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SEISMIC PERFORMANCE OF STEEL PLATE SHEAR WALLS BASED ON A LARGE-SCALE MULTI-STOREY TEST

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ABSTRACT

Steel plate shear walls are a lateral load resisting system for buildings that consists of vertical steel plates one storey high and one bay wide welded all around to the surrounding boundary of beams and columns. These plates can be installed in one or more bays for the full height of a building to form a stiff steel wall. Steel plate shear walls are well-suited for either new construction or as a relatively simple method for seismic upgrading. They possess many properties that are fundamentally beneficial to resisting seismically-induced loads, including redundancy, ductility, and robust resistance to degradation under cyclic loading.

A four-storey, single bay specimen, fabricated using industry-standard details and methods, was tested under controlled cyclic loading to determine its behaviour under an idealized severe earthquake event. During the test, the specimen endured 30 cycles of loading, including 20 cycles in the inelastic range. The lowest, and most severely loaded, panel exhibited great ductility: it reached a deformation in the final cycle of nine times the yield deformation. The maximum load capacity was reached in Cycle 22, after which deterioration was very gradual and stable. The shear panels, combined with beam-to-column continuity in the boundary members, resulted in a redundant system that displayed significant energy absorption, ductility, and resistance to degradation. This paper describes the behaviour of the specimen and the manner in which damage to the shear wall eventually developed.

KEYWORDS

Shear wall; steel; plate; panel; tension field; multi-storey; buildings; experimental; test.

INTRODUCTION

This paper presents an overview of the results of a cyclic quasi-seismic test conducted on a four-storey steel plate shear wall by researchers at the University of Alberta during the spring of 1995. This test is a major part of the Natural Sciences and Engineering Research Council of Canada Collaborative Research Project being carried out at the University of Alberta and the University of British Columbia. The purpose of the investigation is to examine the behaviour of steel plate shear walls when subjected to earthquake loading.

SPECIMEN AND TEST SET-UP

The set-up for the shear wall test is depicted in Figs. 1 and 2, and the test specimen alone is shown in Fig. 3. The test specimen consists of a four storey moment-resisting steel frame infilled with thin steel panels at each storey. The panel thicknesses shown in Fig. 1 are nominal values: mean measured thicknesses for infill Panels 1 (lowest) to 4 (uppermost) are 4.54 mm, 4.65 mm, 3.35 mm and 3.40 mm, respectively. The steel

