Steel Plate Shear Walls—An Overview

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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 vertical steel infill plates one story high and one bay wide connected to the surrounding beams and columns. The plates are installed in one or more bays for the full height of a building to form a stiff cantilever wall. North American practice is to use unstiffened plates. Steel plate shear walls are well-suited for new construction, and they offer a relatively simple means for the seismic upgrading of existing steel or concrete structures.

Several researchers have conducted tests on single-story and small-scale multistory laboratory specimens. The most significant physical testing has been that of a large-scale four-story, single bay specimen. Tested under controlled cyclic loading to determine its behavior under an idealized severe earthquake event, it endured 30 cycles of loading, including 20 cycles in the inelastic range. It showed excellent ductility and energy dissipation characteristics, and exhibited stable behavior at very large deformations and after many cycles of loading. Both a non-linear finite element model and a plane frame analysis model suitable for design office use are available.

The seismic performance of the steel plate shear wall concept has been further evaluated using a hypothetical multi-story building located in Vancouver, Canada. The examination was done in accordance with the National Building Code of Canada. The seismic response is assessed with a linear static analysis and a response spectrum analysis, both standard analysis procedures in seismic design practice. A nonlinear static "pushover" analysis was also performed to determine the inelastic static response. The inelastic dynamic response was obtained from nonlinear dynamic time history analyses using a set of appropriately selected earthquake accelerograms.

INTRODUCTION

Steel plate shear walls are an innovative lateral load-resisting system capable of effectively bracing a building against both wind and earthquake forces. This type of shear wall consists of vertical steel plates—referred to as infill plates—one story high and one bay wide connected to the surrounding beams and columns. The surrounding steel frame may use either simple or moment-resisting beam-to-column connections. The arrangement would typically be used in two or more parallel bays for the full height of a building to form stiff cantilever walls, as shown in Figure 1.

Although the infill plates can be either stiffened or unstiffened depending on the design philosophy, labor costs in North America indicate that unstiffened panels are preferable. Steel plate shear walls are well-suited for new construction and are also a relatively simple means for the seismic upgrading of existing structures. Both steel and concrete frame buildings can be upgraded with steel plate shear panels.

Steel plate shear walls possess properties that are fundamentally beneficial in resisting seismically induced loads. These include excellent ductility, a 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—as compared with an equivalent reinforced concrete shear wall—reduces both the gravity loads and the seismic loads transmitted to the foundation.

This paper provides an overview of several of the principal developments over the past 20 years in the analysis and design of unstiffened steel plate shear walls. The seismic performance of the steel plate shear wall concept has been further evaluated using a hypothetical multi-story building.

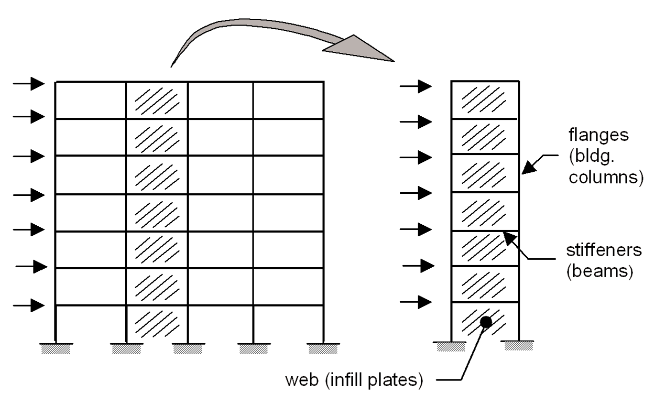

Fig. 1. Steel plate shear wall and plate girder analogy.

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BEHAVIOR OF STEEL PLATE SHEAR WALLS — CONCEPTS

Early designs for steel plate shear walls were based on the premise that out-of-plane buckling constituted the limit of usefulness of the infill panels. Because buckling of panels of the sizes contemplated for use in a steel plate shear wall would take place at low loads, either stiffeners were needed or the plate had to be relatively thick in order to fulfill this philosophy. Although it has been shown that substantial stiffening of a panel can produce an increase in the amount of energy dissipated under cyclic loading (Takahashi, Takemoto, Takeda, and Takagi, 1973), the magnitude of stiffening needed is likely to be uneconomical in most markets. (The economics of a steel plate shear wall lateral load-resisting system in highrise buildings has been discussed by Timler and Ventura, 1999). Use of a thick plate is likewise an unattractive option. Moreover, Wagner (1931) showed that buckling does not necessarily represent the limit of useful behavior and that there is considerable post-buckling strength in an unstiffened shear panel. 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 will occur at very low loads and the resistance of the panel is dominated by tension field action. The consideration of the post-buckling strength of plates has been accepted in the design of plate girder webs for many years based largely on the work of Basler (1961).

The principal difference between the model developed by Basler and others for plate girder strength and the analysis of a steel plate shear wall lies in the treatment of the plate girder flanges. In the steel plate shear wall, the "flanges" (Figure 1), which are the building columns, have considerable bending strength. This is in sharp contrast to a plate girder of usual proportions, where the flanges have little bending strength. To neglect this in the steel plate shear wall would be to give up a considerable portion of the capacity present. The role of the flanges in developing a tension field in the thin, unstiffened web is significant. In other words, the situation is much closer to design concepts used in the aerospace industry and as put forward by Wagner. Thus, the real limit state of the system, including the tension field action, can be taken into account in the steel plate shear wall.

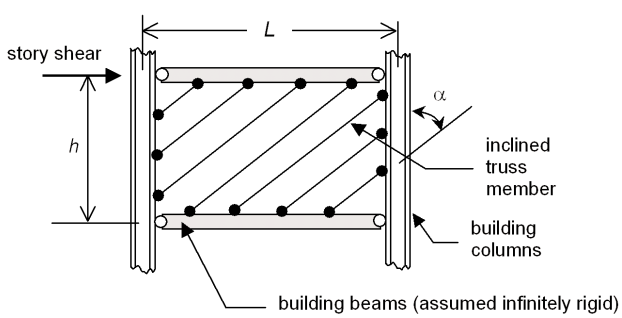


Fig. 2. Strip model representation of typical story.

ANALYSIS OF STEEL PLATE SHEAR WALLS

Treatment of the steel plate shear wall system that included tension field action was first proposed by Thorburn, Kulak, and Montgomery (1983). Those researchers suggested that the steel plate could be modeled for analysis purposes by replacing the continuous infill plate tension field with a series of inclined bars. The bars are assumed to act only in tension, and are pin-connected at their ends to the beams or columns they intercept. The concept is shown in Figure 2. The number of bars required for satisfactory modeling depends upon the panel geometry involved. However, Thorburn et al. (1983), and other researchers, indicate that about 10 strips is generally sufficient. The analysis of the sub-assembly shown in Figure 2 is then carried out using any plane frame analysis program.

