Cyclic Test of Four-Story Steel Plate Shear Wall

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Abstract: A large-scale, four-story, single bay steel plate shear wall specimen with unstiffened panels was tested using controlled cyclic loading to determine its behavior under an idealized severe earthquake event. The shear wall had moment-resisting beam-to-column connections, resulting in a lateral load-resisting system that possesses an inherent redundancy. Gravity loads were applied at the top of the wall and equal horizontal loads were applied at the four floor levels. The specimen endured 30 cycles of loading during the test, of which 20 cycles were in the inelastic range. Prior to failure of the specimen, the deflection reached in the lowest story was nine times the yield deflection. The test specimen proved to be initially very stiff, showed excellent ductility and energy dissipation characteristics, and exhibited stable behavior at very large deformations and after many cycles of loading. A description of the test setup, loading procedures, and specimen behavior is presented.

INTRODUCTION

Steel plate shear walls are an innovative lateral load-resisting system capable of effectively bracing a building against both wind and earthquake forces. The system consists of steel plates one story high and one bay wide connected to the adjacent beams and columns. The plates are installed in one or more bays for the full height of a building, thereby forming a stiff cantilever wall. The surrounding steel frame may use either simple or moment-resisting beam-to-column connections, and the panels themselves can be either stiffened or unstiffened, depending on the design philosophy. Steel plate shear walls are well-suited for either new construction or as a means for the seismic upgrading of existing structures, whether of steel or concrete framed construction. It is anticipated that the system will be economical as compared with concrete shear walls, for example, because foundation costs are reduced, rentable floor area is increased, and the construction process requires the use of only one trade on the site. When unstiffened plates are used, fabrication of a steel plate plate shear wall core is relatively simple.

Steel plate shear walls possess properties that are fundamentally beneficial in resisting seismically induced loads. As is demonstrated herein, these include superior ductility, robust resistance to degradation under cyclic loading, high initial stiffness, and, when moment-resisting beam-to-column connections are present, inherent redundancy and significant energy dissipation. Moreover, the low self-weight of a steel plate shear wall reduces both the gravity loads and the seismic loads transmitted to the foundation. This may lead to construction cost savings.

Some existing steel plate shear wall buildings were designed with shear panels that are stiffened in order to preclude out-of-plane buckling. Although it has been shown that stiffening the panel heavily can produce a significant increase in the amount of energy dissipated under cyclic loading (Takahashi et al. 1973), the cost involved is likely to be prohibitive in most markets. However, it has been known for a long time that buckling does not necessarily represent the limit of useful behavior and there is considerable postbuckling strength in an unstiffened shear panel (Wagner 1931). At the point of buckling, the load-resisting mechanism changes from in-plane shear to an inclined tension field. When the panel is thin, buckling occurs at very low loads and the resistance of the panel is dominated by tension field action. The consideration of the postbuckling strength of plates has been accepted in the design of plate girder webs for many years (*Specification* 1993; LRFD 1994).

The Canadian steel design standard, *CAN/CSA-S16.1-94* ("Limit" 1994), includes an appendix that outlines design requirements for unstiffened thin-panel steel plate shear walls. The methodology is based primarily upon an analytical model developed by Thorburn et al. (1983) that has been substantiated by tests (Timler and Kulak 1983; Tromposch and Kulak 1987). The physical tests conducted were both single-story shear wall panels employing either true pins or standard bolted shear-type connections at the beam-to-column joints. Other tests have been carried out by Caccese et al. (1993) and Elgaaly et al. (1993). Analytical studies by other researchers have also been reported (Xue and Lu 1994).

Because no large-scale multistory test had been conducted on shear walls with thin, unstiffened panels, extrapolation to multistory applications has had to be based on computer analysis, model tests, and engineering judgment. This paper describes the cyclic testing of a large-scale, four-story steel plate shear wall. The specimen is a four-story structure, fixed at the bottom and loaded laterally at the four floor levels. In order to maximize the ability of the shear wall to dissipate energy under seismic loading, moment-resisting beam-to-column connections were used. This test has provided important additional evidence supporting the suitability of unstiffened thin-panel steel shear walls for seismic applications.

OBJECTIVES AND SCOPE

The objective of this research was to study the behavior of steel plate shear walls when subjected to extreme cyclic loading, such as would be expected in a severe earthquake. Because of the expense involved in large-scale testing, only one test specimen could be constructed. It was intended to simulate as closely as practicable a steel shear wall as it would be constructed in practice, and therefore no special or unusual fabrication techniques were employed. The test was conducted according to an established method for applying simulated earthquake loading. Gravity loads were applied to the columns throughout the test.

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The two general objectives in conducting the test were to assess the performance of the details selected for the test specimen and to evaluate the overall performance of the shear wall. The latter included the ability of the shear wall to dissipate energy during inelastic cyclic loading and the contribution of the moment-resisting frame to the total performance. Another major objective of this research, to develop analytical tools for predicting the behavior of steel plate shear walls, is addressed in the companion paper (Driver et al. 1998), and full details are reported in the original work (Driver et al. 1997).

PREVIOUS TESTING OF STEEL PLATE SHEAR WALLS

The survey of the literature is limited to the topic of physical testing of unstiffened steel plate shear walls. A more complete review of the literature on the general behavior of steel plate shear walls, including the methods of analysis, can be found elsewhere (Driver et al. 1997).

Timler and Kulak (1983) tested a single-story steel plate shear wall to verify the analytical technique established by Thorburn et al. (1983). In the test, a two-panel arrangement of vertically oriented beams and horizontally oriented columns was connected by pinned joints at the four extreme corners. The member sizes were chosen so as to be representative of typical building construction. The specimen was loaded statically, with three complete cycles of loading to a service load deflection limit of $h_s/400$, or 6.25 mm. During these cycles, the test specimen behaved elastically. Subsequently, the shear wall was loaded in one direction to its ultimate capacity. Axial loads were not applied to the columns.

