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Performance of Single- and Multi-Storey Steel Plate Shear Walls Under Simulated Seismic Loading

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ABSTRACT

The use of the post-buckling strength of steel infill shear panels in a boundary frame has been proposed as a viable lateral load resisting system for areas of high seismic risk. Previous research has concentrated on the monotonic and slow cyclic response of steel shear wall frames and the development of simplified design code provisions. This paper presents the results of an experimental test program conducted at the University of British Columbia to further define the seismic performance characteristics of multi-storey steel plate shear wall assemblies. Two single storey and one four-storey specimens were tested under slow cyclic loading. A similar four-storey specimen was tested on a shaking table using simulated and historical earthquake input base motions. Stable hysteresis loops with consistent elastic behaviour and onset of yield properties were obtained for all the tests. In each case, significant inelastic action was observed at the column bases. The results showed that reverse cyclic loading can produce a good approximation to the overall dynamic performance of a steel shear wall assembly, but will not identify some serviceability issues such as infill panel vibration effects.

INTRODUCTION

The use of steel plate shear walls as a primary lateral load resisting system in medium- and high-rise buildings largely remains a challenge for structural engineers and has been used in only a few cases so far. A steel plate shear wall frame can be idealized as a vertical cantilever plate girder, in which the steel plates act as the web, the columns act as the flanges and the storey beams represent the transverse stiffeners. The theory that governs the design of steel plate shear wall structures is essentially the same as that of plate girders developed by Basler in 1961, although the relatively high bending strength and stiffness of the beams and columns are expected to have a significant effect on the overall behaviour for the application in buildings.

To verify the guidelines and design principles provided in the latest version of Canada's national standard on Limit States Design of Steel Structures, CAN/CSA-S16.1-94 (1994), and to broaden the scope of the code provisions, a collaborative study was initiated between researchers at the Universities of British Columbia and Alberta to investigate the performance of steel plate shear walls as the primary lateral load resisting system for medium- and high-rise buildings located in regions of high seismic risk. The objective of the UBC research program was to investigate the behaviour of steel plate infill panels and boundary frames under simulated earthquake motions and slow quasi-static cyclic loading. Slow cyclic quasi-static tests of two single storey steel plate shear wall specimens and one four-storey specimen (Lubell, 1997) were carried out prior to a more realistic shake table testing of a four-storey specimen (Rezai, 1999).

The design of the test specimens was primarily based on the limitations of the shake table facilities. The main restrictions of the UBC shake table facilities are: clearance above the table, maximum weight allowed on the table and the stalling force capacity of the actuators that produce horizontal longitudinal motions in either direction. The effects of shear panel aspect ratio, panel thickness, stiffness of the surrounding frame (including moment-resisting and shear-only connections), number of storeys, and storey masses on the overall behaviour of a scaled steel plate shear wall specimen were examined. After a number of computer simulations, a four-storey steel plate shear wall model with panel aspect ratio of 1.0 and length scale factor of 1/4 was found to be suitable for experimental investigations. This was based on the base shear limitations of the shake table actuator to establish the onset of yield at the base of the columns or within the infill plates. The specimen was comprised of S75×8 columns, continuous through the height of the frame, S75×8 beams for the bottom three storeys and a deep stiff S200×34 beam at the top to anchor the tension field forces generated in the upper storey plate. At the bottom panel of the shear wall, the tension field was anchored internally by providing a S75×8 beam. Beam-to-column joints were fully welded with web stiffeners in the columns opposite the beam flanges, thus creating full moment connections. The 900 mm centre-to-centre column spacing and floor-to-floor height, the 1.5 mm thick infill steel panels, the number of floors, and the specified beam and column cross-sections were selected such that the test objectives could be achieved using available resources. Because the material properties of the infill plate for the test specimens are extremely important to ensure applicability of the final test results, the thinnest available commercial quality, hot-rolled steel plate was selected. Figure 1 shows the four-storey steel plate shear wall specimen used for the quasi-static and shake table studies.