The angle of inclination of the tension bars used to model the system must be established. Using an energy approach, the angle of inclination of the tension field (and therefore of the tension bars) can be calculated as (Timler and Kulak, 1983)

$$\alpha = \tan^{-4}\sqrt{\frac{1+\dfrac{t\,L}{2A_c}}{1+t\,h_s\left(\dfrac{1}{A_b}+\dfrac{h_s^3}{360\;I_c\,L}\right)}} \qquad (1)$$

where

A_b = cross-sectional area of beam
A_c = cross-sectional area of column
I_c = moment of inertia of column
h_s = story height
L = width of infill panel
t = thickness of infill panel

In the development of Equation 1 it was assumed that the beams are infinitely stiff, that is, there is no resultant bending of the beam as a result of the vertical components of the tension field action. This is a reasonable assumption because the magnitude of the tension field does not change very much between stories for all but the lowest buildings. Hence, the tension field is more or less the same on each side of a given beam. However, at the top and bottom of the shear wall, attention must be paid to the boundaries so that the tension field is properly developed. At the bottom of the building, anchorage can be made directly to the foundation. At the top of the building, two options are available. The top story can be treated in the same way as the other stories, but with a tension field angle of inclination that reflects the actual stiffness of the boundary member (the roof beam). For this case, an expression similar to that of Equation 1 is available (Timler and Kulak, 1983). Alternatively, a truss or other stiff element could be used for the full depth of the story, in which case the tension field below would be properly anchored and meet the requirements of Equation 1.

Equation 1 is for the particular case where the beam-to-column connections are pinned. However, for cases where moment connections are used, it serves as a means of approximating the tension field orientation.

The other assumption used to develop Equation 1 is that the connection between the infill plate and the boundary members (i.e., the beams and the columns) must be effectively continuous.

REVIEW OF PREVIOUS RESEARCH

Research into the behavior of stiffened steel plate shear walls started in the early 1970s. The majority of the studies, both analytical and experimental, have been done in Japan (earlier work) and in North America (later work). An extensive review of all of the literature can be found in Driver, Kulak, Kennedy, and Elwi (1997). Only a short summary of selected research is presented here.

Research at the University of Alberta

Prior to the research started at the University of Alberta in about 1980, steel plate shear walls were constructed either with heavily stiffened panels (Japan and elsewhere) or with thick panels that precluded shear buckling (US and elsewhere). It was recognized that neither of these solutions would likely be widely economical in the North American market. Moreover, it was logical that the same approach that had been used successfully in plate girder design for many years could be adapted for the steel plate shear wall assembly. This led to the research of Thorburn et al. (1983), whose study resulted in the development of the inclined bar model (Equation 1). Following that analytical work, physical testing was carried out in order to verify the approach. The test specimens were one bay wide by one story high and were about ⅔ scale. The fabrication details were reasonably consistent with what could be expected to occur in real structures.

The first experimental program, carried out by Timler and Kulak (1983), used a 5 mm (3⁄16 in.) thick infill panel and pinned beam-to-column connections. The test showed that the behavior of the frame was linear up to a load level well above that corresponding to the service load and that the frame behavior then softened gradually. Several loading excursions were applied to simulate wind loading, followed by loading to ultimate. Web out-of-plane behavior was simply the gradual amplification of shallow buckles that were present following fabrication. The Thorburn et al. (1983) analysis gave a good prediction of the actual behavior.

The excellent behavior displayed in the Timler and Kulak test was followed by a similar test (Tromposch and Kulak, 1987) used to explore the behavior of a steel plate shear wall assembly under simulated earthquake loading. This also was a single story one-bay specimen, and it also was nearly full size. The bolted web beam-to-column connections used were assumed to be pinned, and the columns were preloaded in compression. Twenty-eight fully reversed cycles of displacement were applied in order to provide quasi-seismic conditions. The hysteresis loops developed during this cycling were S-shaped and stable. The amount of energy absorbed was about comparable to conventional cross bracing used in steel frames or conventional concrete shear walls. An analytical model was developed that gave a good prediction of the hysteresis behavior. This model showed that using full moment connections between the beams and columns would result in a considerable increase in energy absorption. Once the cycling tests had been completed, the specimen was loaded to failure monotonically. This test again validated the simple bar model method of analysis.

In both of these test programs (Timler and Kulak, 1983; Tromposch and Kulak, 1987) the connection of the infill panel to the boundary members was made by welding to a fish plate, as shown in Figure 3, which provides a means for overcoming potential field fit-up problems in the real structure. This proved entirely satisfactory. It was noteworthy in both test programs that as tears started to appear in the infill plate or in the connection, their growth was always very slow and stable. The connection detail was further explored in a test program of four sub-assemblages that each had a different way of attaching the infill plate to the boundary members (Driver et al., 1997; Schumacher, Grondin, and Kulak, 1999). Two of these were the fish plate connection already described (one with a strap plate at the corner), one was the direct connection of the infill plate to the boundary members (i.e., no fish plate), and one used a chamfered corner connection with a cut-out. The latter was an attempt to reduce local stress concentrations at a location where, potentially, high stresses can be expected.

Each of the four details responded in a totally satisfactory way to quasi-seismic loading. The load vs. displacement response showed gradual and stable deterioration at the

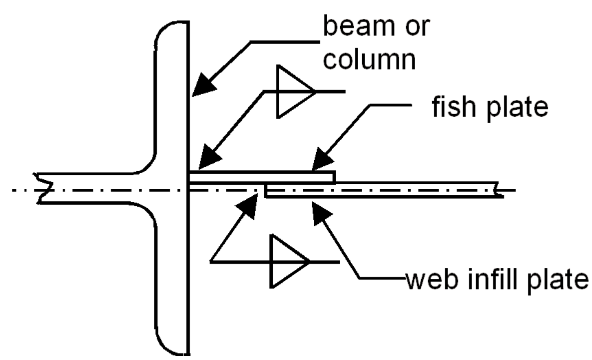


Fig. 3. Connection of infill plate to boundary member.

higher load levels. The formation of tears, which is an expected phenomenon at higher load levels, did not result in the loss of load-carrying capacity. The detail with the small corner cut-out did not behave better than the other, simpler, details.

Driver et al. (1997) showed analytically that the effect of the plate offset due to the configuration of the fish plate connection will have only a relatively small effect on the global behavior of the shear wall. Therefore, this effect was neglected in the analyses of the large-scale four-story test specimen discussed subsequently.

Even though the two major test programs (Timler and Kulak, 1983; Tromposch and Kulak, 1987) had shown that the steel plate shear wall system is reliable, gives desirable structural performance, and can be readily analyzed in a design office, it was still considered desirable to carry out a test of a multi-story frame. Furthermore, a test of a shear wall with moment-resisting beam-to-column connections was needed because this was considered likely to be the configuration most suitable for high earthquake zones. These were the bases of the tests done by Driver et al. (1997), Driver, Kulak, Kennedy, Kennedy, and Elwi (1998a), and Driver, Kulak, Elwi, and Kennedy (1998b). Because of its importance in establishing the steel plate shear wall system as a creditable option for the structural engineer, it will be described separately.

Research by Elgaaly and Caccese

An experimental program conducted by Caccese, Elgaaly, and Chen (1993) explored the behavior of six one-quarter scale steel plate shear wall models subjected to cyclic loading. 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 mm (0.03 in.) to 2.66 mm (0.10 in.).

