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Large-Scale Test on a Four Storey Steel Plate Shear Wall Subjected to Idealized Quasi-Static Earthquake Loading

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ABSTRACT

The latest standard for design of steel structures in Canada, CAN/CSA-S16.1-94, includes, for the first time, an Appendix addressing the design of thin-panel steel plate shear walls, together with general rules for seismic design. The methodology for design and analysis described in the Standard is based primarily on recommendations from the results of research conducted at the University of Alberta (Thorburn et al. 1983, Timler and Kulak 1983, and Tromposch and Kulak 1987) that included large-scale steel plate shear wall tests. The tests conducted were of single-storey shear wall panels employing either true pins or standard bolted shear-type connections at the beam-to-column joints. Extrapolation of these results to multi-storey applications has had to be based on computer analysis and engineering judgement.

Because there is a lack of large-scale multi-storey test data, it was concluded that a cyclic test on a four storey shear wall would provide useful information in continuing the development of the Canadian design approach. Therefore, a single large-scale multi-storey test has been planned and will be conducted early in 1995. With gravity loads applied to the columns, the test specimen will be subjected to an idealized earthquake, represented by a controlled quasi-static cyclic loading program. The test will be completed by the time this paper is published and more details of the test will be presented at the conference.

INTRODUCTION

Steel plate shear walls are lateral load resisting systems for buildings consisting of vertical steel plates one storey high and one bay wide welded all around to the surrounding frame 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. The surrounding steel frame may be either simple or moment-resisting. Steel plate shear walls are well-suited both for new construction and for providing a relatively simple means for seismic upgrading. They possess many properties that are fundamentally beneficial in resisting seismic loads including redundancy, ductility, and robust resistance to degradation under cyclic loading. They also exhibit beneficial qualities such as construction simplicity, fabrication repetition, and low self-weight.

CSA Standard CAN/CSA-S16.1-94 provides design rules for steel plate shear walls, including some guidance for seismic applications. The test of Tromposch and Kulak (1987) on a pair of single storey shear walls showed that this type of shear panel exhibits extremely stable hysteresis behaviour under cyclic loading. Since the other test data on which these design rules are primarily based (Timler and

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Kulak 1983) are also from single storey shear walls, a large-scale multi-storey steel plate shear wall test will be conducted to provide additional information for the development of design and analysis procedures. Of primary interest is the relative contributions of the moment-resisting frame and the infill panels to the stiffness of the shear wall during various cycles of the loading history. This test will also determine the ductility and energy absorption capabilities of such a frame under reversing loads.

TEST SPECIMEN DESCRIPTION AND RATIONALE

The set-up for the shear wall test is depicted in Figs. 1 and 2. The test specimen, fabricated using industry-standard details and methods, represents a single bay shear wall from a four storey building at 50% scale for an office building of 3.66 m storey height, or about 60% scale for a residential building. The shear wall consists of a steel moment-resisting frame infilled with thin steel plates. The overall height of the specimen, excluding the loading pedestals at the top, is 7.5 m and the overall width, excluding the base plate, is 3.4 m. The typical storey height is 1.83 m (top three storeys) and the first storey is 1.94 m high. The columns are 3.05 m centre-to-centre. At 50% scale, this could represent the back wall of a two-bay elevator shaft. As-built geometry will be determined prior to the test, including beam and column cross-sectional dimensions, plate thicknesses, and column out-of-plumbs.

Column sizes were selected to maintain stability through to the ultimate strength of the shear wall. Heavy columns would be expected in a lateral system in a severe earthquake zone. A Class 1 (plastic design) section is used in order that local buckling be precluded. One column size is used throughout the height of the shear wall so as to avoid a column splice.

Shallow beam sizes are used at all but the top level to reflect the fact that tension fields above and below the beams tend to be vertically self-equilibrating. However, a stiff, deep beam is used at the top level in order to anchor the tension field below. Beams with relatively wide flanges are chosen to eliminate the need for intermediate lateral bracing.

Full moment connections are present at all beam-to-column joints. Full penetration groove welds develop the flange capacity. This configuration is expected to expand the hysteresis curves as compared to a simple framed connection, thereby increasing the amount of energy absorbed. All requirements for Ductile Moment-Resisting Frames in Clause 27.2 of CSA Standard CAN/CSA-S16.1-94 have been met.