QUASI-STATIC TESTS

The overall strength, elastic post-buckling stiffness, interaction between frame action and shear panel behaviour, effects of beam and column rigidities, the formation of diagonal tension field action combined with diagonal compression buckling of the infill plate and the stability of panel hysteresis curves were the main issues investigated during quasi-static testing.

To better understand the load-deflection behaviour of the four-storey steel plate shear wall specimen and to examine the effects of weld distortion on the behaviour of thin infill panels and the surrounding frame, two single panel specimens representative of the bottom storey panel of the four-storey specimens were constructed and tested. Two different fabrication configurations for the single storey steel plate shear wall specimens were attempted; a flexible and a stiff top beam. Also, the initial out-of-plane deviation of infill plates, as a result of heat distortion, was substantially smaller for the second single storey specimen compared to the first single storey specimen.

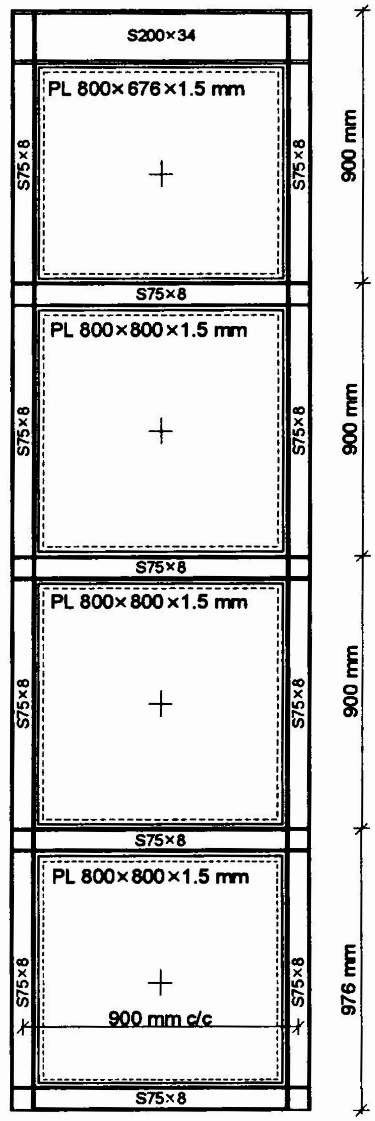

Figure 1: Four-storey steel plate shear wall specimen

The specimens were subjected to a horizontal in-plane load history with gradually increasing cycles until global yielding was observed. Thereafter, displacement amplitudes were increased incrementally by multiples of the yield displacement $\pm\delta_y$. Figure 2 shows the load-deformation behaviour of the first and second single storey specimens under slow cyclic loading. As the storey displacement reached $2 \times \delta_y$, yielding of the various components of the shear wall resulted in a significant decrease in the stiffness of the specimen in the post-yield region. Once the load was removed, the unloading curve showed a very small elastic stiffness decay for the panel. As the load reversed to the opposite (negative) direction, the stiffness of the steel plate shear wall specimen reduced substantially to approximately 30% of the initial elastic stiffness. The loss of stiffness was attributed to the permanent stretching of the plates from previous load cycles. The redevelopment of the tension field increased the stiffness until the load approached the yield strength achieved in the negative direction. For the first single storey specimen, one cycle of load-deformation was conducted at each post-yield deformation step until a displacement of $4 \times \delta_y$ was achieved. Before finishing the first cycle of $4 \times \delta_y$, one complete cycle of ±100 kN lateral load was applied and then the load was increased to reach the top displacement of $4 \times \delta_y$. This was done to examine the behaviour of the specimen to a low level input excitation after going through a few cycles of high load intensities. Beyond this point, the specimen was subjected to a pushover loading to failure up to a maximum displacement of $7 \times \delta_y$. The test was stopped at this point due to excessive twist and deformation of one of the out-of-plane support arms at its far end connection. For the second single storey specimen the ATC-24 (1992) loading history recommendations were adopted. Each loading cycle was repeated three times for each displacement ductility level up to a maximum displacement of $6 \times \delta_y$ when a column fracture occurred.

