Theme Report on Topic 7
REPAIR AND STRENGTHENING OF STRUCTURES

by

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Of the 14 papers on this topic, seven papers describe research tests to determine the efficiency of various methods of repair. The other papers generally describe specific repair procedures that have been used or that may be used on specific structures or types of structures.

One paper "Collapse Analysis of Multistory Buildings" by Dr. Selna (7-11)* presents a method for evaluating the collapse potential of a damaged concrete frame structure.

This is a step by step analysis which accounts for many of the factors presently neglected in determining lateral stability. It should be useful as a research tool. However, when a building is damaged by an earthquake, generally new and stronger elements are introduced into the system so that a detailed analysis of the original damaged frame becomes unnecessary.

Three papers examine methods of repair of concrete frame members. Dr. Gulkan, in his paper (7-10), reported on two interior type column-beam joints that were tested, then repaired by chipping off the concrete cover of the columns and casting a new reinforced concrete shell and then retested. The tests were designed so that the beams were much stronger than the columns so that failure always occurred in the columns. The columns were kept vertical and axial forces applied at the ends. Unfortunately, the paper does not give the amount of axial force.

It was found that the shell of the repaired column acted essentially monolithically with the original concrete and that the strengths could be adequately predicted. In Figure 1 the critical points of failure of 3N at the right was in the column above and below the joint. Special precautions were taken to prevent

* The numbers in brackets are the numerical designations for papers in the preprint volume. The first numeral refers to the theme and the second set refers to the serial number of the paper in the theme.

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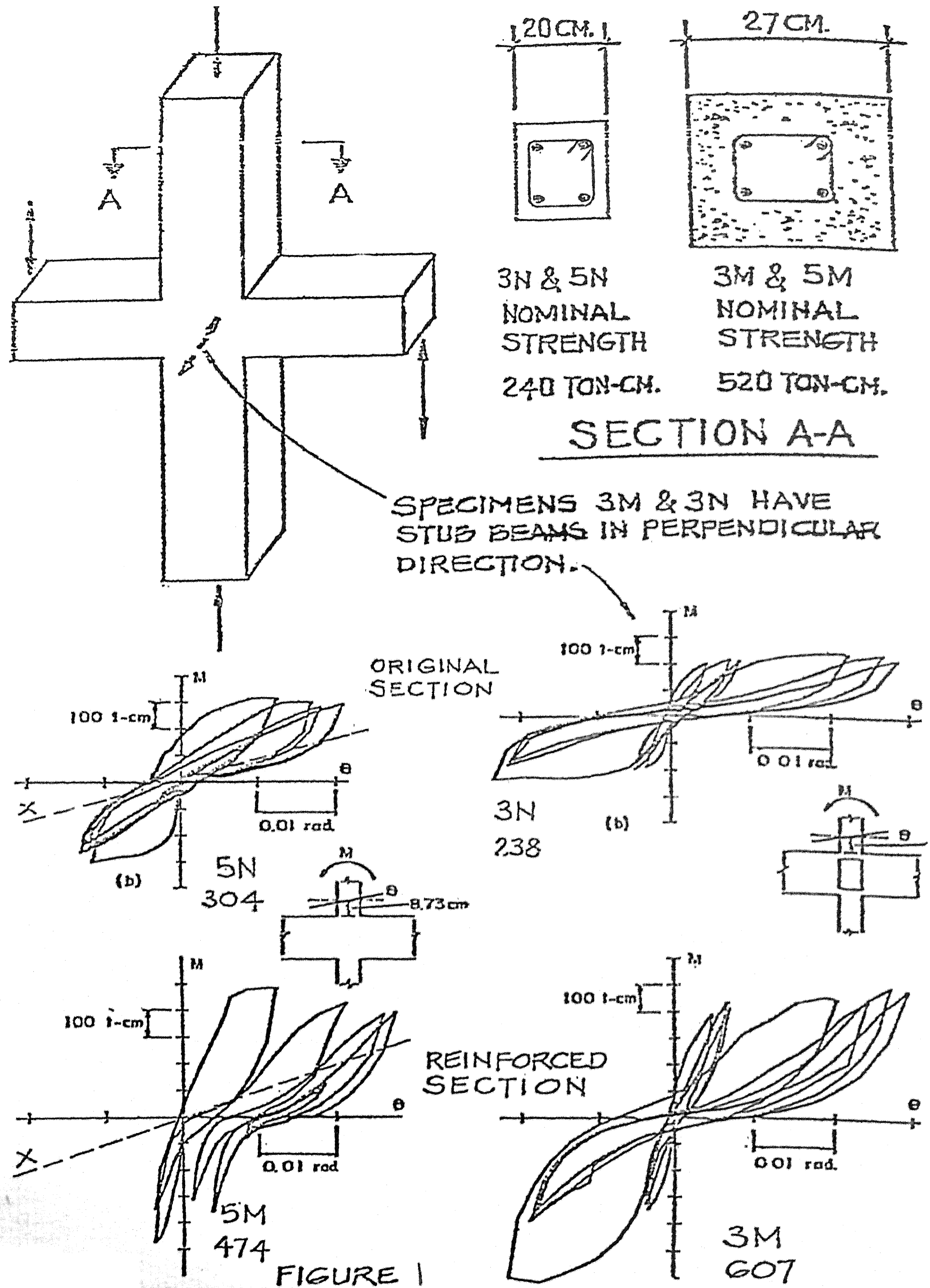