The test specimens were loaded with a single in-plane horizontal load at the top of the shear wall. The complete loading series consisted of 24 fully reversed and gradually increasing cycles and each specimen was subjected to two of these loading series. After the second series was completed, the specimens were loaded monotonically to failure.

Arising out of the Caccese et al. (1993) tests, two analytical models were considered by Elgaaly, Caccese, and Du (1993). The first was a finite element model and the second used a perpendicular grid of truss elements oriented in the directions of the principal tensile and compressive stresses. The finite element model proved unsuccessful because of the severe demands on computing resources demanded by the fine mesh. The truss model was able to predict the ultimate strength of the test specimens, but it overestimated the stiffness. The material model was then modified to include an empirical bilinear elastic-perfectly plastic stress vs. strain relation that fitted the test data well. An empirical method was also described for predicting the hysteretic behavior of steel plate shear walls. The parameters used for defining this behavior were established from their test results. The conclusions from this research were discussed by Kulak, Kennedy, and Driver (1994) and Kennedy, Kulak, and Driver (1994), and a closure was provided by the authors (Caccese, Elgaaly, and Chen, 1994; Elgaaly, Caccese, and Du, 1994).

In a subsequent research program, Elgaaly and Lui (1997) developed a semi-empirical method for analyzing steel plate shear walls based on the strip model that uses a tension strip-gusset plate analogy to represent sequential yielding in the tension field. They also presented a method for analyzing steel plate shear walls with bolted connections between the infill plate and the fish plates that models the slippage and local plate deformation at the connection. In both cases, they report good agreement with their test results.

Research by Xue and Lu

Xue and Lu (1994) carried out an analytical study on four thin-panel steel plate shear wall configurations. In each case, a 12-story three-bay frame with moment-resisting beam-to-column connections in the exterior bays and with steel infill plates in the interior bay was used. Frames with either moment-resisting or simple beam-to-column connections in the interior bay were studied and infill plates were connected either on all four sides or only to the beams (with no connection to the columns), resulting in a total of four combinations.

The four shear wall structures were modeled using the finite element method. The structures were loaded monotonically with lateral forces at each floor level. Gravity loads were not applied.

These researchers concluded that the type of beam-to-column connection used in the bay with infill panels had only a small effect on the lateral stiffness of the entire frame. Despite the somewhat higher stiffness with the infill plates connected to both the beams and the columns, a number of factors led to the conclusion that a system with the infill panels connected only to the beams is superior. The main factor that led to this conclusion is that the analyses predicted that the columns of the stiffer system carry a proportionately larger share of the story shears, which could, in turn, lead to early failure of the columns.

(The writers consider that increased column shear is not a significant detraction: the columns can simply be designed to accommodate this increase. Moreover, if the panel is not anchored to the columns, the unanchored portion becomes ineffective, resulting in a substantial reduction in both strength and stiffness of the steel plate shear wall. Attaching the infill panel continuously to its boundary members is an inexpensive way of reducing the amount of sway, which is often of great importance in the design of a high-rise building. For a structure designed to resist earthquakes, infill panels continuously connected also provide the potential for reduced seismic damage. An incidental benefit when the infill panel is connected to both beams and columns is that the plastic hinge in the columns tends to move away from the beam-to-column junction.)

Xue and Lu also studied the effect of the width-to-thickness ratio of the panel and the panel aspect ratio on the load vs. deflection behavior of a single story, single bay steel plate shear wall with pinned beam-to-column connections and with the infill panel connected only to the beams. The researchers found that the influence of the width-to-thickness ratio on the response was small. From the results of the 20 cases studied, simplified empirical equations were presented to predict load vs. deflection response.

The approach to steel plate shear wall design of Xue and Lu, which consists of panels that have no connection to the columns and moment-resisting connections present only in adjacent bays (with no infill panels), represents a departure from the more traditional single bay configuration with the panels fully connected. Therefore, a comparative study is desirable to assess their relative merits. Issues such as the ability of the shear wall to dissipate energy, the failure mode, and relative construction costs should be addressed. Clearly, with shear panels and moment-resisting connections, both configurations have the benefit of providing an inherently redundant lateral load-resisting system.

PHYSICAL TESTING AND MODELING OF A FOUR-STORY STEEL PLATE SHEAR WALL

Driver et al. (1997; 1998a) tested a four-story assembly under quasi-seismic loading. Because this test is the most comprehensive of all the testing reported, it is described in detail. A four-story, single bay specimen, fabricated using standard details and methods, was tested to determine its behavior under idealized earthquake loading. The specimen endured 30 cycles of loading, including 20 cycles in the inelastic range. The most severely loaded panel at the base of the shear wall exhibited great ductility, reaching a deformation in the final cycle of nine times the yield deformation. The test specimen was modeled using both a commercial finite element program and by the plane frame simplified method.

The test specimen was 7.5 m (24.6 ft) high and 3.05 m (10 ft) between column centerlines and had a mass of 5.5 tonnes. A view of the specimen in the laboratory prior to installation and testing is shown in Figure 4. The expense of conducting a test on this scale dictated that only one specimen could be tested. The four-story moment-resisting steel frame had hot-rolled steel panels in each story welded all around to the boundary members. The mean measured thicknesses for the infill panels were 4.54 mm (0.179 in.), 4.65 mm (0.183 in.), 3.35 mm (0.132 in.), and 3.40 mm (0.134 in.) for Panels 1 (lowest) to 4 (uppermost), respectively. Connection of the infill panels to the surrounding beams and columns was by means of the fish plate connection depicted in Figure 3. A 100 mm (4 in.) wide by 6 mm (¼ in.) thick fish plate and continuous fillet welds were used in this test specimen.

Horizontal loads were applied to the test specimen using double-acting jacks at each of the four floor elevations. The jacks, which were connected to a common manifold, applied essentially equal loads. Gravity loads were applied to the column tops through distributing beam and gravity load simulators. The load and deflection sequences for the test were based on the method prescribed by the Applied Technology Council (1992).

The story shear versus story deformation behavior of the lowest panel (Panel 1) was used to control the test. It is of prime importance for defining the performance of the test specimen. From Figure 5 it is seen that significant ductility was exhibited by the shear wall during the test. In the final cycle (Cycle 30), the panel had reached a deformation of nine times the yield deformation. 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. Failure was by fracture of a column flange at the base of the structure. Had this failure mode been avoided, there is reason to believe that the trend of very gradual deterioration of capacity would have continued. However, since the load-carrying capacity of the test specimen had already decreased to 82 percent of the maximum, repair of the column was not attempted.

The relatively wide hysteresis loops of Figure 5 are indicative of 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

Fig. 4. Four-story steel plate shear wall (7.5m x 3.1m).

full development of the tension field. However, the appreciable stiffness of the moment-resisting boundary frame prevents the severe pinching of the hysteresis loops that is seen in shear walls with frames that have simple connections (Tromposch and Kulak, 1987).