Tromposch and Kulak (1987) tested a shear wall similar to the one tested by Timler and Kulak (1983). However, important modifications were introduced, including the use of bolted shear connections in the frame, a thinner infill panel, and very stiff beam members. (The stiff beams were intended to provide anchorage to the infill panel similar to that expected in a multistory condition with panels above and below.) The column size was selected to be representative of typical building construction.

Prior to the application of lateral loading the columns were preloaded using two full-length prestressing bars at each column. These loads represented the gravity loads present in a building that act concurrently with the lateral loads. Fully reversed cyclic lateral loads were then applied. They were of gradually increasing magnitude and reached a maximum of 67% of the ultimate load obtained subsequently, which corresponded to a maximum deflection of $h_s/129$, or 17 mm. This sequence comprised 28 load cycles. Beyond this load level, the testing machine was capable of loading in one direction only and the final test phase consisted of monotonic loading to the ultimate capacity of the specimen. It was necessary to remove the column prestressing rods prior to the final loading excursion.

The response of the test specimen during the cyclic loading phase showed that behavior was very ductile but that the hysteresis curves were severely pinched because of the very thin infill plate and the flexible boundary frame. The ductility of the specimen was demonstrated even more convincingly during the final monotonic loading excursion up to a maximum deflection of $h_s/31$, or 71 mm. However, the cyclic behavior at this extreme deformation was not investigated.

An experimental program conducted by Elgaaly and Caccese (1990) investigated the behavior of ten one-quarter-scale steel plate shear wall models subjected to cyclic loading. Six of the tests were described in the paper. These tests, and the associated analytical study, were described in further detail in the companion papers Caccese et al. (1993) and Elgaaly et al. (1993). The test specimens were three stories high and one bay wide. Parameters that were varied were the panel thickness and the beam-to-column connection (fixed or shear-type). Panel thicknesses ranged from 0.76–2.66 mm. The test specimens were loaded cyclically with a single in-plane horizontal load at the top of the shear wall, and each specimen was subjected to two series of 24-load cycles. The specimens were subsequently loaded monotonically to failure.

The Elgaaly and Caccese papers were discussed by Kulak et al. (1994) and by Kennedy et al. (1994).

SPECIMEN DETAILS AND TEST SETUP

A diagram of the specimen tested is shown in Fig. 1. The overall height of the specimen, excluding the loading pedestals at the top, is 7.4 m, and the overall width, excluding the base plate, is 3.4 m. The typical story height is 1.83 m (top three stories) and the first story is 1.93 m high. The columns are 3.05 m apart center-to-center. These dimensions are representative of a shear wall at 50% scale for an office building of 3.66 m (12 ft) typical story height, or about 60% scale for a residential building. The test specimen was constructed entirely in a commercial steel fabrication shop using normal industry procedures.

The columns are W310 × 118 sections (W12 × 79) that run through the four stories of the shear wall without splices. Beam sections at levels 1, 2, and 3 are W310 × 60 (W12 × 40) and the beam section at level 4 is a W530 × 82 (W21 × 55, not produced in the United States). All beam and column members have cross-sectional proportions that meet the design requirements for plastic design of beam-columns ("Limit" 1994; *Specification* 1993). Moment connections were used at all beam-to-column joints. Connection of the beam flanges to the columns was made using complete penetration groove

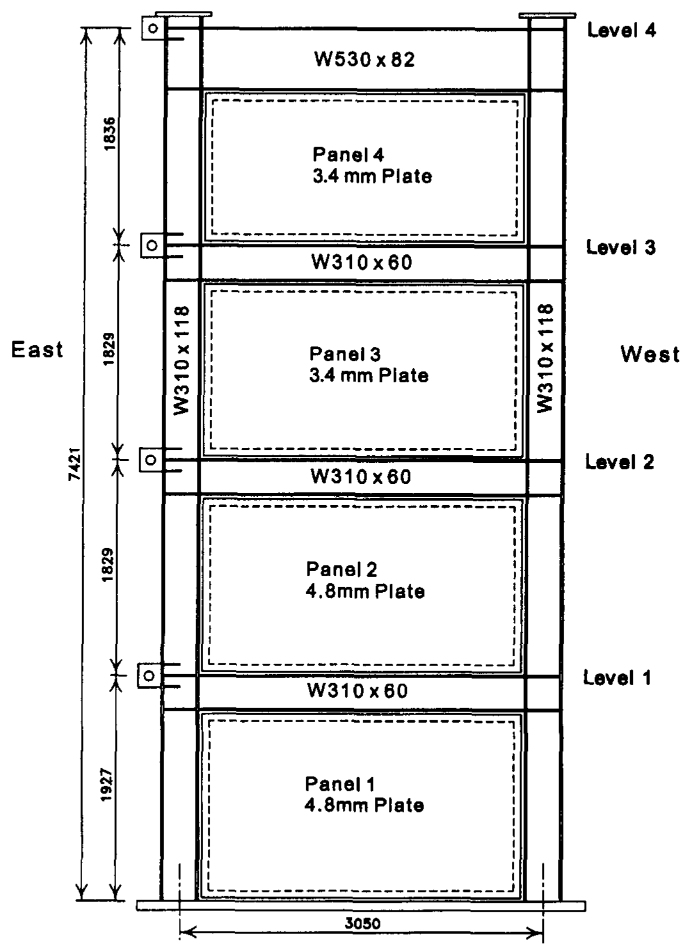

FIG. 1. Steel Plate Shear Wall Test Specimen (North Elevation)

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welds, including a backing bar and runoff tabs that were left in place. Welding access holes of 20 mm radius in the web ensured the continuity of the groove weld from one side of the flange to the other. The beam webs were connected to the column flange by two-sided fillet welds. The columns were connected to the base plate using full penetration groove welds at the flanges and fillet welds at the webs.

