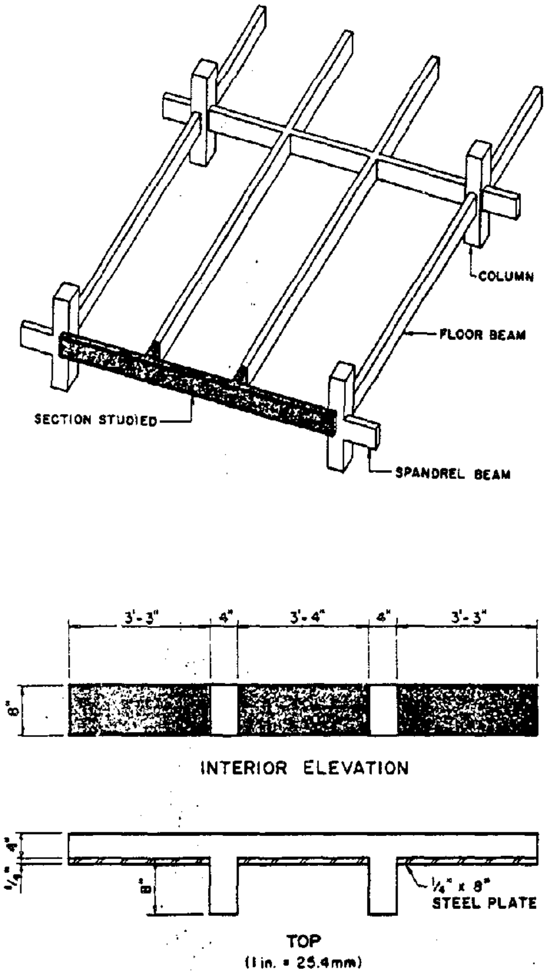


Figure 2.57 Specimen configuration and dimensions in Holman and Cook's test program [2.20].

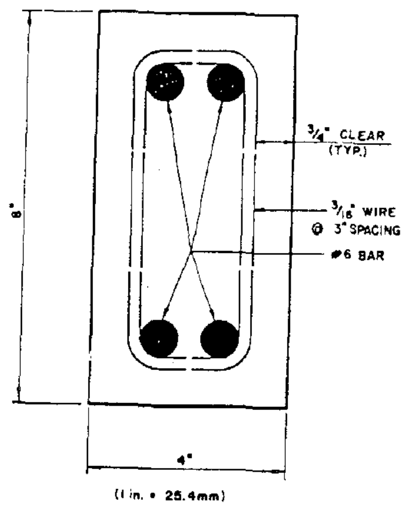


Figure 2.58 Reinforcement arrangement on beam's cross section.

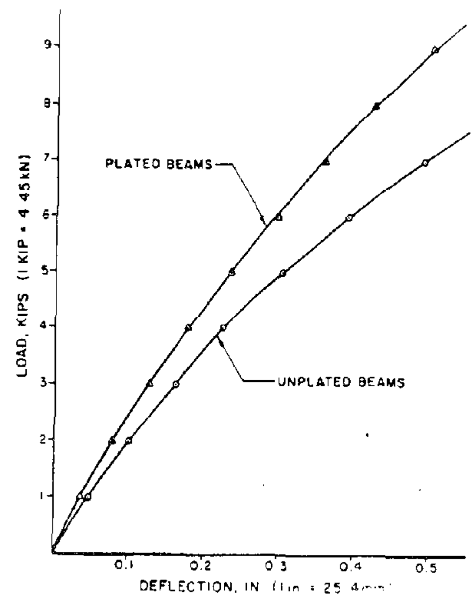


Figure 2.59 Load-center deflection curves [2.20].