Observed tearing of the infill plates, which is a mechanism for dissipating energy, occurred in a gradual manner: increases in tear lengths in any given cycle were only incremental. The major reason that the tearing did not result in a marked decrease in stiffness is the ability of the continuous infill plate to redistribute loads to areas unaffected by the tearing. Furthermore, the tearing was distributed relatively uniformly over the area of Panel 1, with the result that tears remained small throughout the test. The ability of the panels to redistribute load provides a redundancy in the lateral load-resisting system that is beneficial for seismic applications. The efficiency of this stress redistribution is reflected in the fact that the tears had little effect on the overall strength of the shear wall.

Finite Element Model

A finite element model of the four-story test specimen was developed using the commercial program ABAQUS. The model consists of beam elements for the beams and columns and plate/shell elements for the infill plates. As-built dimensions were used.

In all practical cases, the infill plate will have initial out-of-plane deformations and these must be taken into account in the model. Even slight initial imperfections will substantially reduce the in-plane shear stiffness. The initial configuration used in the model was based on the first buckling mode of the plate obtained from an eigenvalue buckling analysis in which the loading was applied in the same manner as for the subsequent strength analysis. Since the modal amplitude of the buckled shape is arbitrary, it was normalized to a peak value of 10 mm (0.39 in.) in order to simulate a reasonable initial condition.

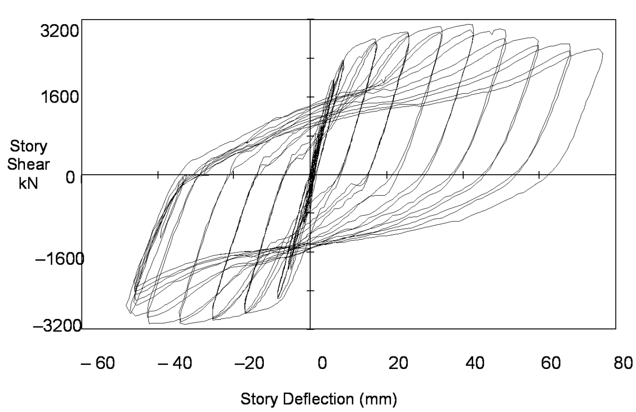


Fig. 5. Hysteresis behavior of panel 1 of four-story test specimen.

The analysis provides an excellent prediction of the peaks of the cyclic curves and, provided that nonlinear geometric effects are included in the analysis, also gives a very good estimate of the stiffness at lower load levels (Driver et al., 1998b). This model was not able to provide a good prediction of the actual response during unloading and subsequent reloading of a cycle, however. In this region, the tension field reorients itself upon reversal of the loading direction. From another ABAQUS model, a phenomenological tension–compression strip model, suitable for use with the program DRAIN-2DX, was developed. This modified model, with multi-degrees of freedom, accounts for P–Δ effects, non-linear material behavior and the spread of plasticity across the cross section. The model (although it cannot account for degrading strength due to plate tearing and local buckling) was confirmed in large measure by comparing the finite element analysis predictions for both the load and energy absorbed in the bottom panel with the Driver et al. (1998b) cyclic test results through 22 cycles of deformation.

Strip Model Analysis

The finite element method provides a powerful technique for modeling the behavior of complex structures, but the resources for conducting such an analysis are not yet universally available to designers. In the case of routine analysis of building components, designers generally require simpler methods that can be processed with available computing resources. As explained earlier, the strip model, a simplified method for analyzing thin-panel steel plate shear walls, can be employed using any commercially available plane frame analysis program. This makes the method accessible to structural design offices.

Because most plane frame programs are capable of representing elastic behavior only, a full analysis up to the ultimate strength requires a multi-step approach. As lateral load is applied to the model, any individual strip that reaches its yield strength is removed from the model and replaced by equivalent forces at each end prior to any further increase in loading. Thus, the analysis presumes a bilinear (elastic–perfectly plastic) stress versus strain relation.

Inelasticity in the moment-resisting frame must also be represented in the analysis in order to model properly the shear wall behavior. In a manner analogous to that used for the tension strips, plastic hinges in the frame are modeled by inserting a true hinge plus opposing moments on each side equal in magnitude to the moment that was present when the plastic hinge formed. However, it is recognized that the formation of a plastic hinge is actually a gradual process and varying degrees of cross-section yielding take place over a finite length of the member. (The zero-length plastic hinge tends to be non-conservative in nature but is compensated for in part by neglecting the beneficial effects of strain hardening.)

Gravity loads expected to be present in combination with the lateral loads are applied in the first load step and maintained throughout the analysis. Some method of accounting for second-order effects due to these gravity loads acting on a deformed frame is required because, in an ultimate strength analysis, deflections are significant. Most commercial plane frame programs have this feature.

The results of an ultimate strength strip model analysis of the test specimen are available in Driver et al. (1997; 1998b). The model gives a good prediction of the ultimate strength, but it tends to underestimate the initial stiffness of the shear wall slightly. This appears to be related to the small contribution of the infill panel in carrying loads in compression.

The strip model provides a relatively simple means of predicting the envelope of load versus deformation curves for a steel plate shear wall loaded inelastically and cyclically. The procedure can be conveniently performed using a personal computer and any commercial plane frame analysis computer program. Thorburn et al. (1983) also showed how the amount of input required in a multi-story stack could be reduced when beam, column, and infill plate sizes change only at intervals throughout the shear wall height. This is done by identifying the size of a single diagonal (corner-to-corner) member in a single lift that has the same effect as the multi-strip model (in which the strips do not run in the corner-to-corner direction but meet the requirements of Equation 1). When all such diagonals have been identified, the analysis of a high-rise building, say 40 stories, can be set up and analyzed very quickly to determine the stiffness and ultimate strength. The full strip model must be used to assess the forces in the individual members, however.

More information on both the strip model and the finite element model can be found in Driver et al. (1997; 1998b).

SEISMIC PERFORMANCE

The eight-story building shown in Figure 6, as adapted from Chien (1987), is used to evaluate the seismic performance of steel plate shear walls. The building is located in Vancouver, Canada. In this study, only the steel plate shear walls in the east-west direction are designed and analyzed.

The gravity load on each column of the steel plate shear wall consists of a dead load of 262 kN (59 kips) at each level and a live load of 60 kN (13 kips) at each floor and 102 kN (23 kips) at the roof. The beams spanning east–west have uniformly distributed dead and live loads of 16.5 kN/m (1.13 k/f) and 10.8 kN/m (0.74 k/f), respectively. The snow load on the roof is 1.66 kPa (35 psf). The 1 in 10 year and 1 in 30 year hourly wind pressures are 0.36 kPa (7.5 psf) and 0.44 kPa (9.2 psf), respectively.