The overall shapes of the hysteresis curves indicate that the behaviour of the small-scale single storey steel plate shear wall specimens under cyclic loading was very robust and stable. The degradation of strength at high ductility levels was primarily related to severe plate tearing and weld cracking. Even though the specimens experienced ductility demands of seven and six times the yield displacement, no sudden loss in stiffness or strength occurred in the specimens. In any case, current earthquake engineering practice (NBCC, 1995) limits the ductility demand of building structures to storey deflections of lesser magnitude than applied to the specimens.

Both initial and post-yield stiffnesses of the second single storey specimen were considerably higher than the initial and post-yield stiffnesses of the first single storey specimen. The yield displacement of the second single storey test was recorded as 6 mm compared to about 8.5 mm for the first single storey test. The corresponding yield loads were 180 kN and 190 kN for the first and second single storey specimens, respectively. The maximum storey shear achieved in the second single storey test was 260 kN for a displacement ductility level of $5 \times \delta_y$ as opposed to 200 kN for the first single storey test at a displacement ductility of $4 \times \delta_y$. The significant improvement in the overall load-deformation behaviour of the second single storey specimen was primarily attributed to the stiffer storey beam and the lessening of initial out-of-plane imperfections.

After testing two single storey steel plate shear wall specimens, one four-storey specimen with storey dimensions and beam and column sizes similar to the first single storey specimen was constructed and tested under quasi-static cyclic loading. The specimen proved to be somewhat more flexible than the one-storey specimens tested before. This was expected because the influence of overturning moment becomes more significant as the height of a structure increases. The yield deflection in the first storey panel was determined as 9 mm at a storey shear of about 150 kN. After three cycles of ±150 kN the load was

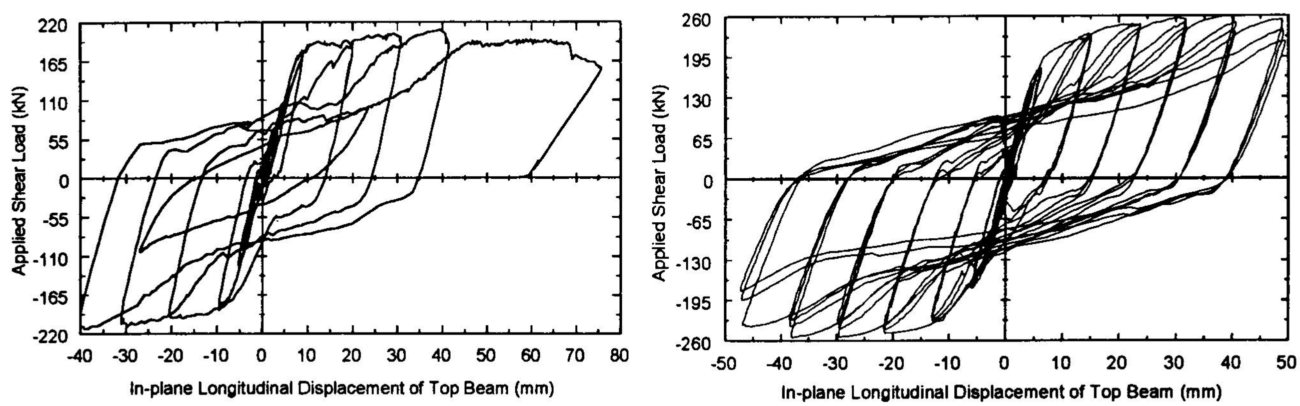

Figure 2: Hysteresis curves of the first (left) and second (right) single storey shear wall specimens

increased to a first storey deflection of about 15 mm. As the load reversed, the unloading stiffness was parallel to the initial elastic stiffness. Increasing the load to the opposite (negative) direction caused some pinching in the hysteresis curve with a moderate reduction in the stiffness. This was also observed in the previous tests because the first storey infill plate had stretched and buckled inelastically during the previous loading direction. It would thus not be fully effective in the reversed loading direction until the load increased enough to compensate for the effects of Poisson's ratio. Once the tension field started to form in the web plate, the stiffness increased. The load was brought up to the negative yield strength of the specimen and increased by a manually adjusted control valve. As the first storey deflection reached -15 mm, the compression column buckled in the out-of-plane direction. The test was terminated at this point.