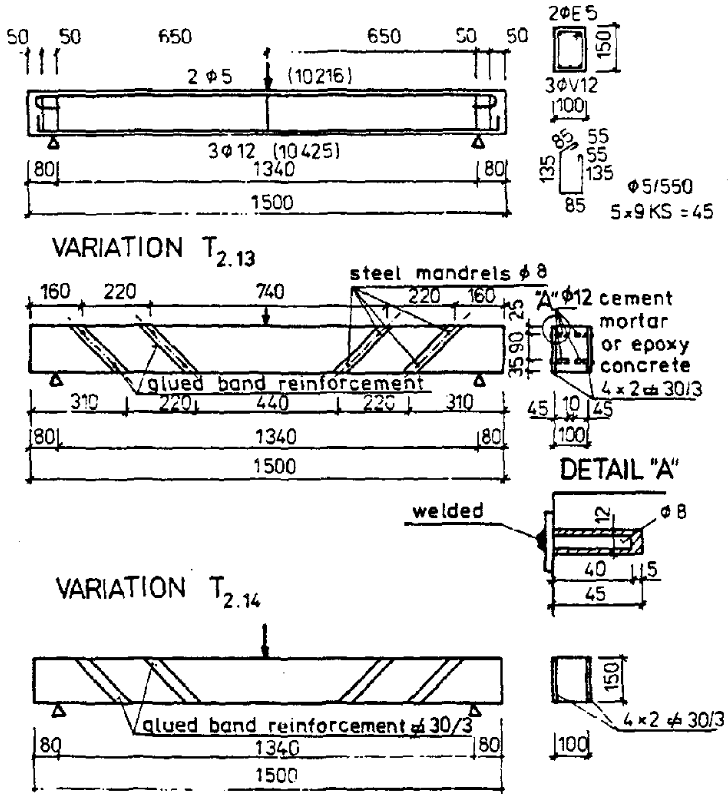


Figure 2.60 Arrangement of external reinforcement in Vanek's study [2.21].

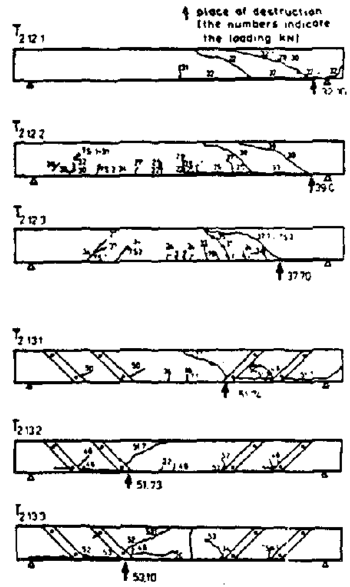


Figure 2.61 Crack patterns in beams [2.21].

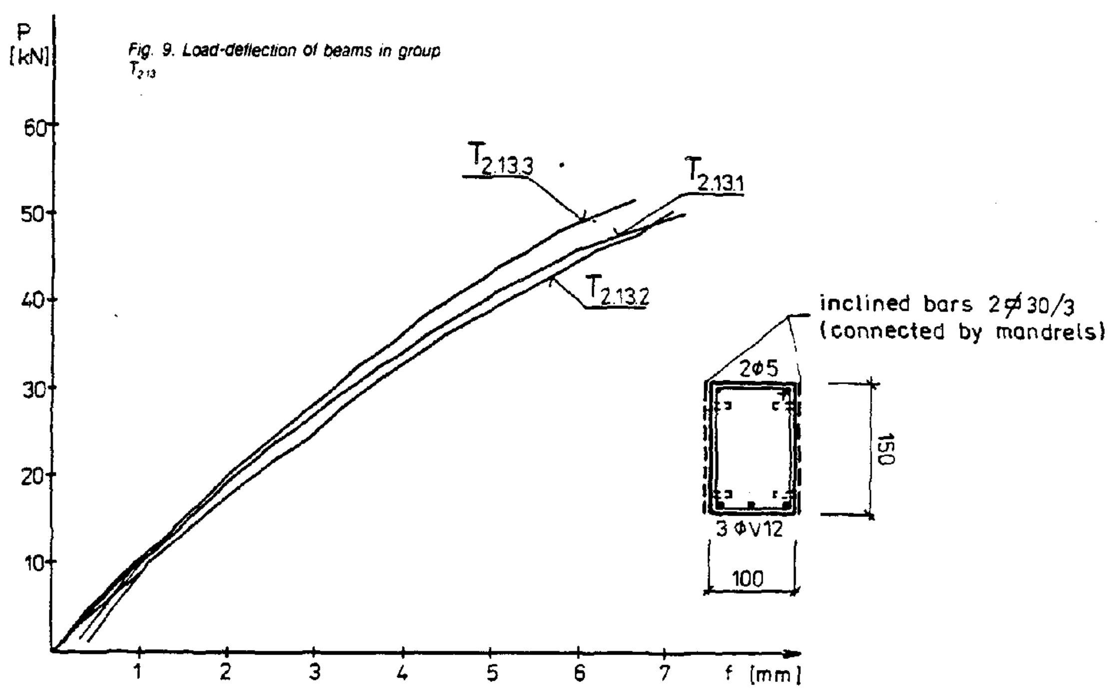


Figure 2.62 Load-center deflection of plated beams.