The base shear specified by the National Building Code of Canada (NBCC) (CCBFC, 1995) is

$$V = \frac{vSIFW}{R} U \tag{2}$$

where v is the zonal velocity ratio (equal to 0.2 for Vancouver), S, the seismic response factor, is a function of the seismic zone and the building period, I is the importance factor (taken as 1.0), F is the foundation factor (taken as 1.0 for buildings founded on rock), W is the effective seismic weight (estimated at 54 400 kN, with a load of 7 240 kN at the roof including 25 percent snow and 6 740 kN at each floor), R, the force modification factor, is taken as 4.0 for a ductile steel plate shear wall, and U is the NBCC calibration factor of 0.6. The NBCC estimate of the fundamental period, T, of a steel plate shear wall that is required to define the seismic response factor, S, is

$$T = 0.09\, h_n / \sqrt{D_s} \tag{3}$$

where h_n is the height (29.7 m) and D_s is the width (8.0 m) of the steel plate shear wall. Thus, the design period is 0.945 s in the east-west direction and the S factor, equal to $1.5/\sqrt{T}$, is 1.54. Equation 2 thus gives a base shear of 2 520 kN (567 kips) for the complete building. A portion of the base shear, F_T, equal to $0.07TV$, is applied at the roof level and the remainder is distributed among all levels by

$$F_x = (V - F_T)\left(W_x h_x \Big/ \sum_{i=1}^{n} W_i h_i \right) \tag{4}$$

where W_x is the effective weight and h_x is the height to level x. One half the lateral force is applied on each steel plate shear wall. In implementing the minimum accidental torsion provision of the NBCC, the steel plate shear walls in

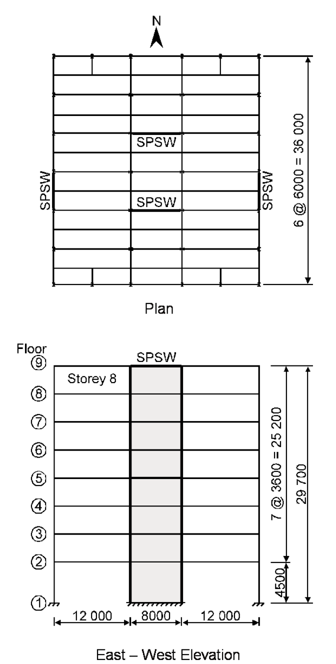


Fig. 6. Layout of eight-story building.

the north-south and east-west directions are assumed to have equal lateral stiffness. Torsion increases the base shear for the east-west steel plate shear wall from 1260 kN (283 kips) to 1350 kN (303 kips).

The preliminary design of a single strut idealization of the steel plate shear wall (Thorburn et al., 1983) is carried out using the linear structural analysis and design program SODA (Acronym Software, 1994). The beams and columns are grade 350W steel and the infill plates are grade 300W steel (yield strengths of 350 MPa (50 ksi) and 300 MPa (44 ksi), respectively). The columns are provided in two-story lifts. The steel plate shear walls have full moment beam-to-column connections, while all others in the structure are simple and relatively inexpensive shear connections. The steel plate shear wall is fixed to the foundation. The NBCC limit on the interstory drift under the 1 in 10 year hourly wind is 1/500 of the story height, h. Under the earthquake force, the interstory drift from an elastic analysis, when amplified by the force modification factor, R, must not exceed $0.02h$. The load combination of $1.0D + 0.5L + 1.0E$, where D, L, and E are the specified dead load, live load (including snow), and earthquake load, respectively, governs the design. Figure 7(a) shows the preliminary selection of members for strength. With these members, the drift limits for wind and seismic action are also satisfied. Thorburn et al. (1983) relate the thickness of the infill plate, t, to the cross-sectional area, A_{br}, of the diagonal strut by

$$A_{br} = tL \tan\theta / (2\sin 4\theta) \tag{5}$$

where

$\theta = \frac{1}{2} \tan^{-1}(L/h)$

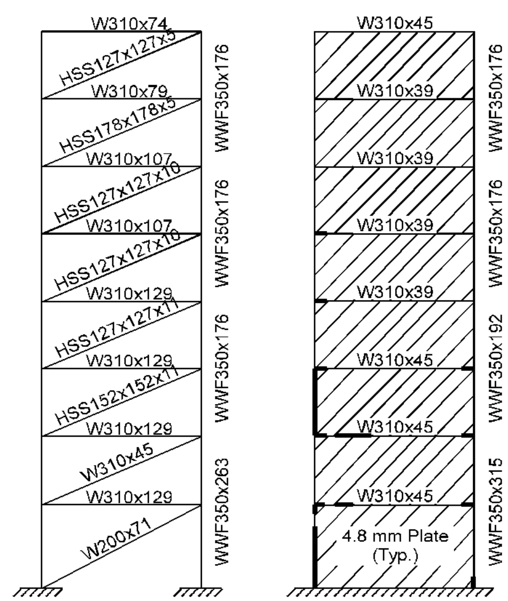
L = panel width

h = panel height

This gives a panel thickness ranging from 3.33 mm (0.13 in.) for story 1 to 0.66 mm (0.026 mm) for story 8. Based on thicknesses that are easily handled, an infill panel thickness of 4.8 mm (0.19 in.) is provided throughout. With revised areas of the diagonal struts, the preliminary design is re-analyzed to estimate the P-Δ effect under lateral drifts. In accordance with the NBCC (CCBFC, 1995), these are amplified by the force modification factor. The amplified P-Δ effect increases the design base shear by 1.22 times from 1350 kN (303 kips) to 1650 kN (371 kips). The preliminary design is found to be adequate.

Detailed design is then carried out using the tension-only strip model shown in Figure 7(b). (The heavy lines indicate the extent of yielding under the most severe earthquake, to be discussed subsequently). Using Equation 1, the angle of inclination of the tension field ranges from 40° to 43°, and an average value of 42° is used for all panels. Each panel is discretized into 10 pin-ended strips. The columns, as part of a ductile moment-resisting frame—described in CSA Standard S16.1 (CSA, 1994)—are constrained to be Class 1 (plastic design) sections and are considered to be braced laterally at the floors. The beams are constrained to be Class 1 or 2 sections, and are taken to be continuously braced laterally by the floor diaphragm. The design process is repeated on SODA with the infill panel thickness held constant at 4.8 mm. Revised beam and column sections are shown in Figure 7(b). The angle of inclination of the tension field is now found to be 36° in panel one and 38° in the remainder. (Although the roof beam satisfies the strength and stiffness requirements of the CSA Standard, it does not provide the "infinite stiffness" implied by Equation 1.) This strength design also satisfies the NBCC drift limits for wind and seismic action as shown in Figure 8. The mass of steel in each steel plate shear wall is about 23.5 tonnes.