Plate thicknesses of 4.8 mm in the lower two storeys and 3.4 mm in the upper two storeys were selected. The first and third storey panels are, therefore, expected to be critical. The plate thicknesses, although somewhat arbitrary, have been selected to represent lower limits for plate that exhibits the classical *linear elastic–perfectly plastic plus strain hardening* stress vs. strain curve of hot-rolled structural quality steel, as would be expected in a real structure. This characteristic is considered to be extremely important for applicability of the final test results. Due to the potential for cold-working of such thin plates during rolling, tension coupon tests have been conducted to confirm the desired behaviour and to ensure that the steel is neither grossly over- nor under-strength. Material displaying cold-worked characteristics was discarded and appropriate plate material has been set aside for use in the multi-storey test specimen. The grade of steel used in the lower two storeys is 300W, and at 4.8 mm, is the thinnest plate readily available in this grade. In order to obtain the thinner plate, hot-rolled commercial quality steel was selected which generally exhibits a somewhat lower yield strength than does 300W steel, but possesses similar stress vs. strain behaviour. The material selected has a yield strength of 250 MPa.

The panels are connected to the boundary members using a "fish plate" connection as shown in Fig. 3. The continuous fish plates are 100 mm wide and 6 mm in thickness and are welded to the beams and columns by means of fillet welds. Where column fish plates and beam fish plates meet at the panel corners, they are connected together using small strap plates to provide continuity. The infill panels are, in turn, fillet welded against one side of the fish plates. This detail allows a simple means of compensating for normal fabrication tolerances, thereby avoiding fit-up problems in the field. The fillet welds connecting the infill plate to the fish plate and the fish plate to the boundary members are capable of developing the ultimate strength of the infill plate.

The shear wall will be braced out-of-plane at the ends of each beam at each floor level (eight locations) using a Watt-type articulated bracing system. One such brace is illustrated in Fig. 2. A derivative of a similar brace originally developed at Lehigh University (Yarimci et al. 1966), this brace is able to accommodate large in-plane displacements of the shear wall without offering any restraint or requiring any manual adjustments. It consists of three rigid links connected with ball-and-socket joints, plus ball-and-socket attachment brackets at each end. The brace point on the test specimen is located at the mid-point of the centre link. The brace points at the lower three beams are located on the columns 100 mm below the W310x60 beam bottom flanges in order that the centre brace link can clear the horizontal loading clevises. At the top level, the brace points are at the W530x82 beam top flange.

The 80 mm thick continuous steel base plate will be anchored to the strong floor using 14 two inch diameter high strength steel anchor bolts. Due to the large overturning moments expected at the base of the wall, the anchor bolts will be prestressed to the strong floor so as to minimize the amount of movement due to elongation of the anchor bolts during the test.

The shear wall test will take place on the strong floor of the Centre for Frontier Engineering Research (C-FER) in Edmonton, Alberta. The C-FER facility is being used because a strong wall, built integrally with the strong floor, greatly simplifies the test set-up. Horizontal loads will react against the 500 mm thick strong wall, supported by massive concrete buttresses. The geometry described above is, therefore, constrained by the location of the anchor holes in the C-FER strong wall and floor. Most equipment and technical support will be provided by the I.F. Morrison Structures Laboratory at the University of Alberta.

LOADING HISTORY AND DESCRIPTION

The test will be conducted under a combination of constant vertical loads applied to the columns and gradually increasing cyclic horizontal (in-plane) loads. The first set simulates gravity loads; the second simulates earthquake loads. It should be noted that most quasi-static tests simulating earthquake loading employ a horizontal load history with gradually increasing loads 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 on reinforced concrete shear walls, they recommend a loading history that has alternating large and small amplitude cycles. However, the "traditional" loading sequence, although not a great approximation of typical earthquake actions, has several advantages. First, it is by far the most widely used approach for investigations into seismic structural performance and this allows comparison with other experimental programs. Second, the low intensity initial cycles permit any unforeseen problems to be sorted out without damaging the specimen. Third, one does not have to know in advance what the maximum excursion should be in order to fully exploit the capabilities of the system. Finally, a "true" quasi-static equivalent earthquake load history requires many assumptions regarding the earthquake input, floor masses, effects of non-structural elements, etc. This limits the scope of applicability and adds loading complexity that is simply not justified when examining the type of information sought. In any event, provided that the test behaviour can eventually be predicted, the loading sequence becomes less significant.

