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SEISMIC STRENGTHENING OF A NONDUCTILE RC FRAME USING PRECAST INFILL PANELS

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ABSTRACT

A two-thirds scale model of a non-ductile reinforced concrete frame was strengthened using precast infill panels. The two-story structure was loaded to failure. The main deficiencies in the frame were inadequate column strength in shear and poor splice and anchorage details. The infill wall panels were designed to convert the lateral force system from a frame to a shear wall. Details of the infill wall system were determined from a series of separate tests involving panel-to-panel and panel-to-frame connections.

KEYWORDS

Reinforced concrete; non-ductile frames; strengthening; infill walls; precast panels; post-tensioning; design.

BACKGROUND

Many existing reinforced concrete moment-resisting frames lack strength and ductility. In order to correct these deficiencies, infill walls can be constructed to strengthen and stiffen the structure. Cast-in-place construction is often used; however, it may be costly, time consuming, and cause disruptions to building operations. An infill wall constructed of precast concrete panels is proposed for cases where owner or user constraints preclude use of cast-in-place infill walls.

The precast infill wall system eliminates the need for formwork and the need for connection dowels. Additionally, the problems associated with casting large quantities of concrete in an existing building are eliminated. Construction time and inconvenience to occupants can be reduced along with the costs associated with time of construction and interruption of operations.

Three phases of experimental research were conducted. The first phase was investigation of the precast panel to panel connection. In the second phase, the precast panel to existing frame connection was studied. In both phases, direct cyclic shear tests were conducted to determine satisfactory details for the connections. The third phase consisted of testing a large-scale model specimen. The test specimen, a two-story nonductile frame infilled with precast panels was used to evaluate the overall system behavior and verify performance of connection details. In the two-story specimen, the performance of post-tensioning used to provide column tensile capacity at the boundary elements of the wall was also studied.

DESCRIPTION OF RESEARCH

A precast panel system has been developed that will enable the panels to be assembled into an infill wall. The precast panels can be brought into an existing structure through the use of elevators and light forklifts. The panels have shear keys along the sides and are connected to one another through the use of a reinforced grout strip. Panels are connected to the existing frame through the use of steel pipes. The existing structure is cored in selected locations to allow for insertion of pipes and continuity of the wall vertical reinforcement. A schematic of the proposed wall system is shown in Fig. 1. In addition to the infill wall system, the tensile capacity of the existing frame columns must be increased to provide overturning capacity. Typical nonductile structures were constructed with only compression lap splices which do not provide for tensile yielding of column reinforcement. A post-tensioning system located adjacent to the existing frame columns (boundary elements) is used to improve column tensile capacity. The post-tensioning system is illustrated in Fig. 2.

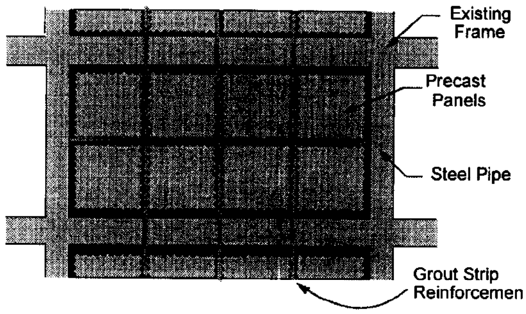

Fig.1 Precast panel infill wall system

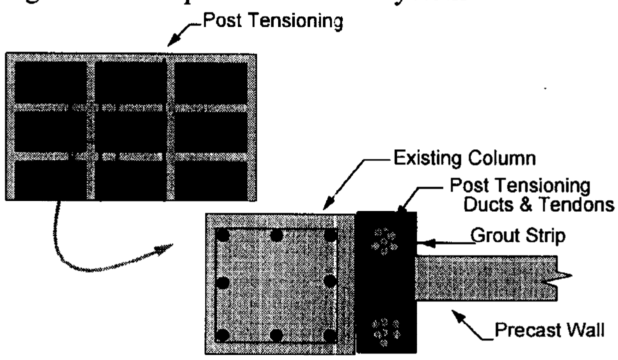

Fig. 2 Post-tensioning for column tensile capacity.