A free vibration analysis of the tension-only strip model gives a fundamental period of 1.65 s. In addition to the linear static analysis used in design, a response spectrum analysis is also carried out, as suggested but not required in the NBCC, to estimate the effect of the higher modes of vibration on the distribution of lateral forces over the building height. The interstory drifts obtained from the response spectrum analysis are first scaled by the ratio of the base shear from the NBCC to that from the response spectrum analysis and then amplified by the force modification factor, R. These drifts, and similarly amplified drifts due to the NBCC prescribed loading, normalized by dividing by the story height, are plotted in Figure 8. The drift ratios due to wind range from 0.50 to 0.70 of the NBCC limits and those due to NBCC seismic loads from 0.49 to 0.88 of the NBCC limits. However, the truly significant drifts due to seismic action are those determined from the time history analyses as discussed subsequently. The distribution of lateral force



(a) single strut idealization and preliminary design

(b) tension strip model and detailed design

Fig. 7. Design of the example building.

by the NBCC results in slightly greater interstory drifts than does the response spectrum analysis except for the top two stories, and is used in the nonlinear static pushover analysis. Although the interstory drift from the response spectrum analysis exceeds the NBCC limit in story 8, the structure was not stiffened because the tension-only strip model neglects any compressive resistance of the infill panels.

The inelastic static and dynamic response of the steel plate shear wall is assessed with a tension-compression strip model. This is an extension of the tension-only strip model and has inclined strips in both directions to resist lateral load in either direction. A nonlinear static pushover analysis of this model is carried out with the program DRAIN-2DX (Prakash, Powell, and Campbell, 1993). The columns and beams are modeled with the fiber element of DRAIN-2DX, which accounts rationally for axial load–bending moment interaction on the cross-section. The model does not account for out-of-plane behavior. The import of this is discussed subsequently. The infill strips are modeled with inelastic truss elements that may yield in tension and buckle elastically in compression. The compressive capacity of the strips is taken as 0.08 of the tensile capacity. This fraction was determined by calibrating the tension-compression strip model for both the load sustained and the cumulative energy absorbed with experimental hysteresis loops (Driver et al., 1997) through 22 cycles of deformation. The material behavior is taken to be trilinear with linear elastic behavior to yield, strain hardening to the tensile strength of 1.4 times the yield strength at a strain of 0.15, and a horizontal plateau thereafter. The P–Δ effect is included by adding a stack of dummy columns to the model and applying appropriate gravity loads on them. The NBCC prescribed lateral force pattern is applied to the model and is increased until the lateral displacement of the roof equals 594 mm (23.4 in.), which is the NBCC drift limit of 0.02 of the building height.

The base shear versus roof displacement response of the steel plate shear wall, shown in Figure 9, indicates that it has an overstrength of about two times with respect to the NBCC prescribed base shear. This overstrength results from using minimum infill plate thickness of 4.8 mm (0.19 in.), which is considerably greater than that required. The columns of the steel plate shear wall yield in compression before any yielding takes place in the infill panels. Logically, they could be strengthened so that the infill panels yield first. However, this was not done because the overstrength provided to the steel plate shear wall would likely reduce the inelastic demand on the columns, as discussed subsequently.

Nonlinear dynamic time history analyses of the tension-compression strip model are also carried out with DRAIN-2DX. The floor mass is lumped at each column. Rayleigh damping is used with the coefficients chosen to give 5 percent damping in the first and eighth mode. The steel plate shear wall is subject to an ensemble of 20 scaled accelerograms selected for use in Vancouver, Canada (Medhekar and Kennedy, 1997). The dynamic response is obtained with a time step of 0.001 s or 0.0005 s. The factored gravity load is applied to the model prior to dynamic analysis. The P-Δ effect is taken into account by using a geometric stiffness based on the gravity load on the columns.

Figure 10 shows the interstory drift ratios obtained from the time history analyses at the maximum and the mean-plus-one-standard deviation (mean + 1 sigma) level of response for the 20 scaled earthquakes. (The interstory drift ratios are the respective statistical values for each story and do not occur simultaneously.) The mean + 1 sigma level is considered to be the more significant statistically. These drift ratios of 0.0040 to 0.0059 are only 0.20 to 0.30 of the NBCC provisions and should not cause appreciable damage to gypsum board partitions. The interstory drifts from dynamic analyses are also significantly less than the amplified seismic static interstory drifts. The latter are obtained by multiplying the elastic interstory drifts (from the tension-only strip model subjected to the NBCC prescribed

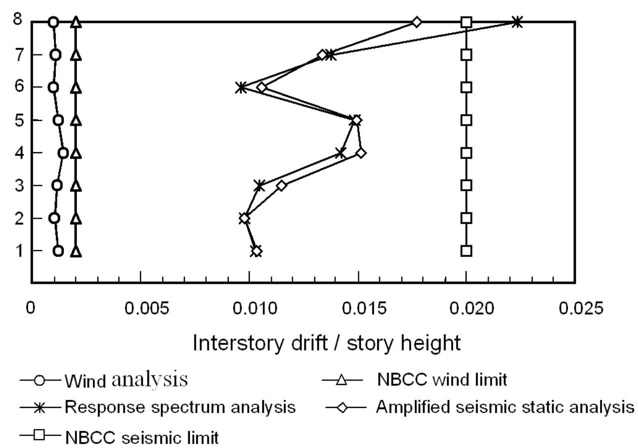


Fig. 8. Elastic interstory drift ratios and code limits.

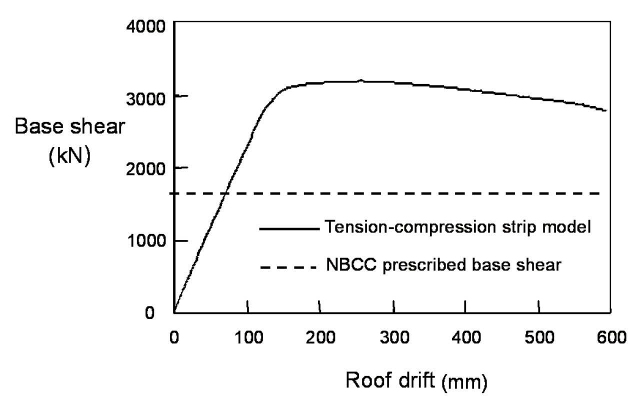


Fig. 9. Pushover analysis of example building.

base shear) by the force modification factor, R. Thus, the steel plate shear wall provides excellent control of interstory drift and thereby protects both the structural and the non-structural elements from damage. The mean +1 sigma drift ratios for the lower two stories of less than 0.0046 are less than those of the remaining stories. For stories three to eight, there is little variation in the drift ratios, with a mean value of 0.0056 and a coefficient of variation of 0.052. From the time history analyses for 20 earthquakes, the maximum story shears developed in the eight stories range from 2.07 to 2.97 times the prescribed NBCC values, with a mean value of 2.46. Thus all stories have withstood shears much in excess of the NBCC values. These data, in conjunction with the limited drifts, indicate that a weak story does not manifest itself at these load levels.

Furthermore, because these drifts are limited, inelastic straining in the beams and columns is also restricted. From the time history analyses, the strains in the columns and beams were examined for the particular earthquake that caused the maximum interstory drifts. Even for this extreme condition, significant yielding occurred in only one of the columns and only in stories one and three, with a maximum computed stress of 1.03 times the yield value. One or both ends of floor beams at floors 2, 3, 4, 5, and 6 also yielded, with a maximum computed stress of 1.06 times the yield level. Thus even for this most severe earthquake, in which the building was subjected to 2.45 times the NBCC shears, the inelastic straining is limited. Inelastic straining in the framing members was not investigated for the other earthquakes. The infill panels were found not to yield in any of the 20 earthquakes. (The ductility demand on the infill panel strips cannot be deduced from the interstory drifts based solely on a simple tension-strip-yielding model because of the accompanying column shortening and the elastic compressive hysteretic behavior used for the strips.) Members that yield are shown in Figure 7(b) by heavy lines.