Horizontal loads are applied at the level of the beam top flanges to simulate—in an approximate manner—the location of inertia forces induced by floor masses. It is also convenient to load the wall where a stiffener already exists. Equal horizontal loads will be applied at each floor level by means of 890 kN hydraulic jacks. The relative values of the loads are arbitrary. They depend upon the earthquake input being modelled, the assumed mass at each level, the modal shapes (which change with time for non-linear behaviour), modal frequencies, and damping ratios (different value for each mode). Not only are the ratios of horizontal loads arbitrary, they also vary with time. Therefore, for simplicity, in this test the horizontal loads are assumed equal at each level. This is considered to be no better or worse than other rational schemes. With this loading scheme, varying combinations of storey shear and overturning moment will be obtained at the four levels.

Load control will be used for the horizontal loading at the beginning of the test. Loading will be cyclic, starting with low intensity cycles and progressing to increasingly higher intensities. Two cycles will be conducted at each load level. When loads reach a level near the ultimate capacity of the shear wall, displacement control will be used to apply the most severe cycles to a displacement equal to a value established on the basis of the performance of the wall to this point.

A constant vertical load of 720 kN will be applied to the tops of each column throughout the test by four 645 kN hydraulic jacks through a cruciform-shaped distributing beam, as shown in Figs. 1 and 2. The excess jack capacity reduces the likelihood of oil leakage when pressures are maintained for a long duration. Each jack can be individually controlled for precise levelling of the distributing beams at the beginning of the test to ensure proper seating on the column tops. The vertical loads selected represent reasonable unfactored gravity loads for a typical four storey building at the lowest storey. Although the loads are perhaps greater than would be present in the upper storeys, the columns are also oversized accordingly, as described above. The stress in the columns is about 50 MPa with no horizontal load applied.

Gravity Load Simulators, developed at Lehigh University (Yarimci et al. 1966), are used in applying the vertical loads. These devices (shown in Figs. 1 and 2) form a pin-jointed mechanism that keeps loads oriented close to the vertical throughout large horizontal in-plane displacements without the need for manual adjustments and without causing additional restraints to the specimen. The working capacity of each Gravity Load Simulator is 420 kN, and therefore four are required in total to apply the 720 kN loads to each of the two columns.

INSTRUMENTATION

Commercial, flat load cells of 890 kN capacity will be used to measure vertical loads applied to the tops of the columns. Four strain gauges applied to each tension rod in a Wheatstone bridge configuration permit calibration of the rods, so that they act as additional load cells. They are used for determining the vertical loads applied by the jacks connected to the Gravity Load Simulators, providing redundancy of measurements through the requirement of vertical static equilibrium.

Custom tension/compression load cells manufactured at the University of Alberta have been fitted to the 890 kN hydraulic jacks for measuring horizontal loads applied at each level. Before applying the high sensitivity strain gauges, these load cells were each cycled through ten complete loading reversals slightly beyond the predicted test load range to be measured. This procedure improves the linearity of the calibration curve. Because there is no redundancy of measurement for the horizontal loads, calibration has been carefully and accurately executed.

Vertical displacements will be measured at the tops and bases of the columns. Horizontal in-plane displacements at the floor levels and horizontal out-of-plane displacements at each end of the beams at every floor level will also be measured. All displacements will be determined using large displacement cable transducers. In addition, the inclination of the tension rods connected to the Gravity Load Simulators, which is expected to be very small, will be monitored.

Longitudinal strains will be measured using electrical resistance strain gauges on the column cross-sections, concentrated at the bottom where the strains will be highest. Strain rosettes will be used to measure the strain state in the plates in order that principal stresses and their orientations can be determined.

ANCILLARY TESTS

A limited number of tension coupon tests of the plate material have already been conducted (in two orthogonal directions) in order that the suitability of the plate for these tests be established, as described above. More tests will be done after the completion of the shear wall test. Also, additional lengths of the column and beam sections have been ordered for material tests to be conducted after the completion of the shear wall test.

A large-scale corner test has already been executed to assess the performance of the fish plate corner detail to be used. Satisfactory performance in this preliminary test formed the basis of its selection for the main test. It is anticipated that other possible forms of the connection detail will be explored later in the program.

ANALYTICAL PREDICTIONS

The test specimen has been modelled using the commercial finite element program ABAQUS running on a SUN workstation in a UNIX environment. Doubly-curved 8-node shell elements have been used to represent the plates and quadratic beam elements represent the surrounding frame. A 6x9 element mesh is used for the lowest panel and a 5x9 mesh is used in all other panels, resulting in a total of 5638 degrees of freedom.