A
A
20 CM.
27 CM.
3N & 5N
NOMINAL
STRENGTH
240 TON-CM.
3M & 5M
NOMINAL
STRENGTH
520 TON-CM.
SECTION A-A
SPECIMENS 3M & 3N HAVE
STUB ~~BEAMS~~ IN PERPENDICULAR
DIRECTION.
ORIGINAL
SECTION
M
100 t-cm
θ
0.01 rad
X
(b)
5N
304
3N
238
8.73 cm
REINFORCED
SECTION
5M
474
3M
607

FIGURE 1

buckling of column bars at the panel zone of the joint but what these precautions were is unknown. The stub beams of 3N helped carry the panel shears. In specimen 5N at the left without the stub beams extensive cracking occurred in the panel zone and was the cause of failure, without developing the full capacity of the column.

While strengths could be improved by the reinforcing, the repaired joints were not as rigid as the original undamaged joints. As is customary for most tests of this type, the results are plotted as in Figure 1 without the $P\Delta$ effect. If the axial loads in the columns were not in line as would be the case in an actual building, the base line of the hysteresis curve would be inclined as shown dotted and marked "X". The slope of this base line for these tests is unknown since the axial loads were not given in this paper. With the $P\Delta$ effect, the loss of rigidity of the joint becomes much more important than indicated in the hysteresis curves presented.

Similar tests were performed on exterior joints by Lee, Wight and Hanson (7-12 but in this case the beam was designed to be the weak element and two levels of damage were introduced. Repairs were made by the injection of epoxy in moderately damaged joints and replacement of damaged concrete with various materials in the heavily damaged joints. Although eight specimens were tested, only three are reported in this paper. Figure 2 shows a summary of the three tests reported. Specimens 1 and 2 were constructed in accordance with recommendations from ACI Committee 352 on the design of joints for earthquake resistance and tested with a 40 kip axial load. Specimen 1 was loaded moderately and then repaired with epoxy and retested. The repaired specimen was slightly stronger but less stiff than the original. Specimen 2 was loaded severely, broken concrete removed and the area repacked with very strong high early strength concrete. When loaded, it was substantially stronger and stiffer than the original due to the stronger concrete in the repaired portion. In the curves of Figure 2, the dotted lines indicate the reduction in available strength due to $P\Delta$. Specimen 6 had less transverse steel than Specimens 1 or 2 with detailing more like that used in