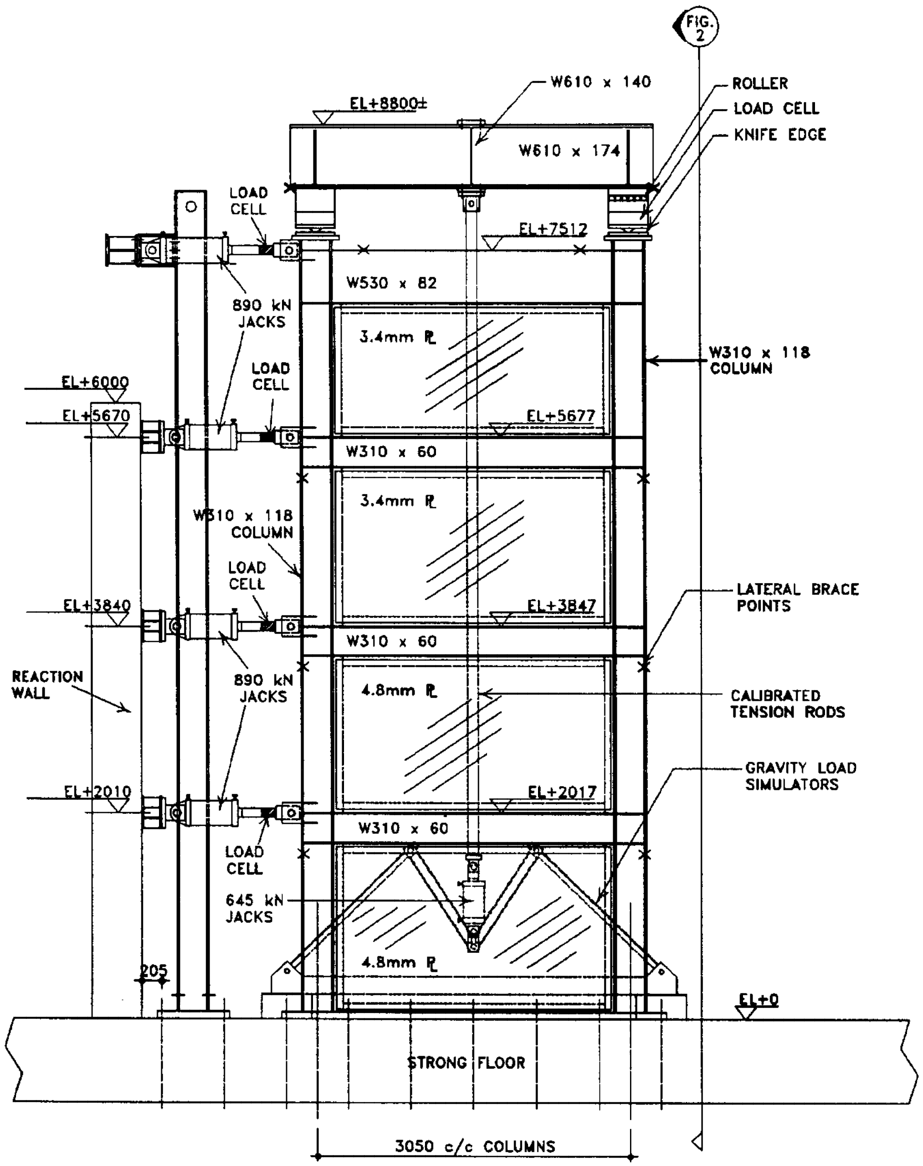

Fig. 1. Shear Wall Test Set-up
Side Elevation

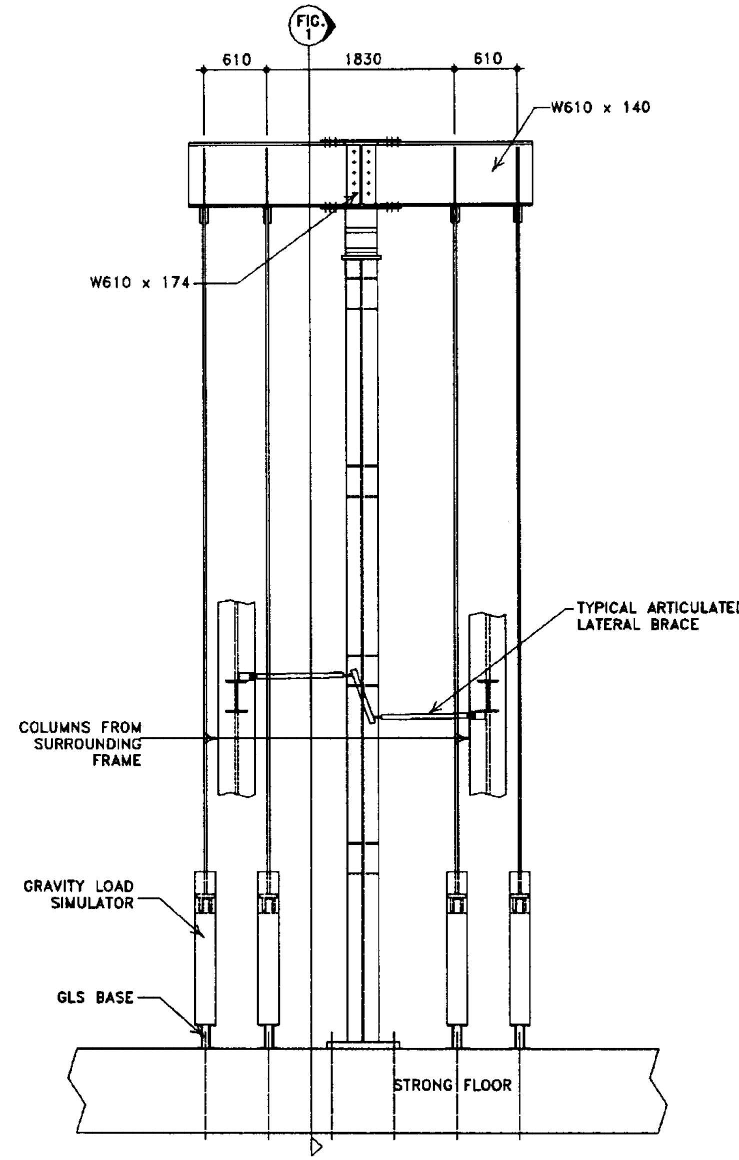

Fig. 2. Shear Wall Test Set-up
End Elevation

Fig. 3 Shear Wall Test Specimen

member sizes from which the moment-resisting frame was constructed are specified in Fig. 1. Connection of the beam flanges to the columns was made using complete penetration groove welds, including a backing bar and run-off tabs that were left in place. The beam webs were connected to the column flange with fillet welds on both sides and column stiffeners were installed adjacent to each beam flange. The columns were connected to the base plate using full penetration groove welds at the flanges and fillet welds at the webs. Each infill plate was connected to the boundary members (the columns and the beams) by means of a "fish plate." The fish plates are continuous 100 mm wide by 6 mm thick connection tabs fillet welded to the flanges of the boundary members. Where column fish plates and beam fish plates meet at the panel corners, a small strap plate is used to provide continuity. The infill plates are, in turn, fillet welded against one side of the fish plates, overlapping by 40 mm. This detail allows a simple means of compensating for normal fabrication tolerances, thereby avoiding fit-up problems in the field. More details about the test specimen can be found in Driver *et al.* (1995).

Horizontal loads were applied to the test specimen using double-acting jacks at each of the four floor elevations. Because the jacks were supplied from a common manifold, their loads were essentially equal. The vertical column loads were applied through a distributing beam at the top of the shear wall and four calibrated tension rods (see Fig. 1) connected to hydraulic jacks at the base. These jacks were connected to gravity load simulators at the base of the structure .

Loads were measured at all eight jacks and at the column tops using load cells. Both in-plane and out-of-plane displacements were measured at each of the four levels of the shear wall. The small vertical and horizontal movements of the base plate were also monitored during the test. A total of 98 strain gauges were used to acquire strain data during the test. Thirty were in the form of ten strain rosettes affixed to Panel 2 (five on each side). The remainder of the strain gauges were located on the beam and column flanges, clustered around five of the beam-to-column junctions.

LOAD AND DEFLECTION HISTORY

The load and deflection history selected for the test was based on the method outlined by the Applied Technology Council (1992). This document, designated as ATC–24, is for experiments that use slow cyclic loading, and it requires that a "deformation control parameter" be selected for controlling the test. It recommends using a feature related to interstorey drift. In the case of the shear wall, the drift of the lowest storey (Panel 1) was selected, because this is where the majority of the deformation and energy absorption takes place. The force quantity best related to the deformation control parameter is the storey shear in Panel 1 (*i.e.*, the base shear). The method for arriving at a loading strategy is described in ATC–24, whereby a deformation, δ_y, and a load, Q_y, are determined to coincide with the point where "significant" yielding has occurred in the specimen. Up to this point, load control is used in the test; subsequently, displacement control is used.

