

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

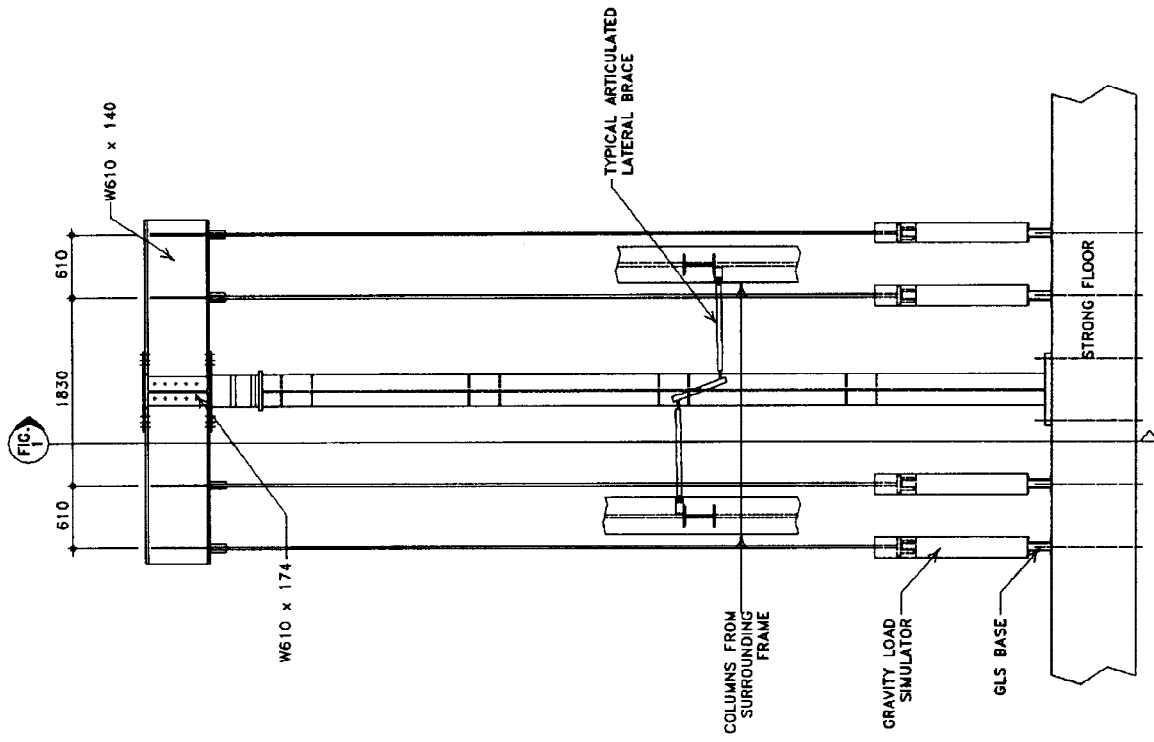


Fig. 1. Shear Wall Test Set-up Side Elevation

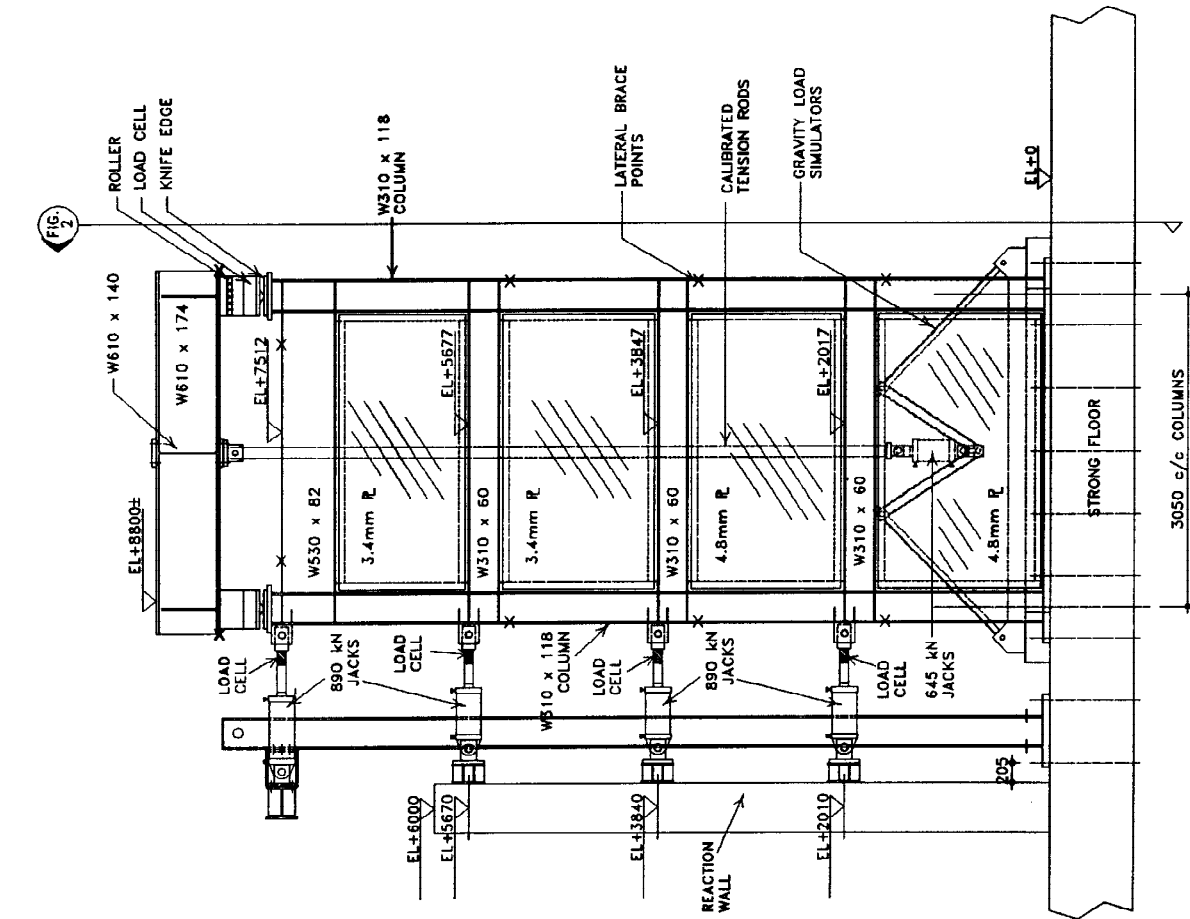


Fig. 2. Shear Wall Test Set-up End Elevation

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 $\delta_y = 8.5$ 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 $3\delta_y$ and two cycles at each deformation step thereafter (following the guidelines of ATC-24).