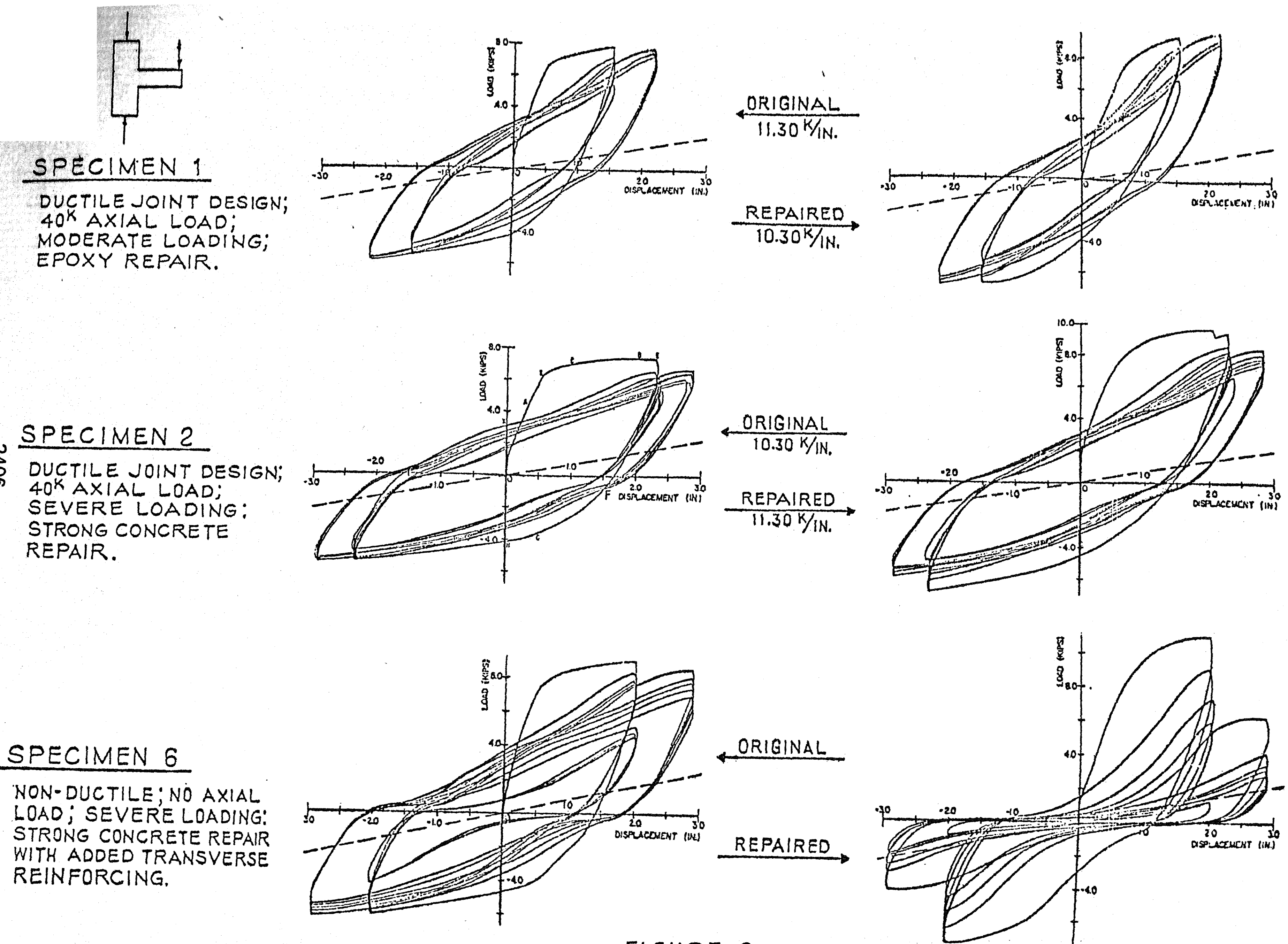

SPECIMEN 1
DUCTILE JOINT DESIGN; 40K AXIAL LOAD; MODERATE LOADING; EPOXY REPAIR.
ORIGINAL 11.30 K/IN.
REPAIRED 10.30 K/IN.
SPECIMEN 2
DUCTILE JOINT DESIGN; 40K AXIAL LOAD; SEVERE LOADING; STRONG CONCRETE REPAIR.
ORIGINAL 10.30 K/IN.
REPAIRED 11.30 K/IN.
SPECIMEN 6
NON-DUCTILE; NO AXIAL LOAD; SEVERE LOADING; STRONG CONCRETE REPAIR WITH ADDED TRANSVERSE REINFORCING.
ORIGINAL
REPAIRED
LOAD (KIPS)
DISPLACEMENT (IN.)

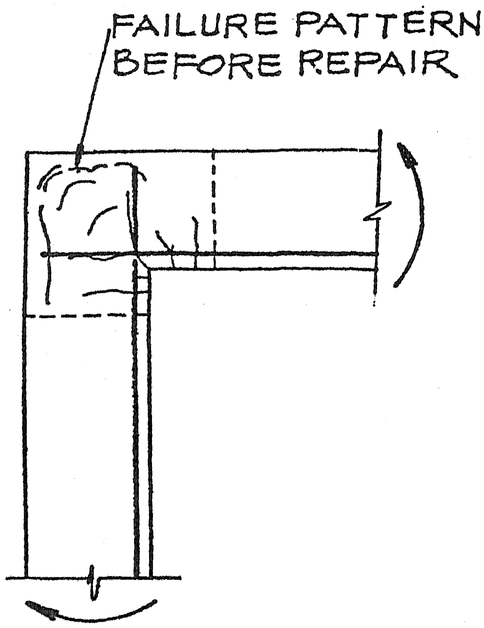
FIGURE 2

non-seismic areas. It was loaded severely, and broken concrete removed and replaced with high strength concrete similar to Specimen 2. The stronger portion of the beam forced failure into the panel zone and the consequent failure of the joint indicated a much less satisfactory cyclic performance than the original.

The third paper on repairs is by Tassios, Plainis and Vassiliou (7-19) and includes the repair of corner joints under opening loads and the repair of flexural damage to beams. Knee joints without diagonal stirrups or corner reinforcing (Figure 3) were tested and suffered premature brittle failure under low loads. Damaged concrete was removed and replaced with the addition of external stirrup collars with slight prestressing. Retesting showed that the repaired joint developed 90 to 100% of the original strength, but it was capable of resisting many cycles of load due to the added collar stirrups.

This paper also reports on the restoration of beam bending capacity. Beams were tested and damaged in bending and the broken concrete removed and recast. A steel sheet was epoxy glued to the bottom of the recast section. The original strength was fully restored but the rigidity decreased considerably.