Engineers involved in seismic design have traditionally avoided yielding in columns, especially when it involves the development of plastic hinges. A distributed plastic hinge

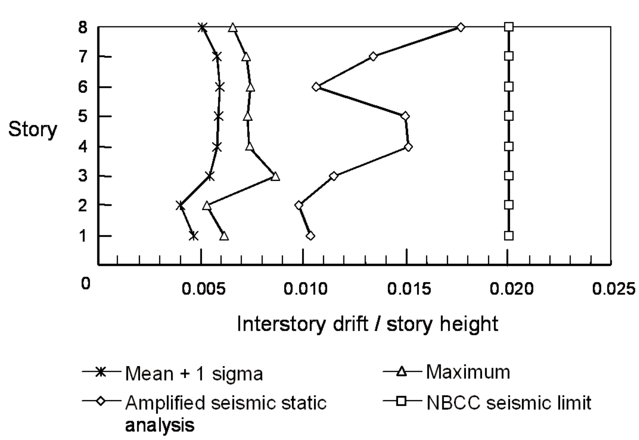


Fig. 10. Interstory drift ratios from time-history analysis.

did develop in the most severe earthquake considered, as deduced from the strain records, at only one location (at the bottom of one column in story 3). It is considered that this condition did not progress to the development of more hinges and collapse because the lateral deformation of the story was restrained by the elastic tension field, the other component of the ductile dual system. In the Driver et al..(1998a) test, with the Class 1 columns loaded to about 1.13 times the yield load in compression, the dual system still maintained 82 percent of the maximum story shear at a ductility ratio of $9\Delta/\Delta_Y$.

The seismic base shear and deflection limits for this building were also determined for Seattle, WA, a city in close proximity to Vancouver, Canada. The Uniform Building Code (UBC) provisions were followed for Seattle, which is in Zone 3 (Uniform Building Code, 1997). The R factor for loads is taken as 8.5 for a dual system and as 6 for amplifying the interstory drifts. The design base shear per shear wall is found to be 1 420 kN (319 kips), as compared to 1 650 kN (371 kips) for Vancouver. The maximum elastic interstory drift, when amplified six times for inelastic behavior, exceeds the allowable UBC value of $0.02h$ only in the top story and then by a factor of only 1.15. Thus the same building, or one with slight modifications, meets the UBC provisions for Seattle.

The example steel plate shear wall building therefore provides about twice the strength necessary to withstand the prescribed factored earthquake loading of the NBCC and, at the same time, provides a relatively stiff system that limits both the non-structural and structural damage. The system provides excellent performance with limited interstory drifts at lateral loads greatly exceeding code requirements. It is suggested that, with this system, the engineer does not have to choose between a structure that stands up, but deforms significantly and is severely damaged, and one that is too expensive to build.

Any building, if loaded sufficiently, will develop a failure mechanism. How does this show up eventually in the example steel plate shear wall building designed for Vancouver? From Figure 11, where the base shear is plotted versus the interstory drifts from the pushover analysis, it is apparent that most of the inelastic action occurs in story 3. This is confirmed by the location of the yielding obtained from the time history analyses for the most severe earthquake, as shown in Figure 7(b).

Figure 12 shows the normalized shear versus the ductility ratio for story 3 from the pushover analysis. The normalized shear is the story shear divided by the NBCC shear and the ductility ratio is the interstory drift divided by the drift at yield. The drift at yield (16.6 mm) is obtained assuming the behavior to be elastic up to the maximum shear level. A ductility ratio, $9\Delta/\Delta_Y$, greater than the value of 10 shown was obtained with the structure still carrying more than the NBCC shear for this story. At a ductility ratio of 7, the shear carried is still 1.73 times the NBCC value. The shear in the story is significantly greater than the NBCC value through a very considerable drift. The story is therefore robust. Thus, the structure exhibits considerable robustness in the story that is deforming. This behavior was demonstrated physically in the Driver et al. (1998a) test, where a ductility ratio of 9 was attained. From the time history analyses, the peak interstory drift of 31 mm (1.2 in.) occurs in story 3 in the most severe earthquake. This corresponds to a ductility demand of 1.87. The corresponding maximum extreme fiber strain in the column is 0.0143. Class 1 sections can maintain the full plastic moment capacity (in the presence of axial load) beyond this strain, say, 15 ε_Y or 0.026. [In the Driver et al. (1998a) test, local buckling started at a ductility ratio of about 4 in the most heavily loaded column.]

Figure 11 shows that the drift reached in story 1 is only a fraction of that in story 3. That this is not of concern is demonstrated by increasing the column size in story 3 (and 4) to a WWF 350 × 315 in order to force the inelastic action to occur in story 1. Figure 13 shows interstory drifts from a pushover analysis for this modified structure. The drift, now concentrated in story 1, reaches a value of 150 mm (5.9 in.), which is a ductility ratio of 7.3. As well, a time history analysis was carried out for the most severe earthquake with double the amplitude of the ground motion. This leads to an interstory drift of 100 mm in story 1. The remaining stories each drift between 38 (1.5 in.) and 44 mm (1.7 in.). The story shears developed in the structure range from 2.62 to 3.29 of the NBCC values. This pattern of interstory drifts is consistent with that from the pushover analysis presented in Figure 13. Under this doubly severe loading, yielding was much more extensive. The lower three infill panels, the bottom two stories of columns, and the beams at levels 1 to 6 all yielded to some extent. However, portions of the infill panels remained elastic, thereby providing lateral restraint. Thus, by strengthening the columns in story 3, the robust behavior now occurs in story 1.

The dual system of steel plate shear walls combined with ductile moment-resisting frames therefore provides a strong, stiff, and robust system. To get this excellent in-plane behavior, out-of-plane buckling of the columns must be prevented, of course. As mentioned previously, the Class 1 W-shape columns in the Driver et al. (1998a) test, with a weak-axis slenderness ratio of about 25 were loaded beyond the yield load and did not buckle laterally.

SUMMARY AND CONCLUSIONS

A steel plate shear wall comprises two adjacent continuous column stacks extending the full height of the building, joined together at each floor by the floor beams, and with steel plate infill panels in all stories, generally fastened continuously to the columns and beams. The wall is fixed at its base. A steel plate shear wall is therefore analogous to a vertical cantilever plate girder in which the columns act as the flanges, the floor beams act as the transverse stiffeners, and the infill panels act as the web. A minimum of one pair of shear walls in each direction provides the lateral load resisting system for the building.

A comprehensive series of conjugate analytical and large-scale experimental studies over a period of nearly 20 years has led to the shear wall configuration presented here. The thin infill panels develop lateral shear resistance through tension field action. When the beam-to-column connections are full moment connections, as proposed, the result is a desirable dual system of the moment frame and the steel plate shear wall. An important feature is that the infill panels, by tension field action, provide a distributed brace rather than a discrete one.