The yield displacement (δ_y) in Panel 1 during the shear wall test was determined to be 8.5 mm. Prior to this, single loading cycles of ±200 kN, ±400 kN, ±600 kN, ±800 kN, and three cycles each of ±1000 kN and ±1950 kN were conducted to explore the elastic and nominally inelastic behaviour. These constituted Cycles 1 to 10. After three cycles with a displacement of δ_y = 8.5 mm, the displacement in the first storey was increased by 8.5 mm in each subsequent deformation step. Three cycles were conducted at each deformation step up to a deformation of 3 δ_y and two cycles at each deformation step thereafter (following the guidelines of ATC–24).

At a displacement of 5.4 δ_y (46 mm), the limit of jack stroke at level 3 was reached in one direction of loading. In all subsequent cycles, this peak deformation was maintained while the peak deformation in the opposite direction was increased as prescribed in ATC–24. The storey shear vs. storey deflection history for Panel 1 is shown in Fig. 4.

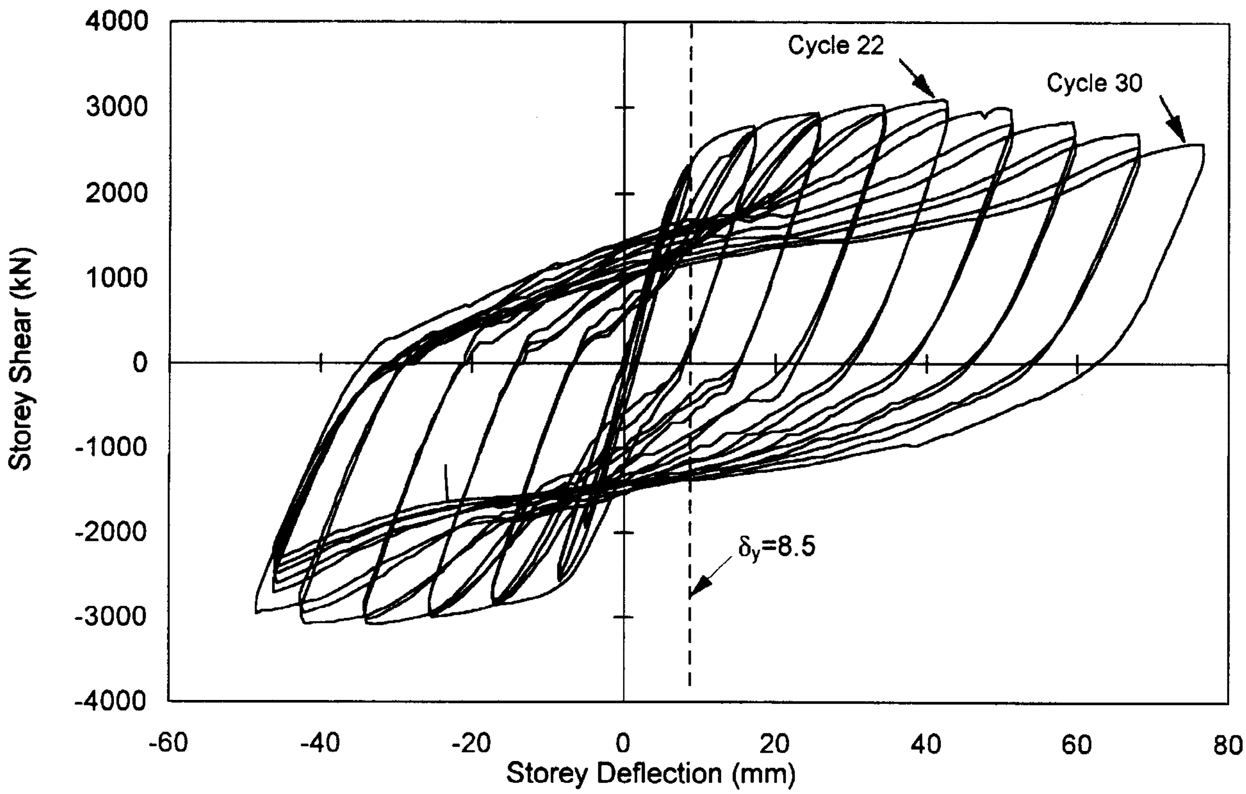

Fig. 4 Storey Shear vs. Storey Deflection - Panel 1

For the first five cycles (up to and including the first cycle at ± 1000 kN base shear), a gravity load of 75% of the target value was applied. During the remainder of the test, the target gravity load of 720 kN was applied at the top of each column. At times there was some difficulty in maintaining the gravity loads at a constant level as the shear wall was pushed horizontally back and forth. Generally, the gravity load variations were maintained within ± 5%, however.