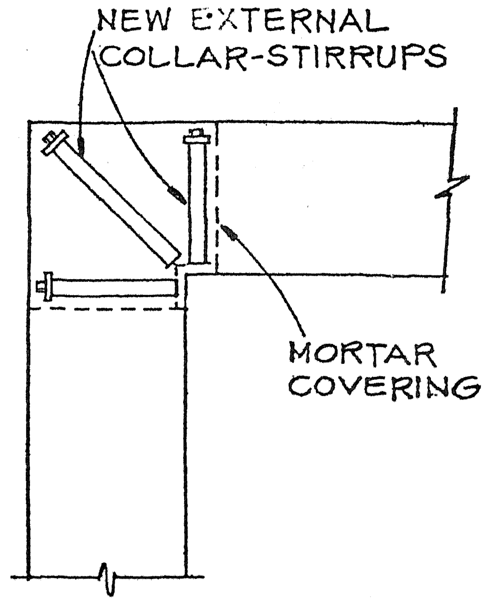
Four papers consider and test various methods of repairing shear walls or of reinforcing frames by adding shear walls of various types. The first paper of this group is by Plecnik, Amrhein, Warner, Jay and Chelapati (7-13) and it summarizes the results of many tests of the epoxy repair of cracked masonry block shear wall elements including the effects of static loads, dynamic loads and elevated temperature conditions. Approximately 240 small scale tests were run with compression tests at various crack angles and direct shear tests. For nearly all of the tests, epoxy debonding was not observed. The authors conclude that concrete masonry block walls can be effectively repaired with epoxy if certain precautions are observed. However, under fire exposure, the strength properties of the epoxy-repaired structural components is reduced extensively both during and after fire exposure. Tests show that epoxy strength beyond 400°F is nearly zero. Under a two hour fire exposure, a 6 inch concrete or



(a)

ORIGINAL JOINT

FAILED AT 75% (STATIC) & 65% (DYNAMIC) OF EXPECTED LOADS.

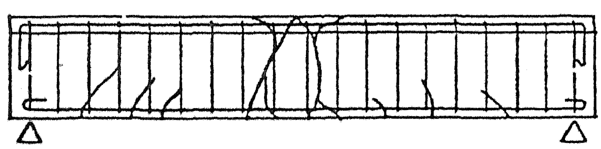


(b)

REPAIRED JOINT

90% TO 100% OF THE ORIGINAL STRENGTH WAS DEVELOPED.

(a)



FAILURE PATTERN BEFORE REPAIR

(b)

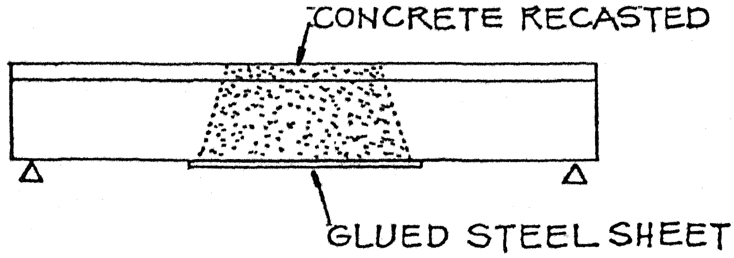


FIGURE 3

masonry block wall may lose 80% of its strength. This is a point that designers should remember in their choice of repair methods.

Doctors Gyoten, Mizuhata and Fukusumi (7-10) tested the efficiency of epoxy repair of 40 mm thick by 1 meter square shear walls inside a heavily reinforced boundary frame. Three levels of wall reinforcing were considered - unreinforced and with 0.195% and 0.389% of the cross sectional area. After cyclic testing of the original panels, the resulting visible cracks were grooved, cleaned and filled with epoxy mortar. After curing, the panels were retested to destruction.

It was found that the repaired panels had nearly 100% of the ultimate strength of the original ones. The initial stiffness was much less (90% of the original for the 0.195 and 50% for the 0.389%) probably due to the unseen and unrepaired cracks and loss of bond. Final stiffness at ultimate was almost the same for both the original and repaired panels. Failures were not brittle and adequate ductility was achieved. Equivalent viscous damping was almost 9%. In repairs with epoxy, it must be remembered that in spite of epoxy's high strength, it is not as rigid as concrete, hence deflections must be somewhat larger.