Even though a premature failure mechanism occurred when the specimen was experiencing inelastic deformations, useful data regarding initial elastic stiffness, post-yield stiffness, storey drifts, overall deformed column profile, variation of principal tensile stresses in the plate and the interaction of the boundary frame with the steel panels was collected from the test. The first storey panel showed stable hysteresis curves, thereby dissipating a significant portion of the input energy through column yielding and infill panel diagonal buckling and yielding. As no evidence of yielding was observed above the first storey level, the load-deformation hysteresis loops of the upper storey floors did not reflect the significant contribution of these floors to the overall energy dissipation mechanism. The load-deformation hysteresis loops of the upper stories reflected a rigid body rotation due to the column shortening or plastic hinging in the first storey. This observation signifies an important attribute of a tall multi-storey steel plate shear wall frame in which the bottom storey panel attracts a major portion of the input energy. The demand on the column bases were shown to be extremely high. It is, therefore, of utmost importance that in a steel plate shear wall structure, the base of the column be detailed carefully to ensure a robust ductile behaviour.

SHAKE TABLE TEST

Shaking table tests are increasingly being used to evaluate the seismic performance of structures. The newly upgraded shake table facility in the Earthquake Engineering Laboratory at UBC provided a unique opportunity to conduct dynamic testing of a scaled multi-storey steel plate shear wall specimen. While there have been a limited amount of static and quasi-static tests performed on unstiffened steel plate shear wall frames, no information is available on the behaviour of these structural systems under earthquake loading.

The steel plate shear wall specimen was subjected to the E-W component of the Tarzana Cedar Hill Nursery Station record from the 1994 Northridge earthquake, the E-W component of the Joshua Tree Fire Station record from the 1992 Landers earthquake, the N-S component of the Petrolia Station record from the 1992 Petrolia earthquake and a synthetically generated acceleration time-history waveform from Bell Communications Research (Bellcore, 1995), the so called "VERTEQII" record, that had been synthesized from several typical earthquakes and for different building and soil site conditions.

The specimen was subjected to the Tarzana Hill record with 20% and 40% intensities. While running the tests, vibration of the infill plates was very visible and accompanied by an audible noise. The reorientation of infill plate tension fields and the corresponding buckling of the plates was very pronounced. The Tarzana 80% run was tried next. The table, however, was not able to function properly and had to be shut down manually. The problem arose towards the end of the test when the table began vibrating with a loud noise and at a high frequency rate. This was attributed to the severe out-of-plane vibration of thin infill panels which influenced the sensitive control accelerometer of the table. To continue the test, it was necessary to reduce the out-of-plane vibration of the plates. To restrain the plates in the out-of-plane direction, rubber composite pads together with high density foam pads were installed on opposite faces of the infill plates using 2×4 timber studs. The timber studs were tightened against the edges of the column flanges. This was done for the second, third and fourth storey plates as no interference with the structural behaviour of the first storey infill plate was intended.

Once the installation of the dampers was completed, the Tarzana Hill 80% run was repeated. The test was successfully carried out. The Tarzana Hill 120% run was tried next. It was expected that the test might cause some nonlinearity to the specimen. After running the test, the specimen was checked for any sign of yielding. Upon visual inspection, a number of horizontal cracks in the whitewash on the outer flange of one of the columns was observed. The cracks were distributed fairly uniformly along the first storey height. The specimen was then subjected to the Tarzana Hill 140% run. The previous yield lines on one of the column outer flange grew a little bit more toward the column web. Also, a few more whitewash cracks developed between the previous crack lines. At the base of the yielded column, whitewash cracking was observed in the weld location both outside the column flange and inside the web.

The typical acceleration time-histories recorded at the base and at storey floors together with the table displacement and storey floor relative displacements for the Tarzana Hill 140% run in the direction of excitation are shown in Figure 3. The maximum table acceleration and displacement reached for the Tarzana Hill record before the termination of the shake table test were 2.3g and 58.6 mm. The maximum absolute acceleration reached at the top of the specimen was 3.6g. The maximum relative displacement reached at the first, second, third and fourth storey floors were 8 mm, 12 mm, 20 mm and 30 mm, respectively.