Beams with relatively shallow depths were used at all but the top level to reflect the fact that opposing tension fields occur above and below the beam. However, a stiff, deep beam was used at the top level in order to anchor the tension field below. All beams were selected such that out-of-plane buckling would not occur, eliminating the need for intermediate lateral bracing. The beam-to-column moment connection was chosen in order to produce expanded hysteresis curves as compared to a frame with simple connections (Tromposch and Kulak 1987), thereby increasing the amount of energy dissipated. All requirements of clause 27.2 for ductile moment-resisting frames of *CAN/CSA-S16.1-94* ("Limit" 1994) were met.

The panels were connected to the boundary members using the fish plate connection shown in Fig. 2. The continuous fish plates are 100 mm wide and 6 mm thick and are welded to the beams and columns with fillet welds on both sides. (Where column fish plates and beam fish plates meet at the panel corners, small strap plates are used for continuity.) The infill panels are, in turn, fitted against one side of the fish plates, with a lap of approximately 40 mm all around, and then welded with continuous fillets on both sides. This detail allows a simple means of compensating for normal fabrication tolerances in the plane of the plate, thereby avoiding fit-up problems in the field. The fillet weld sizes are capable of developing the ultimate strength of the infill plate.

The mean measured thicknesses for panels 1–4 are 4.54 mm, 4.65 mm, 3.35 mm, and 3.40 mm, respectively, and the panel aspect ratios (height/width) are 0.59, 0.56, 0.56, and 0.48, respectively.

The grade of steel used in the lower two stories is G40.21-300W ("Structural" 1992), and the plate selected is the thinnest plate readily available in this grade. In order to obtain the thinner plate in the upper two stories, commercial-quality hot-rolled steel was selected, which generally exhibits a somewhat lower yield strength than does 300W steel. Grade ASTM A569 plate was used in panel 3, and grade SAE J403 GR1010 was used in panel 4. All infill plate material displayed a stress versus strain response typical of hot-rolled structural steel. The mean static yield strength was 341 MPa for panels 1 and 2, 257 MPa for panel 3, and 262 MPa for panel 4. Complete information on the material properties of both the infill plates and the boundary members is given by Driver et al. (1997).

Gravity loads acting on a deflected shear wall (the P-Δ effect) could have a significant effect on the overall behavior under the action of cyclic horizontal loading. Therefore, vertical

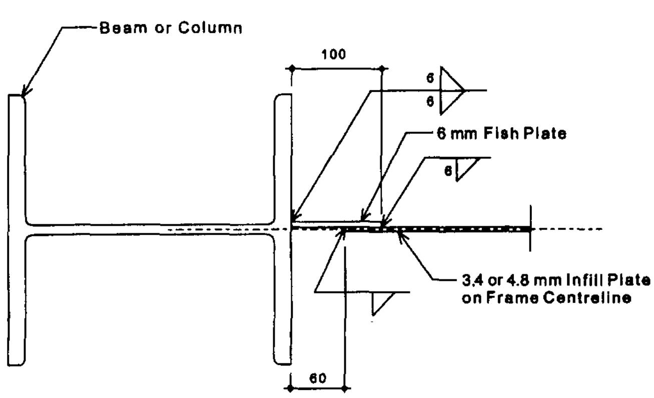

FIG. 2. Fish Plate Detail Used in Test Specimen

loads of a magnitude representing reasonable unfactored gravity loads for a typical building at the lowest story were applied to the columns. To avoid unnecessary complexity in the test setup, these full gravity loads were applied at the tops of the columns.

Equal horizontal loads, representing the action of an idealized earthquake, were applied at each floor level. The relative values of these lateral loads are somewhat arbitrary. They depend upon the earthquake input being modeled, the mass at each level, the mode shapes (which change with time for nonlinear behavior), modal frequencies, and damping ratios (different value for each mode). The ratios of horizontal loads also vary with time. Therefore, equal horizontal loads were considered to be no better or worse than other rational configurations, and the scheme was adopted for its simplicity. The loads were applied at the level of the beam top flange so as to simulate the location of the inertia forces induced by floor masses. Loading in the manner described means that varying combinations of story shear to overturning moment were obtained at the four levels.

The reactions of the hydraulic jacks used to apply the horizontal loads were resisted by the laboratory reaction wall. The vertical loads at the top of each column were applied using hydraulic jacks acting through gravity load simulators (Yarimci et al. 1966). This arrangement maintains the gravity loads in a vertical orientation as the structure sways under the action of the horizontal loads. Bracing was provided at the ends of each beam at each floor level. The bracing was articulated, so that it did not restrain the structure as it underwent the cyclic lateral movement. A large variety and number of load cells, displacement and rotation transducers, and strain gages were used. These provided control of the various loading features (e.g., verticality of the gravity loads), monitoring of the story deflections and other data that were used subsequently to explore the analytical models of the structure. Full details of these devices and the quantities measured, as well as an evaluation of the deduced member forces and panel stresses, are described by Driver et al. (1997).

TEST PROCEDURE

Numerous load and deflection histories could be used to evaluate a structural component for seismic performance. Most slow cyclic tests that are intended to simulate earthquake loading employ a horizontal in-plane load history that uses gradually increasing loads or displacements in successive cycles. Derecho et al. (1980), however, note that in many cases the maximum deformation or an amplitude close to the maximum occurs early in the earthquake response. Therefore, the gradually increasing load cycle history may be unconservative. On the basis of studies of reinforced concrete shear walls, the researchers recommend a loading history that has alternating large and small amplitude cycles. However, the traditional loading sequence, admittedly not a particularly good approximation of typical earthquake actions, has several advantages. First, because it is by far the most widely used approach for investigations into seismic structural performance, it allows comparison with other experimental programs. Second, the low-intensity initial cycles applied early in the test permit unforeseen problems to be sorted out without damaging the specimen. Third, it does not have to be known a priori what the maximum excursion should be in order to fully exploit the capabilities of the system. Finally, a "true" equivalent earthquake load history requires many assumptions regarding factors such as the earthquake input, the floor masses, and the effects of nonstructural elements. This limits the scope of applicability, while at the same time adding unjustified loading complexity.