The third paper dealing with shear walls is that of Kahn and Hanson (7-04) where original concrete frames were strengthened by the addition of different types of infilled walls. Figure 5 shows the walls and the non-dimensional results given by the authors. Wall 1 was a 3 inch monolithically cast wall for comparison purposes. Wall 2 was a similar wall and a concrete infill wall cast later with a drypack space at the top and grouted dowels on all four perimeter sides of the wall. Wall 3 was a precast wall that was bolted to the top and bottom beams with wedge anchors. A gap was left at columns and there were no bolts to the columns for transferring the vertical reaction of the shear. This vertical component of the shear in the panel evidently caused the anchors to pull out at failure. Panel 4 consisted of six precast wall elements joined at the sides by welding and bolted to the frame top and bottom. The authors conclude that the latter two panels deteriorated less under cyclic loading and that Panel 3 gave the largest strength in non-dimensional terms. However, the non-dimensional terms include a "d" of 75 inches for Panels 1

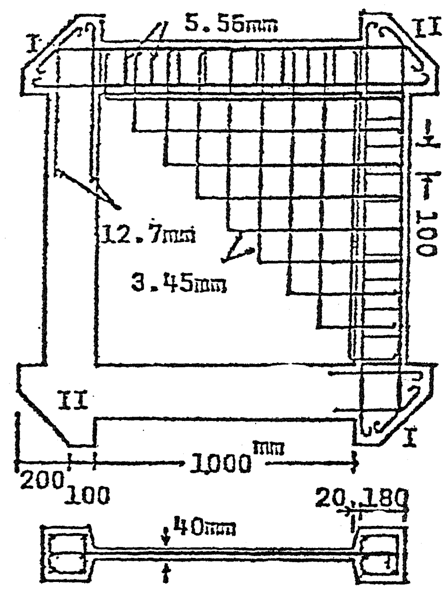


Fig. 1 Specimen
(in CASE RCW-A)

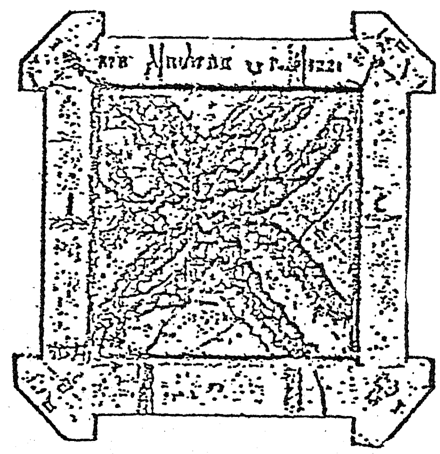
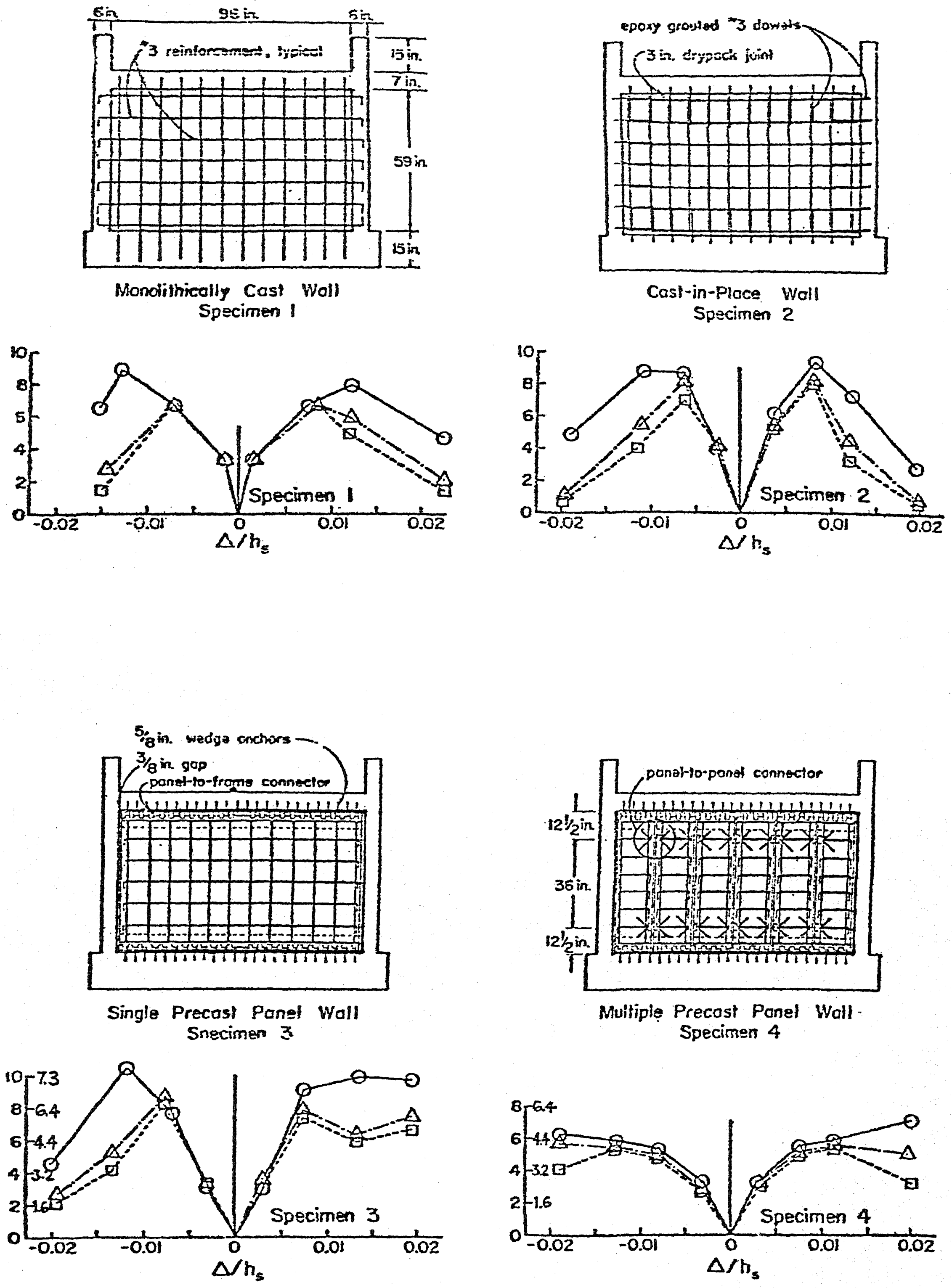