Connection Tests

Panel Connection. In order to understand how to connect the precast panels to one another and to the existing building frame, component tests were conducted. In the first phase, performance of the panel to panel shear transfer was evaluated. There was no direct connection of reinforcement in adjacent panels. As shown in Fig. 3, a representative connection detail was extracted and tested by applying cyclic shear across the keyed interface (grout strip). Primary variables are noted. Fourteen specimens were tested. A representative test result is shown in Fig. 4 for two 9 mm vertical bars in the grout strips in both directions. As noted in the figure, the specimen was very stiff initially and behaved monolithically up to the point where adhesion at the shear key/grout interface was lost. Following adhesion loss, loading was increased until a failure occurred through the shear plane. The failure occurred through the shear keys in either the panel or grout depending on the strength of the concrete. After the peak capacity was reached, there was a transition to a lower load plateau or residual capacity which was maintained through large slip levels. A summary of the tests conducted is presented in Table 1. The peak and residual capacities are presented for zero compression across the interface. Both peak load and residual capacity were influenced primarily by the amount of reinforcement in the grout strip.

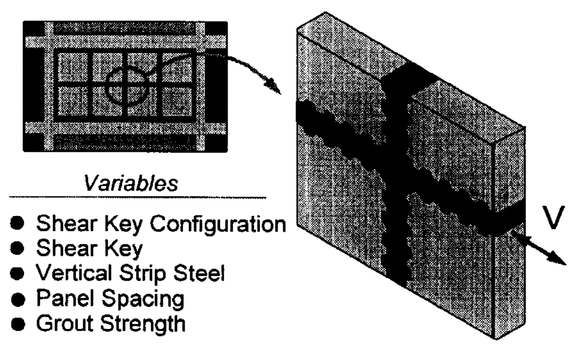

Fig. 3 Panel connection test specimen

Failure of the specimens consisted of sliding along the keyed interface with either grout or panel keys shearing depending upon the concrete strength in each. The ultimate capacity of the connection was controlled by the lower concrete shear strength (grout or panel). Additionally, the amount of vertical reinforcement influenced the ultimate capacity. Residual capacity was maintained by shear friction which was directly dependent on the amount of vertical reinforcement. Shear key configuration and size did not have a significant effect; the only influence was the ultimate capacity as related to the shear area (cross sectional area at the base of the keys). The spacing of panels had no effect on connection behavior in the tests conducted.