At a displacement of $5.4\delta_y$ (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.

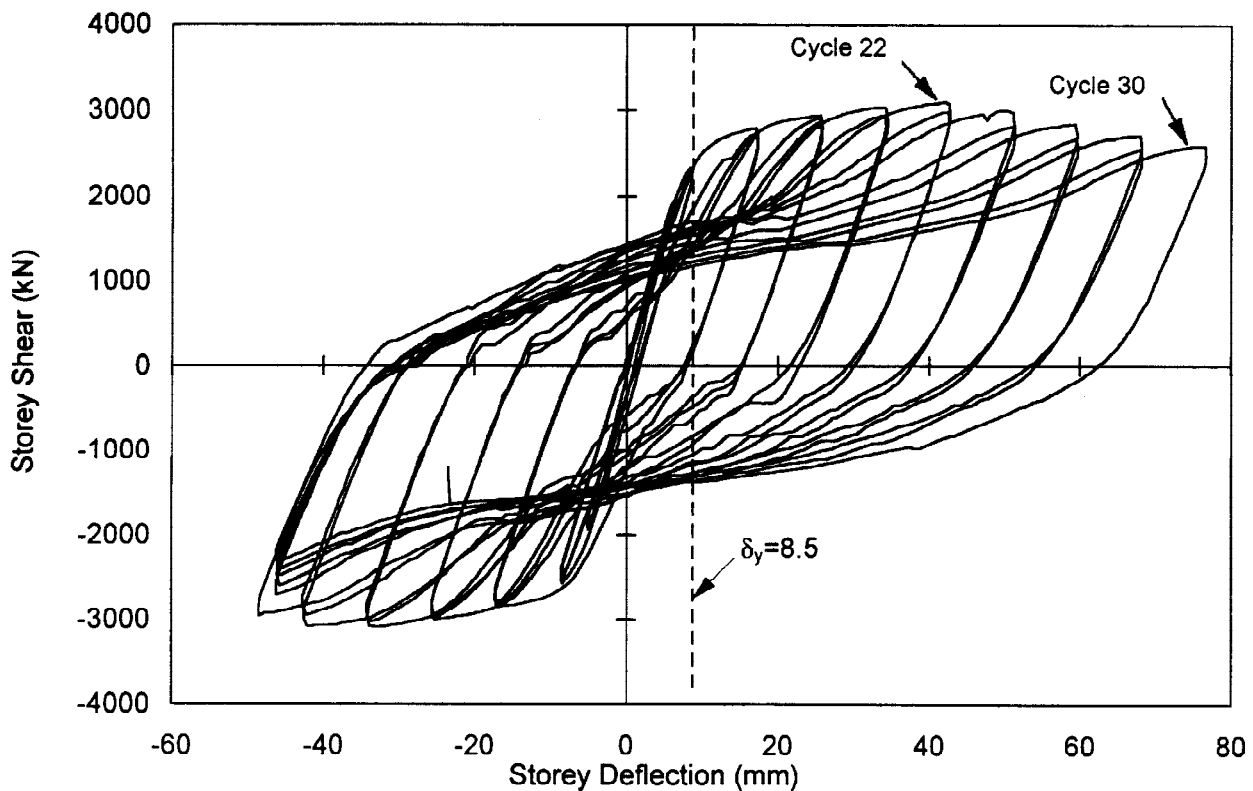


Fig. 4 Storey Shear vs. Storey Deflection - Panel 1

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 $\pm 5\%$, 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.

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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