As a result of this examination, the load and deflection history selected for the shear wall test was based on the method outlined by the Applied Technology Council ("Guidelines" 1992). This document, designated as ATC-24, provides guidance in the selection of loading histories and the presentation of results, thereby simplifying interpretation and comparison among different research projects. The document is specific to slow cyclic load application.

ATC-24 requires that a "deformation control parameter" be selected for controlling the test, and recommends using some parameter related to interstory drift. Drift of the lowest story (panel 1) was selected for the shear wall test because the majority of the deformation and energy absorption takes place in this panel. The force quantity best related to the deformation control parameter is the story 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. Although judgment is involved in selecting the point at which this occurs, the resultant terms δ_y and Q_y are considered sufficiently precise for use as test control parameters. The values may be determined experimentally (from a monotonic load test) or predicted analytically.

The yield displacement (δ_y) in panel 1 was determined during the early stages of the test as 8.5 mm. Prior to reaching this value, single loading cycles leading to shears in panel 1 of ±200 kN, ±400 kN, ±600 kN, ±800 kN, and three cycles each with shears of ±1000 kN and ±1950 kN were conducted to explore the elastic and the initial inelastic behavior. These constituted cycles 1–10. After cycles 11, 12, and 13 with a displacement of $\delta_y = 8.5$ mm, the displacement in the first story was increased by 8.5 mm in each subsequent deformation step. Following the guidelines of ATC-24, three cycles were conducted at each deformation step up to a deformation of $3\delta_y$ (cycles 17–19) and two cycles at each deformation step thereafter.

At a displacement of $5.4\delta_y$ (46 mm), the limit of jack stroke at level 3 was reached in the direction of loading towards the reaction wall. In all subsequent cycles, this peak deformation was maintained while the peak deformation in the opposite direction was increased as prescribed in ATC-24.

Prior to the application of any lateral loads, a gravity load of only 75% of the eventual target value was applied at the top of each column. This load was maintained constant for the first five cycles (up to and including the first cycle at ±1000 kN). This gravity load limit was used so that the effects of these loads at lower values could be explored and the chances of undesirable consequences reduced. For the final two cycles at ±1000 kN, and thereafter, the full gravity load of about 720 kN was applied at the top of each column. Generally, the gravity loads were maintained within 5% of the target value.

APPLICATION OF LOADS AND SPECIMEN BEHAVIOR

During the application of the initial gravity loads when no horizontal loads were present, no yielding of the specimen was apparent. An inspection of the thin infill panels revealed that no plate buckling had occurred.

Yielding was first observed during cycle 8, mostly in the fish plates that connect the infill plates to the boundary members or at the periphery of the infill plates themselves. In addition, characteristic diagonal tension yield patterns also began to form at the top corners of panel 1. By cycle 10, the yielded areas on the fish plates had grown larger.

In cycle 11 (the first cycle with $\delta = \delta_y$), 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, several loud bangs first occurred in this cycle as the plate buckles popped through and reoriented themselves upon reversal of the loading direction. These noises continued to occur in all subsequent cycles.

As loading continued, yielding progressed in various parts of the structure and the amplitude of the buckles in the lower-story panels increased. For example, during cycle 14 the amplitude of the buckle in panel 1 was estimated to be about 50 mm from the neutral position. After unloading of the horizontal forces, residual buckles were clearly visible in a complex surface geometry that did not favor the orientation that formed in either direction of loading.

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 tear did not propagate during subsequent cycles and appears to have had a negligible effect on the behavior of the test specimen.

During cycle 20, local buckles in the west flange of the east column and the east flange of the west column were observed immediately below the beam at level 1. These buckles were of relatively small amplitude but they increased in size during subsequent cycles. After the lateral load had been removed at the end of this cycle, the residual buckle amplitudes were 10 mm in the west column and 40 mm in the east. 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 seen at the top corners of panel 1 at the toe of the fillet weld connecting the fish plate to the columns. One of these was 120 mm long and the other 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. Fig. 3 shows one of the top corners of panel 1 after the tears 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.

In cycle 24, rotation of the flanges at the local buckles in the columns of the lowest story caused substantial yielding in the adjacent column webs.

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. 4 shows one such tear.

By cycle 26, column flange distortion in the first story was extreme. The distortion increased when the column was in compression and decreased when it was in tension.