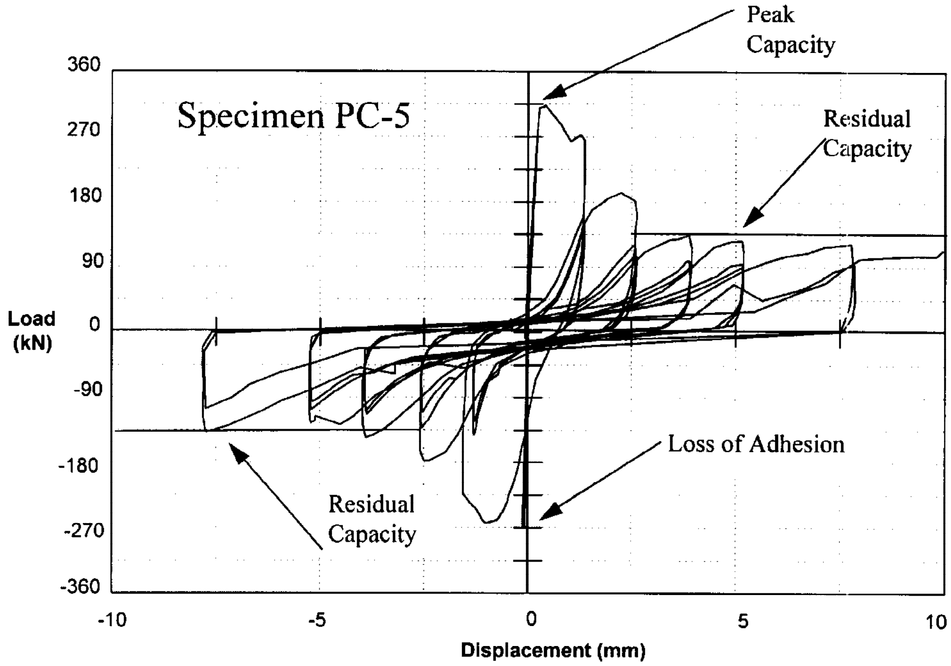

Fig. 4 Load-displacement response of a panel connection.

Table 1 Panel connection tests

Specimen	Panel Thickness (mm)	Key Alignment	Key Spacing / Key Size	Key Size	Grout Strength	Panel Separation (mm)	Reinforcing Steel No. and Size	Peak Load (kN)	Residual Capacity (kN)
PC-1	150	Aligned	1.0	Normal	High	50	2 - 9 mm	374	134
PC-2	150	Aligned	1.5	Normal	High	50	2 - 9 mm	276	134
PC-3	150	Staggered	1.0	Normal	High	50	2 - 9 mm	389	134
PC-4	150	Staggered	1.5	Normal	High	50	2 - 9 mm	312	134
PC-5	150	Aligned	1.0	Normal	High	100	2 - 9 mm	312	134
PC-6	150	Aligned	1.0	Normal	High	100	4 - 13 mm	605[1]	*
PC-7	150	Aligned	1.0	Normal	Low	100	4 - 13 mm	454	258
PC-8	150	Aligned	1.0	Large	High	100	4 - 13 mm	681	223
PC-9	150	Aligned	1.0	Normal	High	100	4 - 9 mm	458	178
PC-10	150	Aligned	1.0	Normal	High	100	2 - 13 mm	400[2]	231[2]
PC-11	150	Aligned	1.0	Large	High	100	4 - 9 mm	552[2]	181[2]
PC-12	150	Aligned	1.0	Normal	High	100	6 - 16 mm	605[1]	*
PC-13	100	Aligned	1.0	Normal	High	100	2 - 9 mm	280	89
PC-14	100	Aligned	1.0	Normal	High	100	4 - 13 mm	329	111

[1] Interface shear failure did not occur. Wall failed in bearing from compression of loading head. In addition, a vertical restraint system was used which caused compression across the interface. The peak loads tabulated are computed values that eliminate the effects of interface compression.

[2] A vertical restraint system was utilized that induced compression across the interface. The peak and residual loads tabulated are computed values that eliminate the effects of interface compression.

Frame Connection. In the second phase, performance of the connection between the precast panels and the existing frame was evaluated. A representative detail of the connection to the beam and the column is shown in Fig. 5. The connection was subjected to cyclic shear along the frame boundary interface. Primary variables studied are noted. A representative test result is shown for a 63 mm extra strong pipe embedded 230 mm into the frame element and 230 mm into the wall side (grout strip) of the connection. The structure initially responded in a monolithic fashion until loss of adhesion occurred along the interface. Loading was continued until a peak capacity was reached. Following the peak, there was a transition to a lower load plateau which was maintained to extremely large slip levels. A summary of the tests conducted is listed in Table 2. The tests showed that embedment into the wall was more critical than embedment into the existing frame. Specimen FC-1 had an embedment of only 100 mm and did not cross the shear key interface. The failure of this specimen occurred above the steel pipe along the shear key interface reinforced with two 10 mm bars. Therefore, the failure observed corresponded with that of specimen PC-5 which also developed a shear key interface failure reinforced with two 10 mm bars.