Studies carried out include: the determination of the angle of inclination of the tension field as a function of the wall geometry; the development of a simple tension-only strip model for design; a large scale test of single story panels under simulated wind loading; a large scale test of single story panels under simulated seismic loading; tests on four different details for connecting the infill panels to the framing elements; a large scale simulated seismic test on a four-story wall; and several inelastic finite element studies

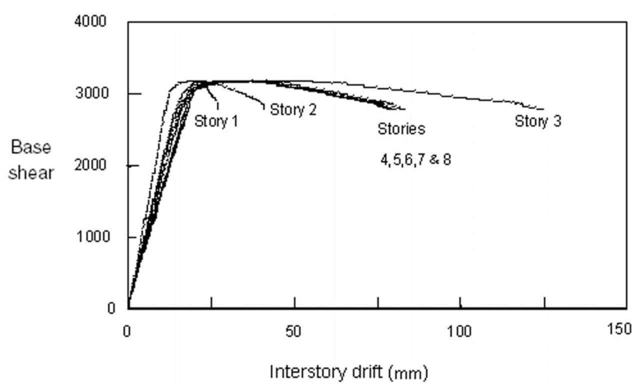


Fig. 11. Base shear vs. interstory drift from pushover analysis.

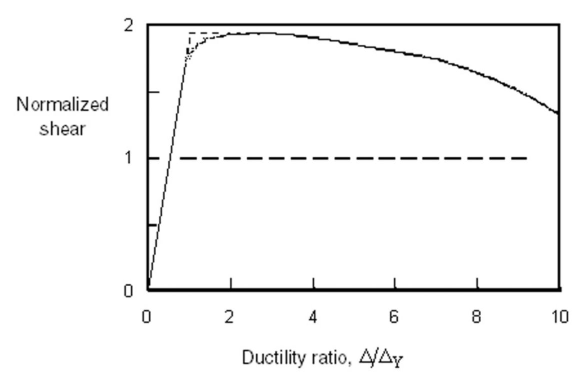


Fig. 12. Normalized shear vs. ductility ratio for story 3.

to develop models to mimic the cyclic load vs. deformation response, including the energy absorption characteristics of the wall. Physical tests are invariably taken to failure.

Connection of the thin infill panels, likely to be carried out in the field, is facilitated by fillet welding to fish plates that are welded to the beams and columns in the shop. Some fabricators may prefer to provide fish plates on two boundaries only and directly weld the infill plate to one column and the top of the lower beam. Either detail gives satisfactory performance. Because the infill plates are very efficient in tension field action, quite thin plates are usually sufficient. Frequently, a plate thickness selected for ease in handling will be more than adequate. During severe cyclic loading or as the ultimate load of the shear wall is approached, tears will develop in the infill plates. Because of the continuous nature of the plates, they still carry the transverse shear, however. In a large scale, four-story test frame that was subjected to 30 cycles of loading to a maximum story ductility ratio, Δ/Δ_Y, of 9, the shear strength was still 82 percent of the maximum value at cycle 22.

The best hysteretic performance is obtained by the combination of the steel plate shear wall with a ductile moment-resisting frame. A phenomenological model, suitable for use with DRAIN-2DX in carrying out time history analyses, has been developed. It mimics well the load vs. deformation response as observed in tests, and is capable of predicting the maximum load observed in cyclic testing and the cumulative area under the hysteresis loops.

The model consists of an eight-story building located in Vancouver, Canada, and designed in accordance with CSA Standard S16.1 and the National Building Code of Canada (NBCC). The tension-only strip model, developed from the extensive studies reported here, is used for design and analysis of the steel plate shear wall. The infill panel thickness of 4.8 mm, selected for ease in handling, exceeded the seismic design requirements throughout the height. A free vibration analysis and a response spectrum analysis are carried out with a tension-only strip model that models the elastic stiffness of the steel plate shear wall. However, a verified phenomenological model reflecting the nonlinear characteristics of the steel plate shear wall is used for nonlinear static pushover analysis and nonlinear dynamic time history analyses of the building subjected to 20 scaled earthquakes.

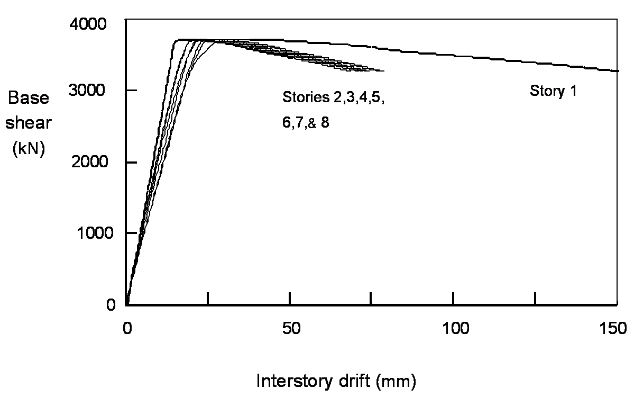


Fig. 13. Pushover analysis with strengthened columns.

The dual system of the steel plate shear wall and ductile moment-resisting frame provides excellent structural performance. The structure easily satisfies the NBCC limits for wind and seismic action: the nonlinear static pushover analysis suggests that the structure can carry twice the NBCC prescribed base shear. This overstrength is largely the result of using an infill plate thickness of 4.8 mm (0.19 in.), which is greater than that required. The base shear is significantly greater than the NBCC value through a drift of 0.02 times the building height. Although most of the inelastic action occurs in a column at story 3, a story ductility ratio, Δ/Δ_Y, of more than 10 is attained. This behavior was shown physically in the test of a four-story shear wall done by Driver et al. (1998a). Moreover, to demonstrate that robust behavior can be attained in any story, by strengthening story 3 the robust behavior was exhibited in story 1. In fact, this modified structure was capable of withstanding the most severe earthquake considered with double the amplitude of the ground motion. The infill panels limit the inelastic straining in the columns and beams.

The maximum interstory drift ratio in any story for the ensemble of 20 earthquakes considered does not exceed about 0.009. Thus, the framing system protects both structural and non-structural elements from damage. The maximum story ductility demand of 1.9 Δ_Y in the most severe earthquake is only about one-fifth of that obtained from the pushover analysis. This indicates that a large reserve in energy dissipation capacity exists. Furthermore, for this extreme condition, significant yielding occurred in only one of the columns in story 1 and 3, where the maximum computed stress is 1.03 times the yield value. The maximum extreme fibre strain in the columns was less than that which a Class 1 (plastic design) section can sustain without buckling locally. Because of the limited plastification in the columns and the restraint provided by the elastic infill panel, a soft story did not develop.

An examination of the Uniform Building Code seismic base shear and drift limits for Seattle, WA indicates that the example building would meet the requirements for that city as well. The dual system of steel plate shear walls combined with ductile moment-resisting frames therefore provides a strong, stiff, and robust system.

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