In cycle 30, a displacement of $9\delta_y$ was achieved. The shear

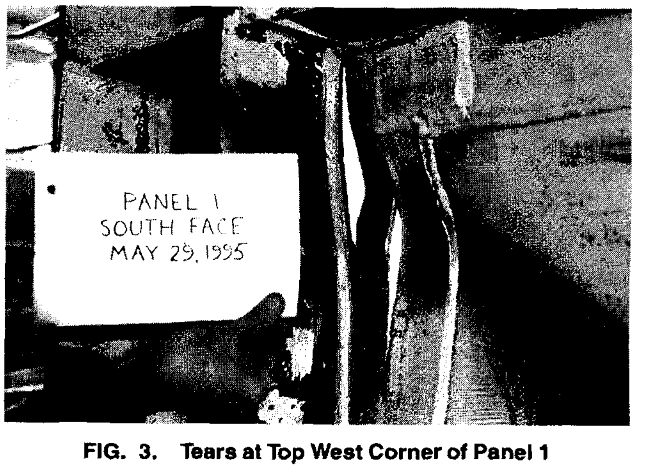

FIG. 3. Tears at Top West Corner of Panel 1

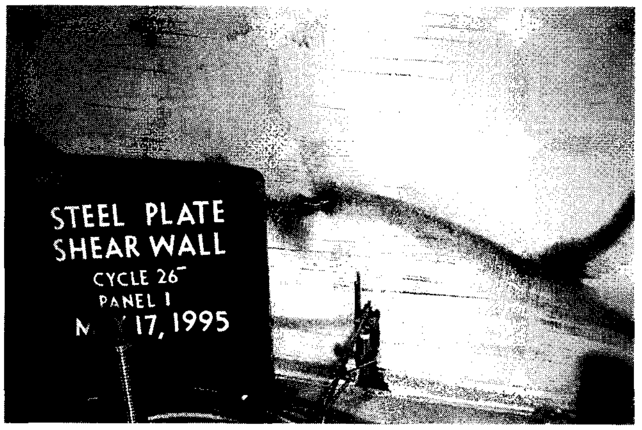

FIG. 4. Tear in Panel 1 Initiated by Cyclic Kinking

wall was then unloaded and was in the process of being reloaded in the opposite direction when, with a large release of energy, the west column fractured at its base. The base shear was approximately 1750 kN. The fracture began at the toe of the weld connecting the west flange of the column to the base plate and propagated through the remainder of the west flange and completely through the web.

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

HYSTERETIC BEHAVIOR

The hysteresis curves, shown in Fig. 5 for Panel 1, exhibit many of the same characteristics of previous tests of unstiffened, thin-panel steel plate shear walls (e.g., Tromposch and Kulak 1987). In the early (elastic) loading cycles, the panel behaves in a stiff manner. As the deformations increase, portions of the steel plate shear wall yield, resulting in a gradually reducing stiffness. After significant yielding of the infill panels has occurred, unloading and reloading in the opposite direction produces a consistent and characteristic hysteresis pattern. Consider the single representative cycle (cycle 22, idealized to form a closed curve) shown in Fig. 6. The unloading curve *ab* has a panel stiffness similar to that in the elastic region, although with the increasing peak deflection in each excursion, the slopes of these unloading curves tend to decrease gradually. As the load reverses, the stiffness reduces substantially (curve *bc*). This reflects the release of the tension field developed in the previous excursion. Because the plate has been stretched inelastically in the previous half-cycle, the diagonal is longer than the opening within the moment-resisting frame upon its return to the neutral (no-load) position. This is manifested during the test by significant out-of-plane buckles that are present when the shear wall is not under load. For the tension field to redevelop in the opposite direction to the point where it again becomes the primary mechanism for resisting the story shear (point *c*), significant story deflection is required. The curves show an increase in stiffness (curve *cd*) because the tension field acts as a diagonal tie and stiffens the story. As the loads again approach the ultimate strength (curve *de*), yielding of the various components of the shear wall (primarily the infill plate) results in another decrease in stiffness. The subsequent curve representing the unloading and reloading of the panel in the opposite direction (curve *efgha*) reflects a repeat of the phenomena described for the curve *abcde*.

Each of the inelastic cycles carried out during the test resulted in the generation of hysteresis curves similar to that described above. The primary difference in the progression of curves is the stiffness reduction of the shear panel during the redevelopment of the tension field and the amount of deflection required in order for the redevelopment to occur, as may be deduced from Fig. 5. In order to assess the contribution of the infill plates themselves to the strength and stiffness of the structure, the frame was analyzed both with and without the panels. The results for panel 1 are presented in Driver et al. (1998).

The maximum load achieved in each cycle increased slightly with each excursion to a new deflection level until the maximum base shear of 3080 kN was reached in cycle 22. This took place at a deflection of $5\delta_y$. Subsequently, the load-carrying capacity of the shear wall declined very gradually from cycle to cycle. Cycle 22 was also the cycle where panel tears first occurred. These tears, along with the local buckles in the column flanges that began forming in cycle 20, are