Initial plate imperfections were specified that duplicate the first buckling mode of the plate panels, although further study revealed that the overall shear wall behaviour is not sensitive to the configuration of initial buckles. The material behaviour for both the plates and frame is linear elastic-perfectly plastic, with the strain hardening represented by a multi-linear curve.

Because the computing demands of the problem are high, monotonic loading was used to estimate the envelope load vs. deflection curve up to the ultimate capacity of the test specimen. The model was then completely unloaded and reloaded again well into the inelastic region. The predicted Total Load vs. Total Deflection and Storey Shear vs. Storey Deflection curves are shown in Figs. 4 and 5, respectively. The total load is the sum of all horizontal loads applied and the total deflection is the absolute deflection at the roof level. These curves give information on required jack and reaction wall capacities and required ranges of motion for the jacks and instrumentation, as well as predicting the behaviour of the steel plate shear wall specimen. As can be seen from Fig. 5, some plasticity is expected in each panel, which is desirable for energy absorption, although the majority of the inelastic deformation will occur in the lowest storey. The finite element analysis also predicts that the demand on the moment-resisting frame will not be severe, resulting in only localized inelasticity. In this sense, the infill plates can be considered the "critical elements" according to the terminology of Clause 27 in CSA Standard S16.1.

SUMMARY

A large-scale, four storey steel plate shear wall consisting of thin plates within a moment-resisting frame will be tested under both constant gravity loads and cyclic, horizontal loads. The horizontal loads will be applied quasi-statically and the cycles will be of gradually increasing intensity representing an idealized earthquake. The latter cycles will take the shear wall well into the inelastic region in order that an assessment of ductility and energy absorption can be made. A finite element study has revealed the expected general behaviour of the test specimen and this information has used in the test design.

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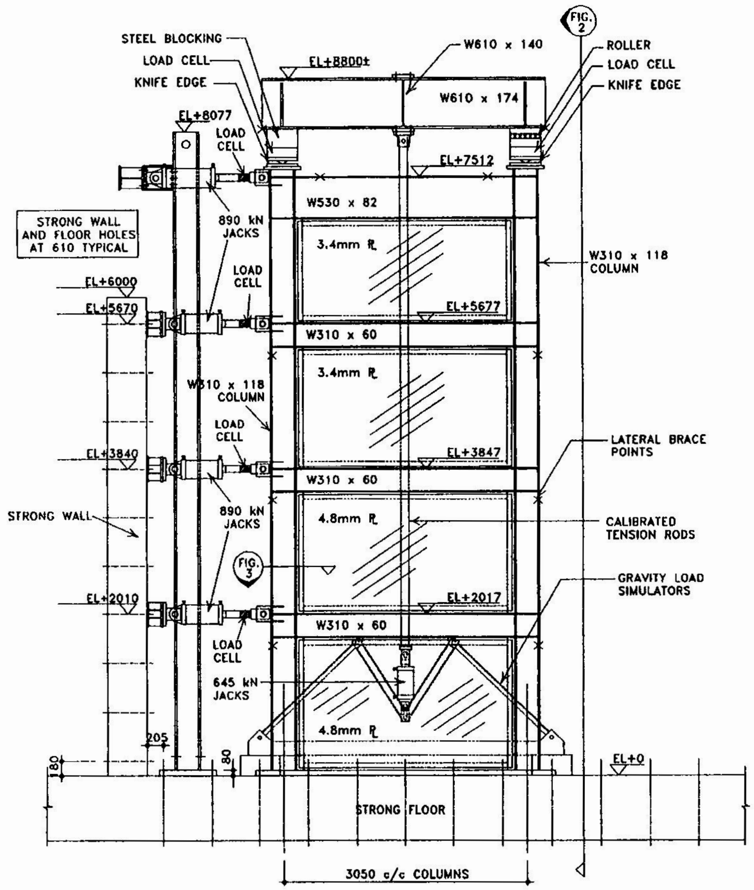