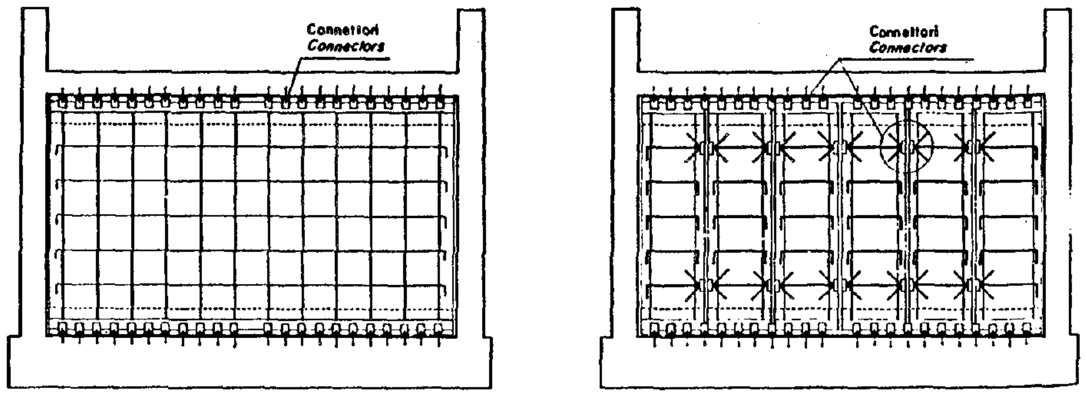


Figure 3.1 Concept of infilling technique.

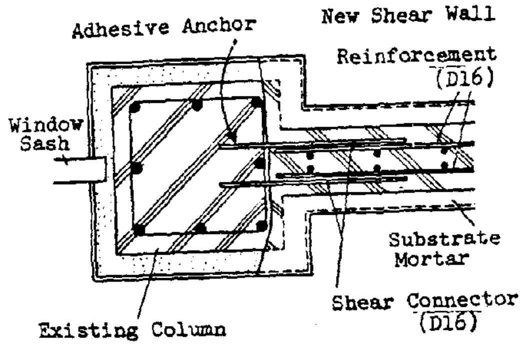


Figure 3.2 Connection between new shear wall and column [3.1].

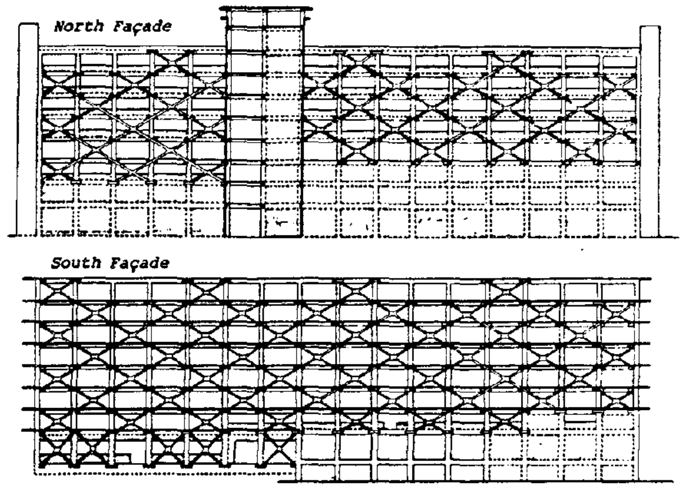


Figure 3.3 Arrangement of cross braces on North and South facades of the Tohoku Institute of Technology in Sendai, Japan [3.2].

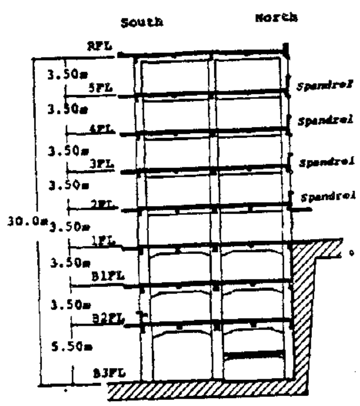


Figure 3.4 Spandrel beam locations [3.2].

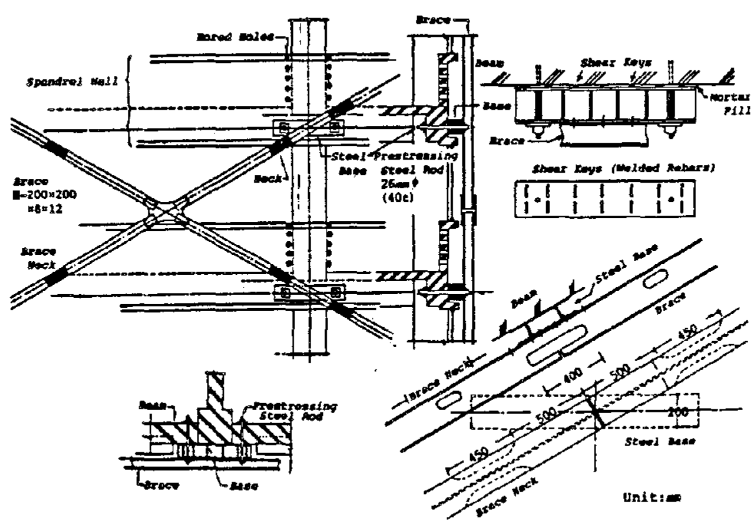


Figure 3.5 Details of braces to frame connection [3.2].