SPECIMEN BEHAVIOUR

As already noted, there was very little yielding during the first 10 cycles. Yield lines were detected during the fourth cycle in the web of the beam at level 1 (located at the top of Panel 1), by the sixth cycle in the web of the beam at level 2, and by the eighth cycle in the web of the beam at level 3. In all cases, yielding was localized near the loading clevis at that level and appeared to be caused by the effect of the concentrated load.

During Cycle 8, yielding was apparent in Panel 1 and, to a lesser extent, in Panel 2. Most of the yielding was in the fish plates that connect the infill plates to the boundary members or at the periphery of the infill plates. Characteristic diagonal tension yield patterns began to form at the top corners of Panel 1.

In Cycle 11 (the first cycle at which $\delta = \delta_y$), the existing yield patterns became considerably more pronounced. Increased yielding was noted in the webs of the beams at levels 1, 2, and 3, as well as in the fish plates and infill plates of Panels 1 and 2. Panels 1, 2, and 3 all buckled visibly at the maximum displacement. In addition, loud bangs first occurred in this cycle as the plate buckles reoriented themselves upon reversal of the loading direction. These noises continued to occur in all subsequent cycles.

During Cycle 14 ($\delta = 2\,\delta_y$), yield lines developed that virtually covered both sides of Panel 1. Additional yielding occurred in Panels 2 and 3, including fairly heavy yielding across the bottom of Panel 2. The first yielding in Panel 4 was noted, along the top and bottom fish plates. Yield lines developed along the web of the beam at level 1 over its full length. The number of yield lines in this area continued to increase as the test progressed.

Although the web of the beam at level 1 and the webs of the columns above and below had already been extensively yielded for many cycles, the first yielding in the level 1 beam-to-column joint panels did not occur until Cycle 17 ($\delta = 3\,\delta_y$). Slight flaking of the whitewash on the joint panel indicated that the extent of yielding was moderate.

The first tear was detected during Cycle 18 in the top, west corner of the south face of Panel 1. It was 6 mm long and located at the corner of the weld connecting the infill plate to the fish plate, transverse to the weld axis. This crack did not propagate during subsequent cycles and is considered to have had a negligible effect on the behaviour of the test specimen.

During Cycle 20 ($\delta = 4\,\delta_y$), local buckles in the west flange of the east column and the east flange of the west column were discovered immediately below the beam at level 1. These buckles were of relatively small amplitude, but they grew in size during subsequent cycles. After the lateral load had been removed at the end of this cycle, residual amplitudes of 10 mm (west column) and 40 mm (east column) were measured. In addition, a local buckle of 13 mm amplitude was discovered in the east flange of the east column near the base.

In Cycle 22, plate tears were observed at the top corners of Panel 1 at the toe of the fillet weld connecting the fish plate to the columns. The east tear was 120 mm long and the west tear was 80 mm long. In addition, a 50 mm tear formed at the toe of the fillet weld connecting the infill plate to the fish plate at the top, west corner. Figure 5 shows the tears at the top west corner of Panel 1 after they had propagated during subsequent cycles. Also during Cycle 22, at a displacement of $5\,\delta_y$ in Panel 1, the maximum base shear of 3080 kN was reached. The load-carrying capacity of the test specimen declined very gradually during each of the remaining cycles of increasing deformations.

Beginning at Cycle 25, tears in the interior of the Panel 1 infill plate formed as a result of kinking of the stretched plate during load reversals. The plate tended to kink and straighten cyclically as the buckles reoriented themselves.