Fig. 1. Shear Wall Test Set-up
Side Elevation

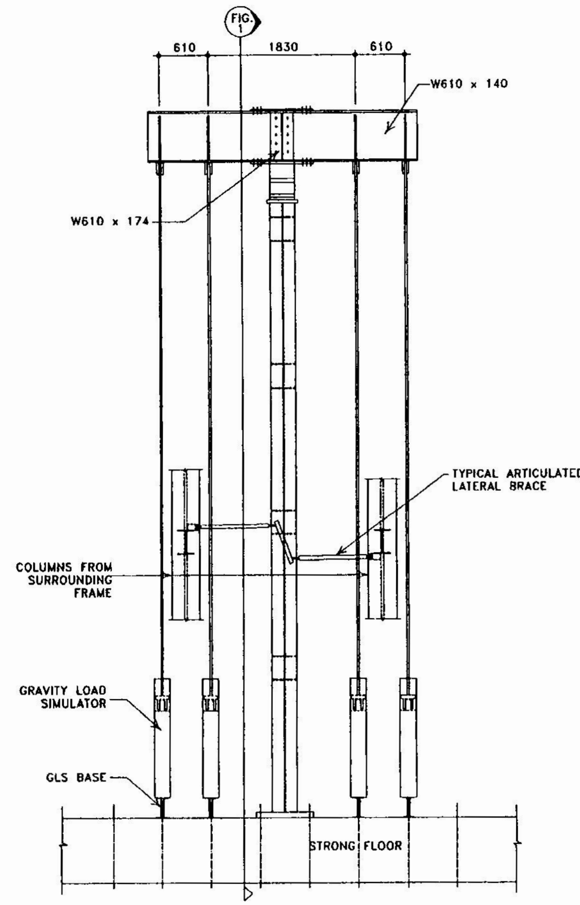

Fig. 2. Shear Wall Test Set-up
End Elevation

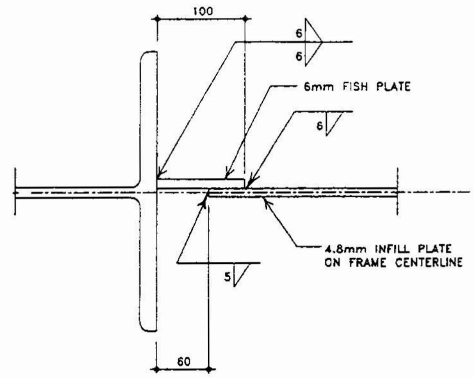

Fig. 3. Typical Fish Plate Detail

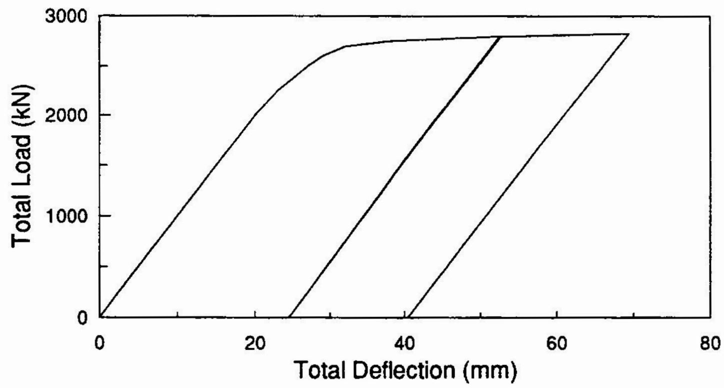

Fig. 4. Predicted Total Load vs. Total Deflection

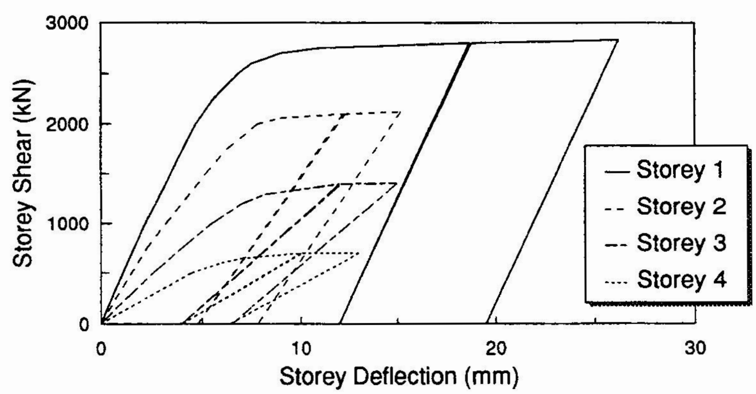

Fig. 5. Predicted Storey Shear vs. Storey Deflection