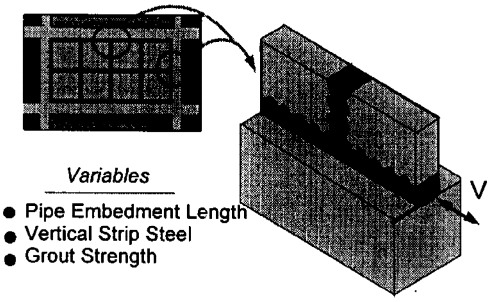

Fig. 5 Frame Connection Test Specimen

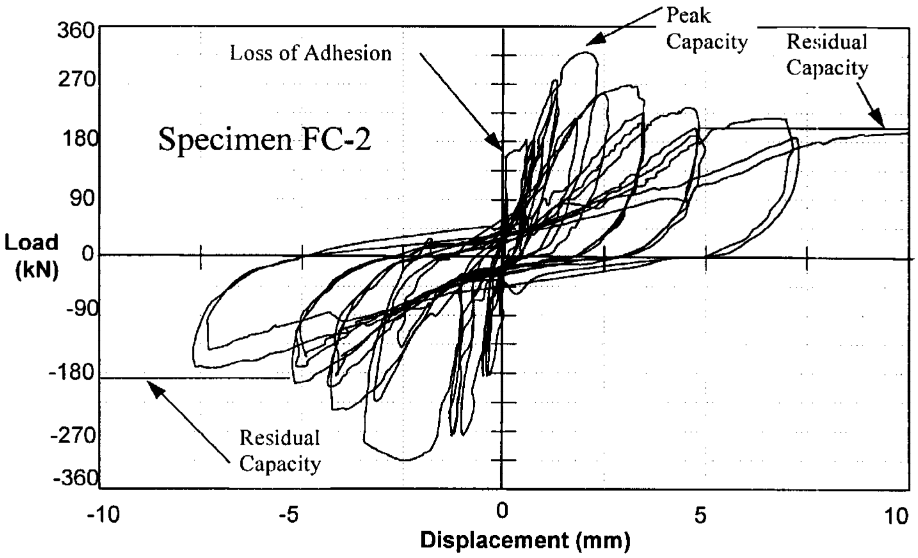

Fig. 6 Load-displacement response of a frame connection

Table 2 Frame Connection Tests

Specimen	Panel Thickness (mm)	Pipe Size	Frame Embedment Length (mm)	Wall Embedment Length (mm)	Peak Load (kips)	Residual Capacity (kips)
FC-1	150	64 mm XS	230 mm	100 mm	276[1]	151[1]
FC-2	150	64 mm XS	230 mm	230 mm	316	191
FC-3	150	64 mm XS	100 mm	230 mm	325	205
FC-4	100	51 mm XS	100 mm	230 mm	225	134

[1] Failure occurred above the steel pipe along the shear key interface.

Failure of the specimens consisted of interface sliding along with yielding of the steel pipe. The residual capacity was determined by the yield strength of the pipe. Yielding of the pipe was ensured by providing adequate concrete bearing as calculated using the projected area in the direction of force application. Tests showed that the pipe must have adequate embedment on both sides of the interface. When embedment was lacking on the wall side, a shear key interface failure (as experienced in the panel connection tests) occurred.

Model Test Structure

The third phase was the testing of a large scale model specimen. The test specimen, an interior, two-story nonductile frame infilled with the proposed precast infill system, was used to establish the overall system behavior and verify the performance of the connection details. The performance of the post-tensioning system used to provide column tensile capacity was also evaluated. The model structure shown was initially constructed with details typical of 1950's and 60's construction. The model structure has a column spacing of 4.06 m and floor heights of 2.44 m which correspond to 2/3 scale for typical prototype dimensions. The structure was infilled with the precast infill wall system developed in the connection tests. A 150 mm wall thickness was used for the first floor and a 100 mm wall was used for the second. The structure was subsequently tested under cyclic loads with a triangular loading applied at the floor levels. Both flexural and shear strength of the structure were investigated. In addition, the effect of the post-tensioning was assessed.