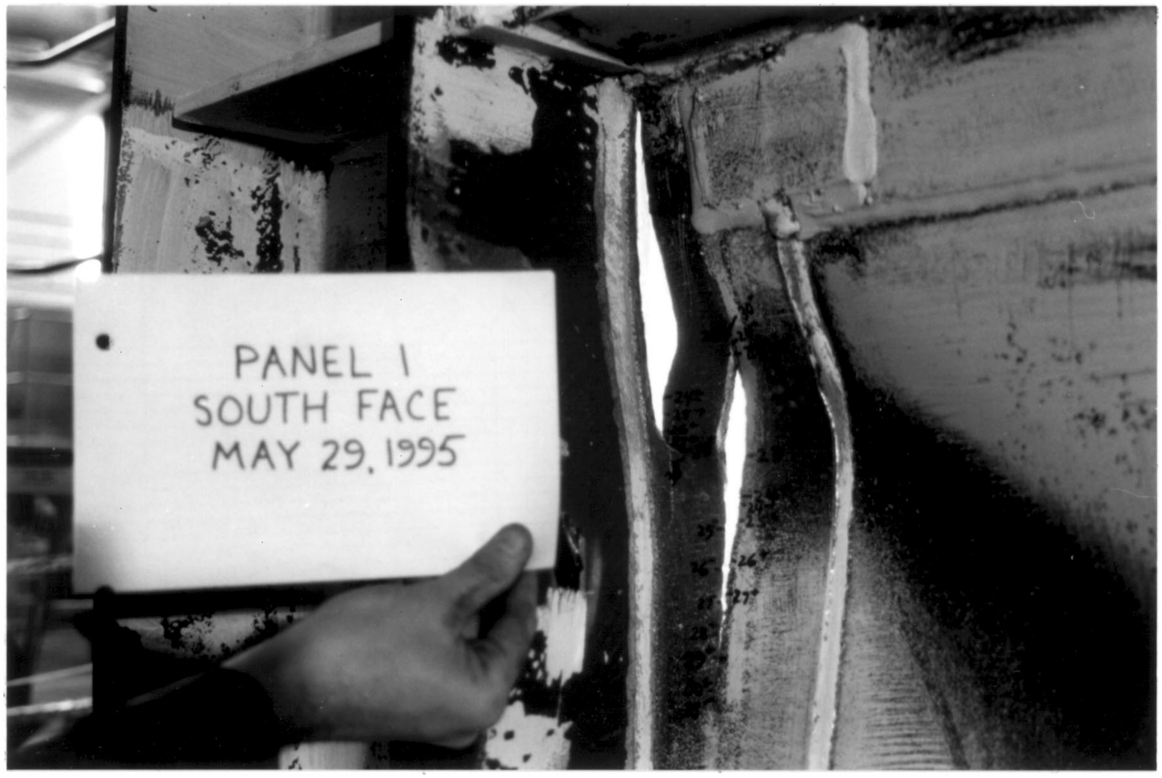

Fig. 5 Panel 1 Corner Tears

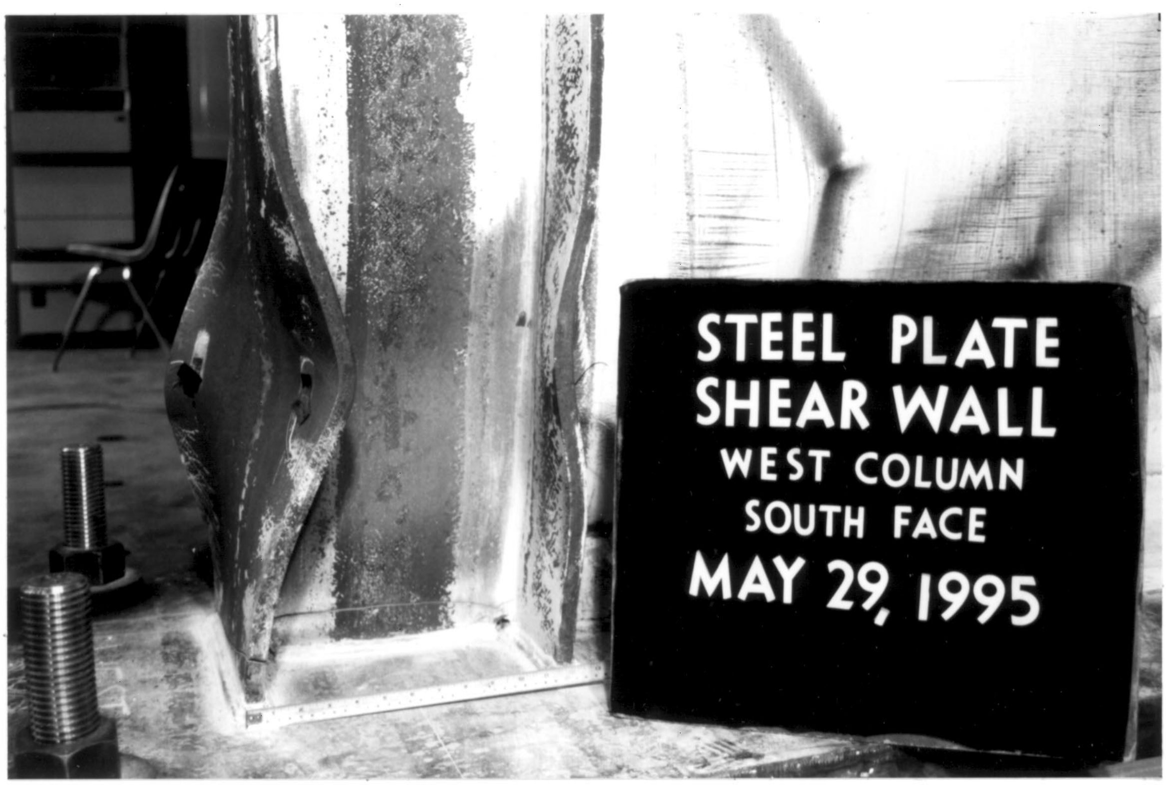

Fig. 6 West Column Local Buckles

By Cycle 26 ($\delta = 7\,\delta_y$), the column flange distortion in the first storey was extreme. Figure 6 shows the locally buckled flanges at the base of the west column at the end of the test.

In the first excursion of Cycle 30, a displacement of $9\,\delta_y$ was achieved in Panel 1. After the shear wall was unloaded and was in the process of being reloaded in the opposite direction, the west column fractured at its base (at a base shear of approximately 1750 kN). The fracture was sudden and was accompanied by a large release of energy. A metallurgical examination revealed that the fracture began in the heat-affected zone of the base material at the toe of the weld connecting the west flange of the column to the base plate. After growing a distance of about 20 mm prior to the final cycle, the crack extended another 10 mm in the final tensile cycle. Both of these regions showed characteristics indicating that the material behaved in a relatively tough manner. The crack then initiated a brittle cleavage fracture that propagated rapidly through the remainder of the west flange and completely through the web. Figure 7 shows the fractured west column.