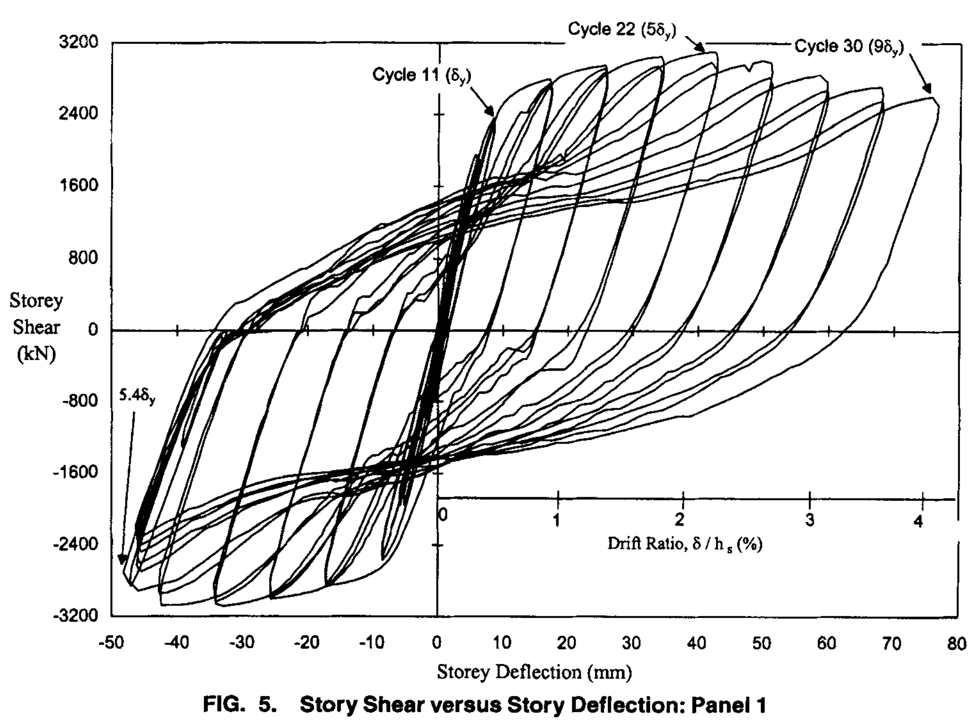

FIG. 5. Story Shear versus Story Deflection: Panel 1

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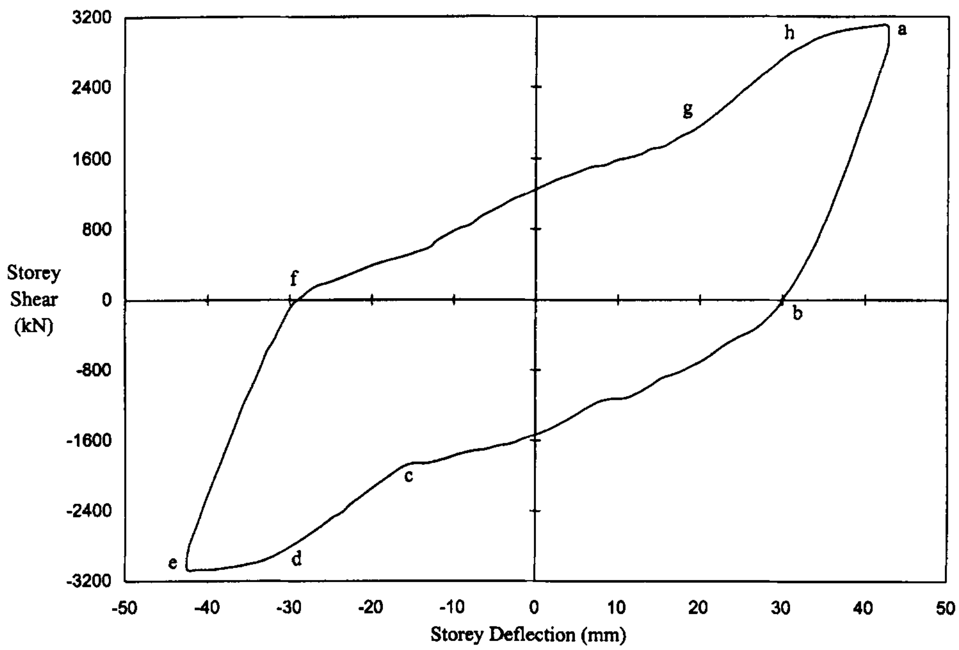

FIG. 6. Typical Loading Cycle: Panel 1

considered to have contributed to the gradual degradation of the specimen.

The family of curves in Fig. 5 would be antisymmetrical had the deflections in panel 1 not been limited to a maximum of $5.4\delta_y$ in the direction towards the reaction wall, as discussed earlier. Thereafter, deflections could be increased in the direction away from the wall only. Hence, the curves shown in Fig. 5 are antisymmetrical only up to this deflection level. This limitation is only of consequence, however, after the maximum capacity of the shear wall was reached. Furthermore, it is considered unlikely to have had a significant effect on the cycles after the maximum shear was reached for excursions in the opposite direction.

A total of 30 cycles of loading were applied to the test specimen prior to failure. In the last 20 cycles, the specimen was loaded well beyond the point of significant yielding. This is probably more severe than the number of inelastic cycles that a shear wall would be expected to resist during an earthquake. For example, Derecho et al. (1980) determined the response histories of concrete shear walls for earthquakes of 20-s duration. A broad range of structural periods and seismic frequency characteristics were studied. For a total of 170 cases, the number of fully reversed large-amplitude cycles was fewer than four in 95% of cases, with an extreme value of six. (A fully reversed cycle is defined as one in which an excursion of 0.75–1.0 times the maximum amplitude is followed immediately by an excursion in the opposite direction of at least 0.5 times this maximum.)

Fig. 5 clearly demonstrates the significant ductility exhibited by panel 1. Although the maximum deflection achieved was nine times the deflection at which significant yielding took place, as defined in the ATC-24 protocol, the true ductility exhibited by the system is even greater. Popov (1980) defines the displacement ductility factor as the ratio of the maximum horizontal deflection of a structure at a selected story to the deflection at the point of significant yielding. Furthermore, the maximum horizontal deflection is taken as the total inelastic excursion during a complete half-cycle. This recognizes the increased demand on an inelastically deformed structure that must deform significantly to reach the neutral position prior to the next inelastic loading excursion in the opposite direction. This, with a deflection of $5.4\delta_y$ in one direction followed by a deflection in the other of $9\delta_y$, the displacement ductility factor for panel 1 of the tested steel plate shear wall is actually 14.4. Had the jack stroke at level 3 not been restricted, the displacement ductility factor based on a half-cycle would have been even greater. Preventive measures to eliminate the local buckling of the column flanges that eventually led to fracture of the column would also have increased the ductility.

The area enclosed by the hysteresis curves is a measure of the energy dissipated by the system in resisting the particular load or displacement history. Fig. 5 shows that the curves generated are relatively wide, indicative of significant energy dissipation during each cycle. The curves exhibit some pinching due to the reduced stiffness in the region where the plate buckles reorient themselves during a load reversal and the tension field is not fully developed. However, the area enclosed is distinctly greater than the area enclosed by curves generated for steel plate shear walls with only shear-type beam-to-column connections (Tromposch and Kulak 1987). Therefore, the effect of constructing the shear wall with a moment-resisting frame is to increase greatly the amount of energy dissipated, thereby improving seismic performance. Furthermore, the amount of energy dissipated increased with each successive cycle of increased deflection. These aspects are discussed fully in Driver et al. (1997). The moment-resisting frame also provides redundancy in carrying loads that is beneficial for seismic applications.