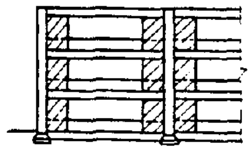


Figure 3.6 Concept of wingwall addition.

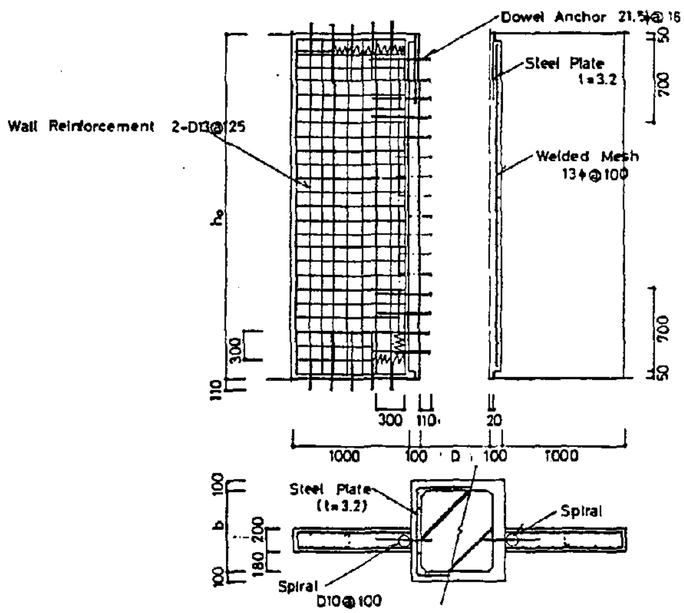


Figure 3.7 Connection between wingwall and column.

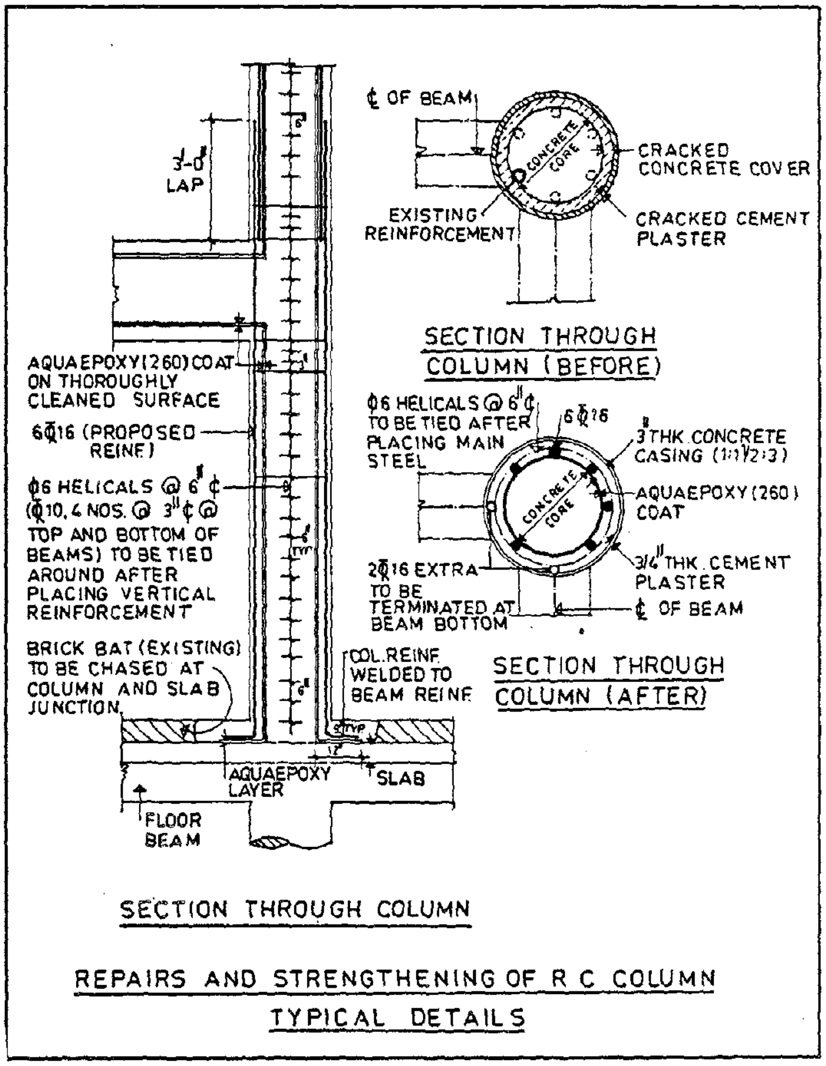


Figure 3.8 Details of Nene's strengthened column [3.4].