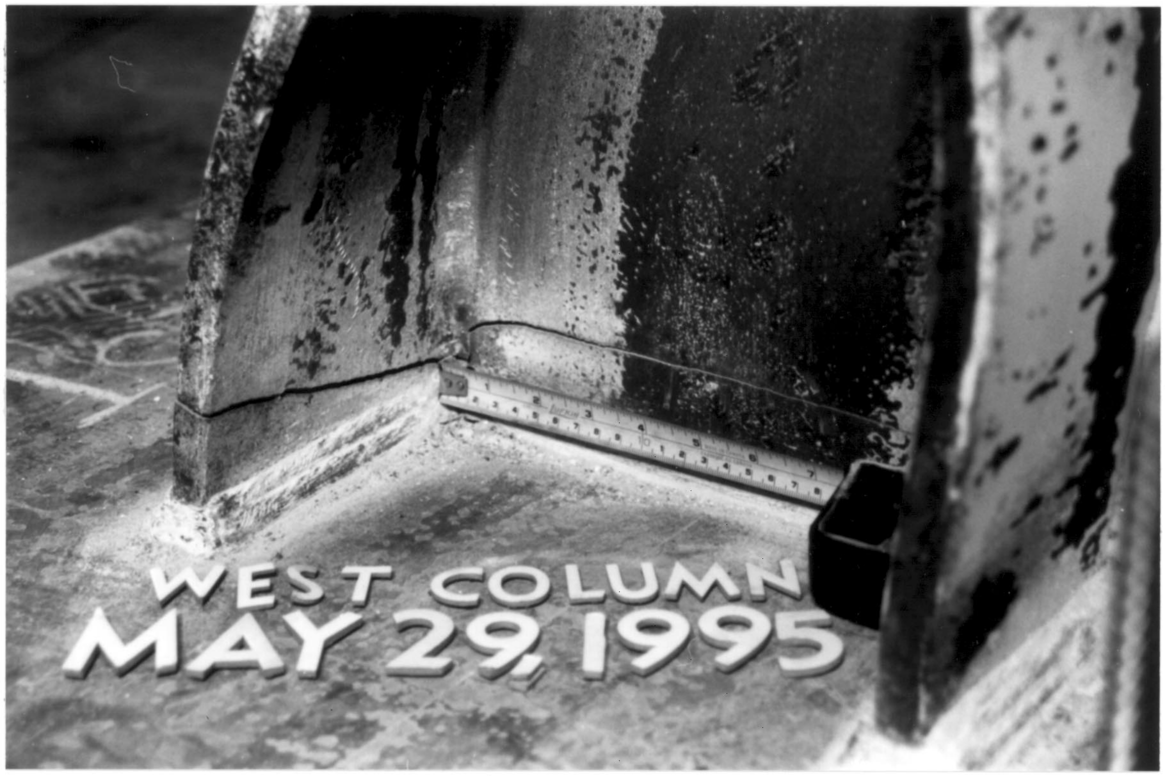

Fig. 7 Fractured West Column

During the first loading excursion of Cycle 30 and just prior to failure, the base shear reached was 85% of the maximum base shear achieved (Cycle 22). The stiffness of the shear panel itself declined in a gradual and stable manner, and it still maintained its integrity at the end of the test.

DISCUSSION

The criterion that was used to control the test, and which is of prime importance for defining the performance of the test specimen, is the storey shear vs. storey deflection behaviour of Panel 1, shown in Fig. 4. This graph clearly demonstrates the significant ductility exhibited by the shear wall during the test. The uniformity of the hysteresis loops implies stability under extreme cyclic loading and, even after the peak load had been reached, deterioration was very slow and controlled. Further, the eventual failure was by fracture at the column base. Had this failure mode been avoided, there is reason to believe that the trend of very gradual deterioration of capacity would have continued. Since the load-carrying capacity of the test specimen had already decreased to 85% of the maximum, no repair of the column was attempted, however.

The large compressive strains that were present at the column bases caused severe local buckling in the flanges adjacent to the column base connections. Reversing loads tended to repeatedly buckle and straighten these areas. This type of loading leaves the structure vulnerable to failure by propagation of a crack initiating at a flaw in the weld or the parent material. Local buckling can be prevented, however, by the judicious positioning of stiffeners, reducing the susceptibility of the joint to this type of failure.

Figure 4 also shows that the hysteresis loops generated are relatively wide, meaning that there was significant energy absorption during each cycle. The curves flatten in the region where the plate buckles reorient themselves during a load reversal prior to the full development of the tension field. However, the significant stiffness of the moment-resisting boundary frame appears to prevent severe pinching of the hysteresis loops seen in shear walls with frames having simple connections (Kulak, 1991).

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