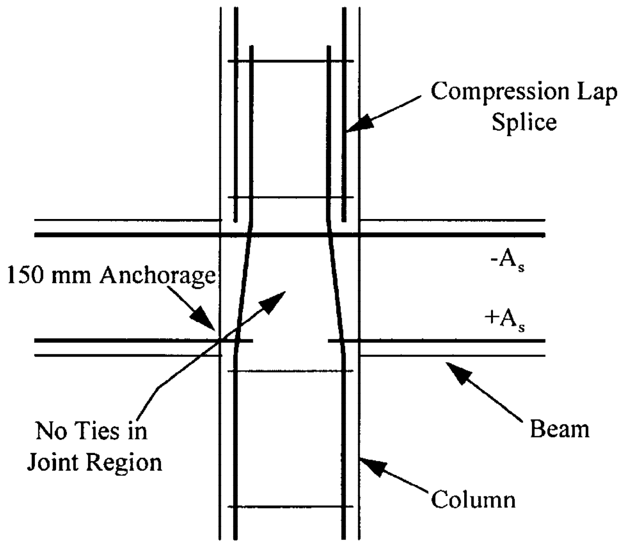

Fig. 7 Typical Beam-Column Joint Detail

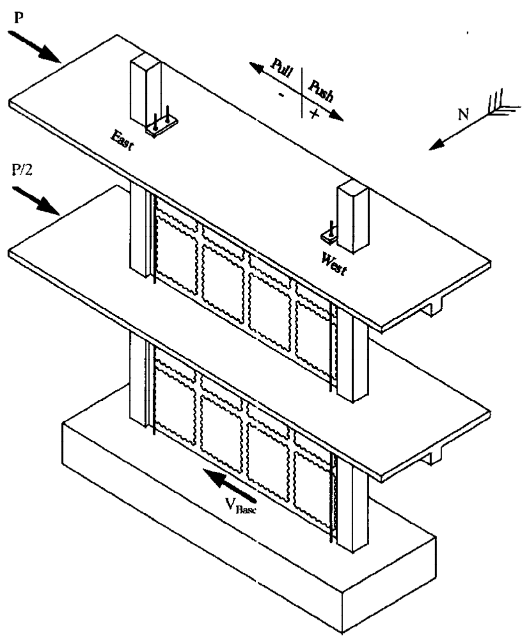

Fig. 8 Model structure.

Flexural Test

The flexural test was conducted with two 25 mm post-tensioning bars on one end of the wall and two 32 mm post-tensioning bars on the other end. The test results shown in Fig. 9 indicate the bars that were in tension. Different bar sizes were used on either side to allow different behavior in each direction in order to maximize the information obtained from the test specimen. The bars were stressed to 254 kN per bar for a total of 1014 kN. As indicated in the test result, the column lap splice failed at a base shear of 1173 kN. The specimen was cycled at this load. Following these load cycles, testing continued with loading in only the positive direction (two 25 mm bars in tension) to avoid failing both column splices in the first test. Loading was continued to specified displacement levels. It can be seen that the load deflection curves are nonlinear-elastic. Upon unloading, the structure was able to return approximately to its original position. The post-tensioning steel remained elastic which accounted for this behavior. The test continued until first yield of the post-tension steel which occurred at approximately 0.55% drift. Large drift levels for a shear wall were achieved while the strength was maintained. Residual deflections resulted when the post-tensioning steel yielded.