Photo.1 Specimen after Test
(in CASE RCW-B-II)

FIGURE 4



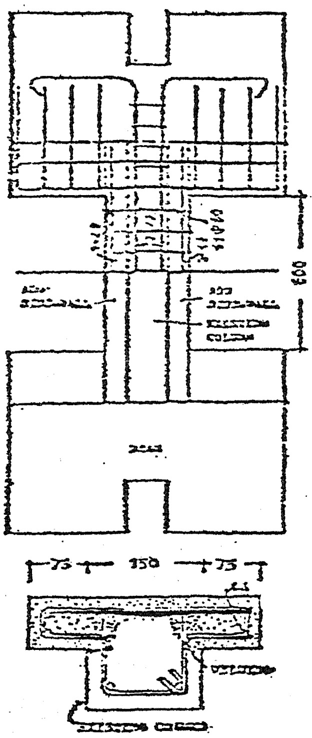
6 in.
95 in.
6 in.
#3 reinforcement, typical
15 in.
7 in.
59 in.
15 in.
Monolithically Cast Wall
Specimen 1
epoxy grouted #3 dowels
3 in. drypack joint
Cast-in-Place Wall
Specimen 2
Specimen 1
Specimen 2
Δ/h_s
5/8 in. wedge anchors
3/8 in. gap
panel-to-frame connector
Single Precast Panel Wall
Specimen 3
panel-to-panel connector
12½ in.
36 in.
12½ in.
Multiple Precast Panel Wall
Specimen 4
Specimen 3
Specimen 4

FIGURE 5

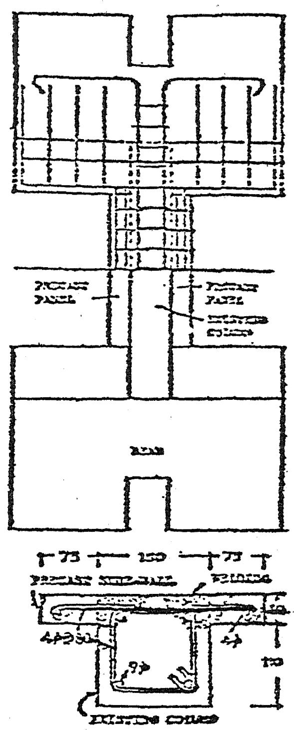
and 2 and a "d" of 56 inches and 55 inches for Panels 3 and 4. This term is unexplained and your reporter cannot find a combination of dimensions in the paper that explain their derivation. Given a certain size column and beam spacing, it is obvious that Panels 1 and 2 were by far the strongest and stiffest. Also it seems curious that by omitting one component of the shear stress as in Panels 3 and 4 (connection to the columns) that a greater strength, even though non-dimensional, can be attained. Again, we see some of the difficulties of very short technical papers where space does not permit the presentation of the necessary detail for a full comprehension of all the test parameters.

The fourth and last paper on walls is by Higashi, Ohkubo and Fujimata (7-15) and again it concerns the strengthening of concrete frames by adding concrete shear panels. Three series of tests by adding wing walls to existing columns were conducted as shown in Figure 6. Either the poured-in-place walls - Type AC-A1 or the precast wings with welded and grouted junctures - Type PW-A2 - performed similarly to monolithic walls and gave remarkable increases in strength and stiffness. The wing wall type with mechanical fastenings - Type PA - did not act monolithically and according to the authors gave inadequate increases in strength and stiffness. The other series of tests (Figure 7) compares a monolithic wall panel between columns with precast panels fastened in with mechanical anchors. The stiffness before yield of the wall with the monolithic panel could not be approximated by the precast panels but the strengths were very close.