The uniformity of the hysteresis curves indicates that the behavior of the shear wall specimen under severe cyclic loading was not only very ductile, but also extremely stable. There was no sudden loss of stiffness at any point in the test. Even after the peak load had been reached, deterioration was slow and controlled. Tearing of the infill plates, which is itself a mechanism for dissipating energy, also occurred in a gradual manner: increases in tear lengths in any given cycle were only incremental. The main reason that tearing does not result in a sudden decrease in stiffness is that the continuous infill plate effectively redistributes loads to areas unaffected by the tearing. The ability of the panels to redistribute load provides a further redundancy in the lateral load-resisting system. The efficiency of this stress redistribution is also reflected by the fact that the tears had little effect on the overall strength of the shear wall.

The moment-resisting joints at the beam-to-column connections showed no signs of yielding in the panel zones until 16

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cycles of loading had been applied. (Only flange continuity stiffeners were used as panel stiffening.) Even during cycle 17 the extent of yielding was limited, with only slight flaking of the whitewash detected in the most heavily loaded joints. This point in the displacement history corresponds to a deflection in panel 1 of $3\delta_y$. At the end of the test, inelastic deformations in the joint panel zones remained small. Therefore, the primary element providing ductility in the shear wall system is the infill plate. In the terminology of clause 27 (Seismic Design Requirements) of the Canadian steel design standard ("Limit" 1994), the infill plates would be considered the "critical elements," because they undergo large plastic deformations. In effect, the presence of the infill plates reduces the demand on the joint panel zone and distributes the energy-absorbing mechanism over a much larger volume of material, while at the same time stiffening the frame significantly. Because of the presence of the infill plate, there is a reduction in reliance on the moment-resisting frame for resisting the story shears. Therefore, the joint panel zone would generally not be a critical element. It is likely that it could be designed to remain essentially elastic.

Engelhardt and Sabol (1994) and Yang and Popov (1995), among others, have investigated the connection failures in moment-resisting frames that occurred in the Northridge earthquake of Jan. 1994. Many of the Northridge fractures initiated at the notch formed by the backing bar used during the welding operation at the beam flanges. The Federal Emergency Management Agency (FEMA) ("Interim" 1995) now recommends removal of the backing bar and runoff tabs, and back gouging prior to depositing new weld material. The procedure is then completed by grinding the weld and testing it using nondestructive techniques. All these operations are expensive and reduce the economic competitiveness of steel frames. The shear wall test described herein indicates that the demands on the connections are significantly reduced in steel plate shear walls. In the fabrication of the shear wall assembly, the FEMA procedures were not used and the backing bars and runoff tabs were left in position. However, no distress of any kind was noted during the test in the areas of the beam-to-column connections, even though very large deformations were imposed for a large number of cycles. It is recognized, however, that the Northridge connection failures occurred under higher strain rates and that the flanges and other plates were generally thicker.

It is of interest to compare the large deflections imposed upon the lowest story with the seismic drift limitations prescribed by building codes. The *National Building Code of Canada* (1995) specifies a limitation on interstory drift of $0.01h_s$ for postdisaster buildings and $0.02h_s$ for all other buildings. For the test specimen, this is equivalent to a deflection at level 1 (h_s = 1928 mm) of 19.3 mm ($2.3\delta_y$) for postdisaster buildings and 38.6 mm ($4.5\delta_y$) for other buildings.

The FEMA (*NEHRP* 1994) recommendations limit the interstory drift to values that depend upon the seismic hazard group. Buildings in group 3 (those having essential facilities that are required for postearthquake recovery), group 2 (those having a substantial public hazard due to occupancy or use), and group 1 (the remainder) are limited to an interstory drift of $0.010h_s$, $0.015h_s$, and $0.020h_s$, respectively. For the test specimen, these deflection limits at level 1 are 19.3 mm ($2.3\delta_y$), 28.9 mm ($3.4\delta_y$), and 38.6 mm ($4.5\delta_y$), respectively. In buildings of four stories or fewer and where the nonstructural elements are specially designed to accommodate large displacements, more liberal limits may be used. The largest permissible interstory drift under these circumstances is $0.025h_s$ for buildings in seismic hazard group 1. For the test specimen, this is equivalent to a deflection at level 1 of 48.2 mm ($5.7\delta_y$).

Without special requirements for the nonstructural elements, the range of permissible levels of interstory drift in these documents is 19.3 mm ($2.3\delta_y$) to 38.6 mm ($4.5\delta_y$) for the test specimen. In the test, the ultimate strength was reached in cycle 22 with a deflection at Level 1 of 42.5 mm ($5\delta_y$), which is greater than the drift limitations cited. So, even if drift were to control the design of the shear wall, deflections would not be expected to reach the post-ultimate region. Indeed, for the most liberal case, where deflections of $0.025h_s$ are allowed, the resulting deflection in the test specimen would still not lead to any significant strength degradation. The deflections imposed during the test were, therefore, much more severe than would take place in a properly designed structure. In any case, if ductility demands of this magnitude were required of the structure during an earthquake, the physical test has demonstrated that no sudden loss in stiffness will occur in a well-detailed steel plate shear wall.

FAILURE MODE

The shear wall specimen failed by sudden fracturing of the west column at its base. The fracture began in the heat-affected zone of the outer column flange tip near the toe of the complete penetration groove weld that connected the column to the base plate. The crack began at the flange tip and propagated approximately 30 mm along the flange in a relatively tough manner, as deduced from the 45° shear lips observed on the fracture surface. Examination of the fracture surface showed that of this 30 mm, only the last 10 mm occurred in the final cycle of loading (cycle 30). This portion of the crack exhibited classical features of ductile failure. The initial 20 mm of the crack surface (at the edge of the flange) had been deformed in compression during one or more of the earlier load cycles, obscuring the surface features.