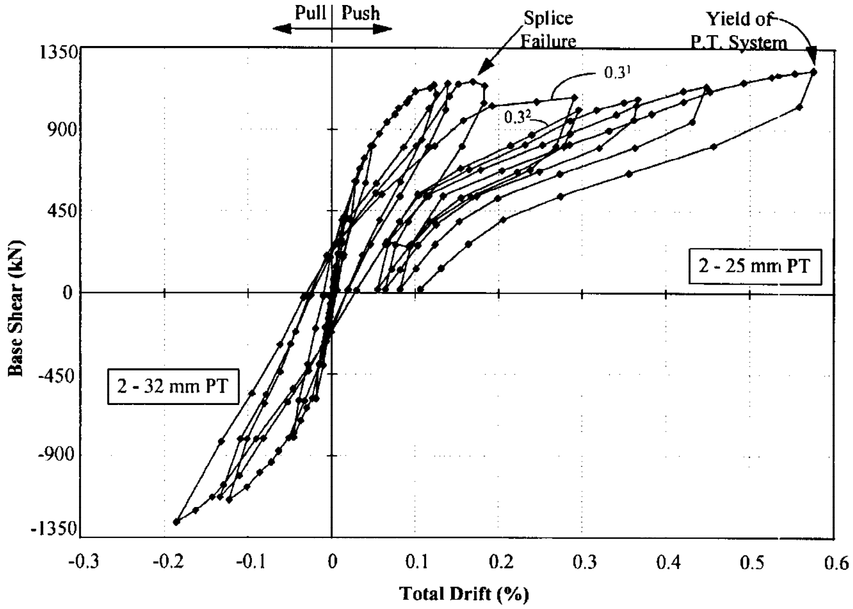

Fig. 9 Flexural test of model structure

Shear Test

A shear test was conducted with two 25 mm post-tensioning bars added at each end and stressed to 290 kN per bar for a total of 2310 kN. Additional steel was required to increase the flexural strength to permit the wall to reach higher lateral capacity. As indicated in Fig. 10, the remaining column lap splice failed at a base shear of 1870 kN. The increase of the post-tensioning force increased the load at which the splice failure occurred. The structure was subjected to a base shear of 2000 kN without reaching a shear failure.

A subsequent test was conducted with the same post-tensioning bars, but no post-tensioning load was applied. The nuts on the bars snug tight. This test was used to determine a lower bound shear capacity for the wall since the precompression should increase the shear strength. In this test, the structure reached a base shear of 2200 kN when crushing of the wall near the top post-tensioning anchorage was observed. No interface grout

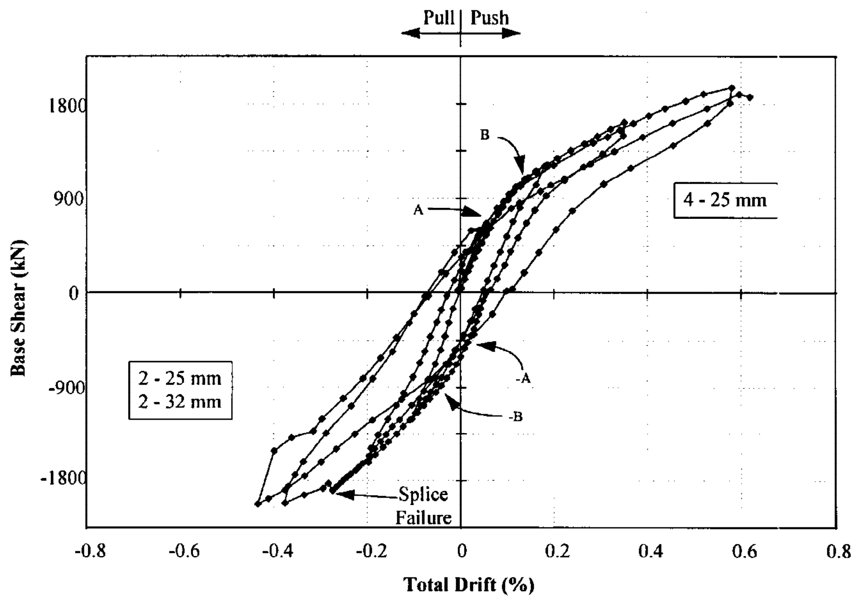

Fig. 10 Shear test of model structure

strips or wall/frame interface failures were evident following inspection of the entire wall at the end of the test.