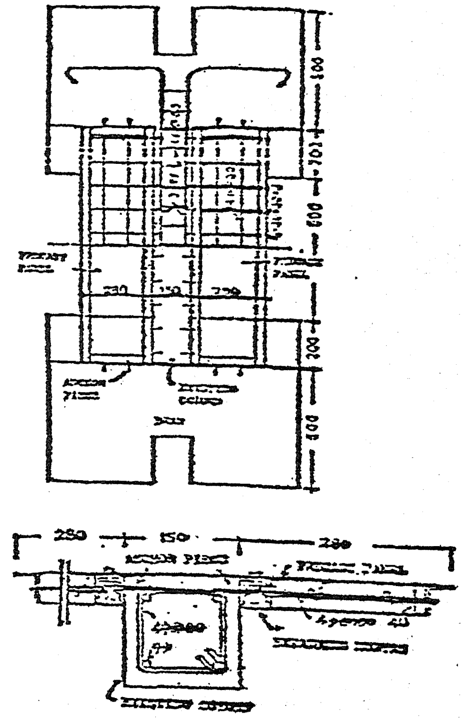
In all of the test series reported herein, only model tests varying from about 1/4 to 1/2 scale were performed. Especially when mechanical fasteners are used, the translation of these results from model to prototype must be approached with caution. It is one thing to develop the stresses in a 40 mm (1-1/2 inch) or even a 3 inch thick wall with mechanical anchors (5/8 inch ϕ at 9 inches o.c.) and quite another to develop a 12 inch (300 mm) or thicker wall as is often necessary in an actual building.