When the crack reached 30 mm in length, the cross-sectional area had been sufficiently reduced such that a brittle cleavage fracture was initiated. The remaining portion of the failure surface was perpendicular to the plane of the flange. Growth occurred completely through the outer flange and then through the web to the inside of the opposite flange. This final fracture happened suddenly and with a large energy release.

Although failure of the column occurred at a very large frame deflection and after many inelastic reversals, the failure mode is nevertheless of importance in assessing the suitability of any structural system for seismic applications. Certainly, the failure mode of sudden fracture of the column is not desirable and should be avoided.

The reason for the failure mode is evident. During cycle 20 (near the ultimate load), local buckles began forming in the column flanges at the base, with one-half of the flange buckling outward and one-half inward, with the web-to-flange junction acting as a node. As the deflections became more severe, the amplitudes of these local buckles grew. At large story deflections, the buckle amplitudes at the column bases were very large when the column was in compression. As the column became subject to tension when the loads were reversed, the local buckles tended to straighten. The resultant severe cyclic bending of the flanges at the column base eventually caused the material to fracture. The problem was compounded by the fact that as the flange became loaded in tension, the heavy welding at the column base effectively restrained the orthogonal contractions that result from the Poisson effect. This creates a condition of triaxial tensile stresses that can significantly reduce the local ductility of the material (Blodgett 1994).

One way to prevent the local flange distortions that contributed to the fracture of the column is to install full-depth horizontal stiffeners between the column flanges. Adding column stiffeners near the top and bottom of each story in

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locations where strains are expected to be several times the yield strain would restrain the formation of these local buckles. The test demonstrated that local buckles may form in the absence of sufficient stiffening under extreme story deformations even if beam-column sections suitable for plastic design are used, because the ductility demands can far exceed those required to reach and maintain the plastic moment. The most critical location for stiffening is at the column base, where the presence of heavy welds increases the likelihood of a low-cycle fatigue failure or brittle fracture. Nondestructive inspection of these welds to ensure high quality is also recommended.

Another measure that may improve the performance of the connection at the column base is to force the plastic hinge away from the area that is restrained by welding. This could be achieved by adding cover plates to the flanges. Tapered cover plates could be used to avoid an abrupt change in cross section. Further tests are required to fully explore and evaluate the detailing options.

Because the shear panel itself retained its integrity when the column fracture occurred, had this failure mode been prevented, there is reason to expect that the very gradual deterioration of capacity would have continued. However, because the load-carrying capacity of the test specimen during cycle 30 had already decreased to 85% of the maximum, no repair of the column was attempted.

SUMMARY AND CONCLUSIONS

A four-story, single-bay steel plate shear wall with moment-resisting beam-to-column connections was tested under idealized earthquake-type loading. The steel infill plate was unstiffened. Gravity loads of constant magnitude were applied to each column and cyclic in-plane horizontal loads were applied at each floor level. The cyclic deflection amplitudes were gradually increased according to the recommendations outlined in ATC-24 until significant degradation of the shear wall was evident. The capacity of the test specimen increased until a deflection of five times the deflection corresponding to significant yield was reached, after which degradation was slow and stable. The cycle in which the maximum shear strength was reached coincided approximately with the inception of plate tearing and local column flange buckling in the lowest story. Prior to failure of the specimen, a deflection in the lowest story of nine times the yield deflection had been imposed.

The four-story steel plate shear wall test specimen exhibited excellent performance. The hysteresis behavior indicates that the shear wall configuration tested possesses an extremely high degree of ductility. At the end of the test, the main ductile component of the shear wall—the infill panel—was still able to carry shears equal to about 85% of those resisted at the ultimate load level. The hysteresis curves were very stable throughout the response and did not show any sudden reductions in strength. The post-ultimate degradation was slow and controlled. Significant degradation occurred only after a large number of displacement cycles and at very large deflections. Furthermore, the amount of energy dissipated during the loading cycles was significantly greater than that shown by similar shear walls but with shear-type beam-to-column connections. The amount of energy dissipated also increased steadily with each cycle of increased deflection. Based on this large-scale test, it is concluded that the steel plate shear wall configuration tested represents an excellent lateral load-resisting system for seismic loading.

Because the story shears are resisted largely by the stiff infill panels, beam-to-column joint demands tend to be less severe than for moment-resisting frames without infill panels. Measured joint moments were relatively small in all cases and no signs of joint distress, as occurred in the Northridge failures, were observed. Very little yielding was observed in the joint panel zones of the test specimen, even at large deflections. Because of the significant beneficial effect of the presence of the infill panels on the behavior of the frame joints, it is unlikely that special design procedures [e.g., "Interim" (1995)] specifically intended for preventing the Northridge-type joint failures in moment-resisting frames are needed.

Towards the end of the test, where large deflections were imposed, severe local flange buckling occurred in the columns of the lowest story immediately above the base and also below the beam at level 1. Under dynamic earthquake loading, this condition could occur in other stories. It is recommended that, even for cross sections suitable for plastic design applications, column flanges be stiffened to prevent local buckling at locations in the frame where analyses indicate that fully plastic behavior is likely to occur. In any case, the interface between the shear wall and the foundation forms an abrupt change in stiffness of the lateral load-resisting system. The local buckling deformations there led to the eventual fracture at the base of the column during the test. Therefore, it is recommended that in all cases precautions to prevent local buckling at the column bases be pursued.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

h_s = story height;
Q_y = load at the point of significant yielding;
δ = deflection; and
δ_y = deflection at the point of significant yielding.

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