The two-story nonductile frame with the precast infill system and with post-tensioning to provide column tensile capacity performed exceptionally well. As shown in Photo 1, diagonal cracks were continuous through panels and grout strips. There was no indication of distress along grout strips. There was some cracking concentrated around the pipe shear lugs between the wall and frame, but failure did not appear to be imminent. Slip between the wall/frame boundaries were minimal. Overall, the wall behaved monolithically with no indication of separation into smaller units. The post-tensioning performed as anticipated from design calculations. A large crack formed at the base of the wall with the wall/frame system rotating nearly as a rigid body around the "toe" of the wall in the direction of applied load.

Design recommendations were developed for the precast infill wall system and are included in Reference 2. Design Guidelines are based on developing a precast infill wall system to reach capacity by developing a flexural hinge at the base or by yielding the shear lugs along the horizontal shear plane at the base of the wall. Both failure modes insure that ductile response is achieved.

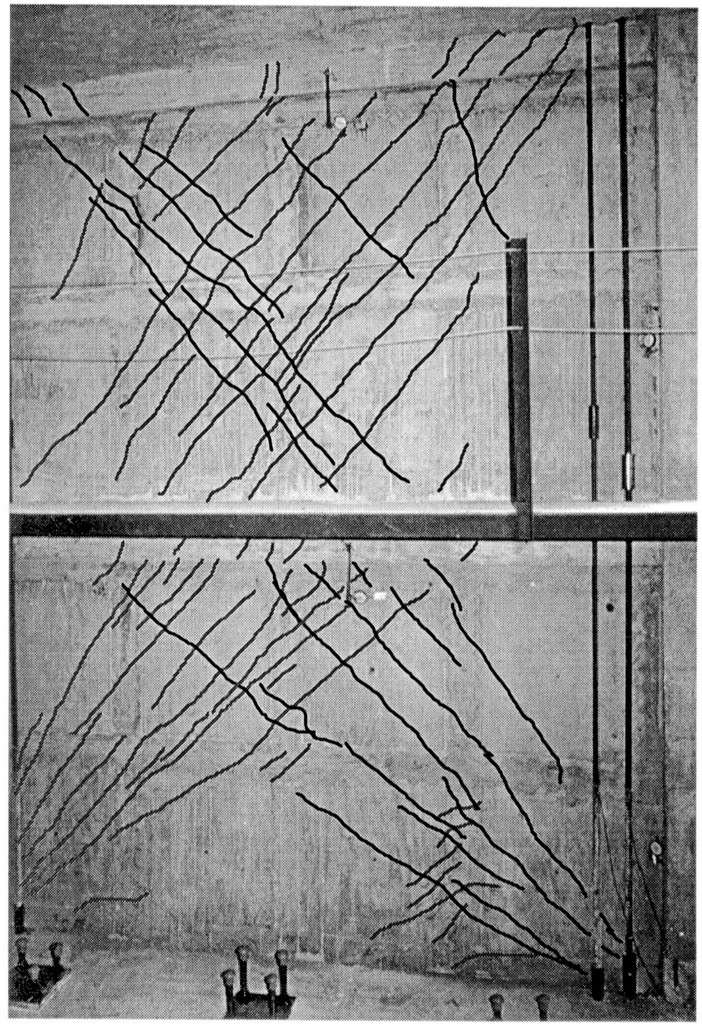

Photo 1 Wall cracking pattern

CONCLUSIONS

The infill wall system developed performed well and transformed the structure from a non-ductile lateral force-resisting frame to a ductile shear wall system with vastly improved strength and stiffness characteristics. Precast wall units may eliminate problems associated with cast-in-place infill wall construction such as, interference with occupants or operations, time of construction, size of construction equipment and manpower needed. Construction of model structure demonstrated that the precast system can be constructed rapidly and with excellent quality control. Overall, the precast infill system developed should provide engineers with a new technique and new approaches for rehabilitating existing structures with shear walls.

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