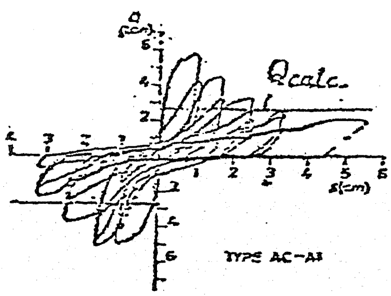
TYPE AC-A1



SERIES CW
TYPE PW-A2

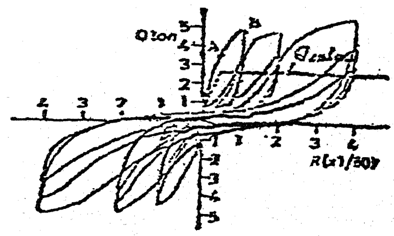


TYPE PA-4H1

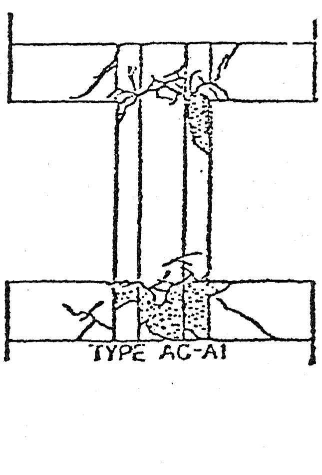


Q-δ CURVE
OF TYPE AC-A1

CURVE & CRACK
PATTERN SIM. TO
TYPE AC-A1



Q-δ CURVE
OF TYPE PA-4H1



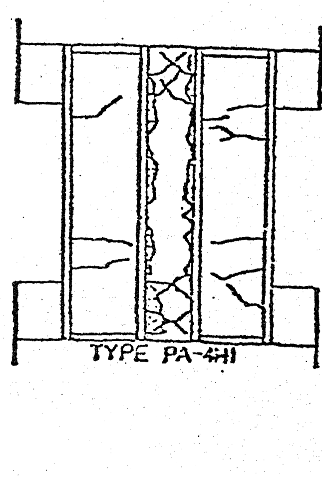


FIGURE 6

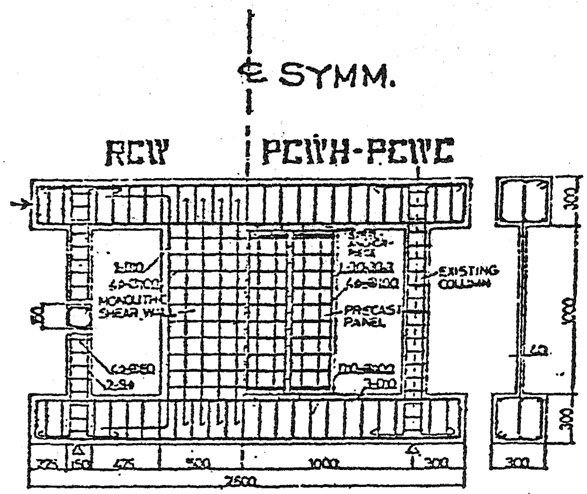


Fig.4 SPECIMEN OF SERIES-PCW

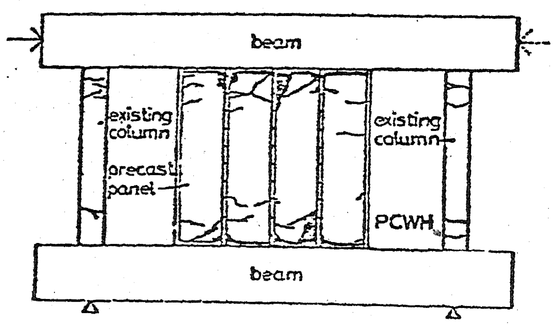


Fig.12 CRACK PATTERN OF PCWH, SERIES-PCW

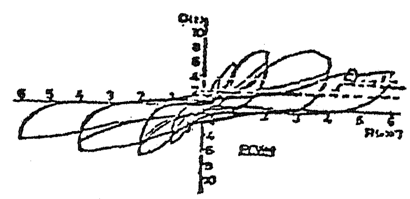


Fig.9 Q − δ CURVE OF SERIES-PCW, PCWH

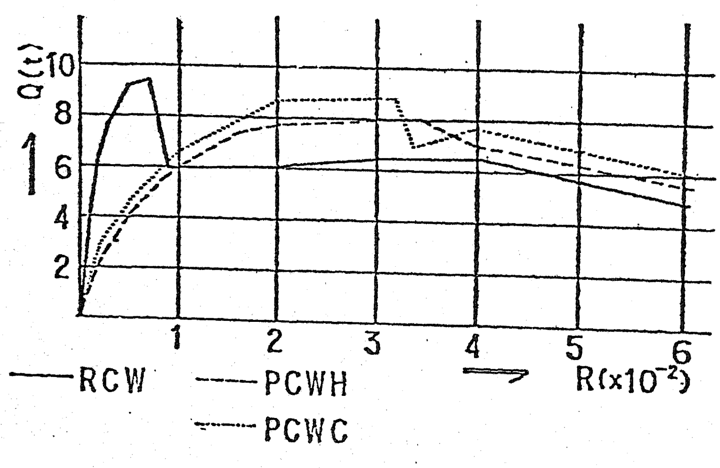


Fig.14 ENVELOPES OF Q-δ CURVES, SERIES-PCW

Two papers briefly describe general methods of repair and/or reinforcement of masonry buildings. One by Martemyanev (7-28) emphasizes both the lack of adequate connections and the lack of strength due to deterioration. By the use of shotcrete (gunite) and the installation of ties and bond beams. he and his associates have been able to increase the stiffness and strength of various 3, 4 and 5 story brick buildings.

The other paper by Guerra and Rienzo ( ) explores the possibilities of increasing the strengths of the walls by introducing a concrete mortar between two parallel skins (wythes) of rough blocks of tufa and of vertically prestressing the walls in order to increase the shear resistance. They discuss the principle of load-sharing or diffusion of stresses both as applied to entire wall systems and to parts of a wall.

Two papers, one by Freeman (3-20) and the other by Simeonov (7-23) discuss the strengthening of specific buildings. The latter example was required by inadequate concrete strength in the lower three floors of an eight story concrete apartment building. Reinforcement was supplied by adding extra concrete and steel encasement of frame members. In the former case an existing hospital structure was reinforced by adding a combination of shear walls and interconnecting spandrels

One of the final two papers of this Session presents the basis of the design decisions for the repair and strengthening of the Koyna Dam. This paper by Pant and Saxena (7-20) describes the dam, damage resulting from the 1967 earthquake and the restorative measures which were taken. In the final analysis the choice of these measures was guided by the observed behavior of the dam in the 1967 earthquake and a study of the profiles of dams that withstood earthquakes in other countries. The authors stress the need for a better understanding of the dynamic behavior of materials.

The final paper by Bresler, Graham and Sharpe (7-21) presents some recommended procedures for "emergency post-earthquake inspection and evaluation of damage in building" to assess the extent of damage and to evaluate the relative safety for continued occupancy. Your reporter finds these recommendations very complete but idealistic and believes that they are unattainable. They require advance planning, availability of construction drawings, and the thoroughness of the field investigation would require much more time than has been available in any previous earthquake situation. As an example of the time available for evaluating safety for occupancy, consider the fact that nine teams of engineers (total about 30 engineers) examined 120 public facilities (probably 300 separate buildings) in 3-1/2 days for the Guatemalan Government after their large earthquake. That is about three team hours per facility or one team hour per building. Similar time restraints were experienced in Bakersfield, Anchorage, Caracas, Managua and The Philippines.

While it would be nice to include in a report the use and occupancy of a building, foundation and soil conditions, the complete structural system, all materials used, finishes, repair cost estimates, etc., etc., under emergency conditions there just is not time. Possibly I misunderstand the authors' use of the term "emergency". If the procedures were to aid in <u>later</u> evaluation of the use of the building or in its demolition, there are many valid suggestions that serve as valuable reminders of potential problems and considerations. But, if they are to assist in making actual emergency decisions with regards to continued occupancy especially during inclement weather, we have not in the past been concerned with cost estimates, operation of the air conditioning system, etc., etc.