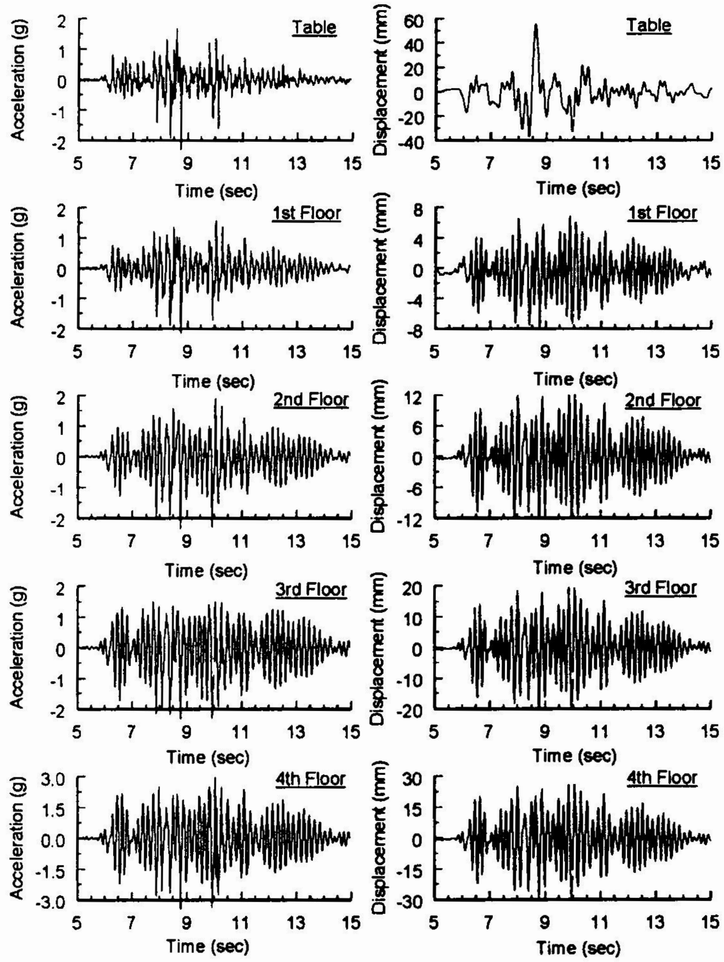

Figure 3: Time-histories of the table and floor accelerations, table displacement and floor relative displacements for Tarzana 140%

The envelopes of maximum response of relative displacements, interstorey drifts, ratio of floor accelerations to the table acceleration, storey inertia forces, storey shear forces, and overturning moments over the height of the test structure for the Tarzana Hill 80%, 120% and 140% runs are shown in Figure 4. The relative floor displacement and interstorey drift profiles illustrate the dominant tendency towards flexural deformation at the top floor. The maximum absolute interstorey drift for all Tarzana Hill tests occurred at the fourth floor. The first floor underwent considerable relative displacements with a peak value of about 8 mm for the Tarzana Hill 140% run. It is noted that the yield displacement of the quasi-static test specimen was established at about 9 mm. The displaced shape of the first and second storey floors was mostly dominated by a shear deformation, while the top floors behaved as a rigid body rotating about the second floor with some flexural action. This is demonstrated by the fact that the displaced shape of the column above the second floor was close to a straight line.

A comparison between recorded interstorey drifts and the seismic drift limitations prescribed by the National Building Code of Canada (NBCC, 1995) was carried out. The Canadian code specifies a limitation on interstorey drift of $0.01h$ for structures that are designed as post-disaster buildings and $0.02h$ for all other buildings where h is the storey height. For the steel plate shear wall test specimen, this would be equivalent to an interstorey drift of 9 mm for a post-disaster building and 18 mm for other buildings. During the Tarzana Hill tests, the maximum interstorey drifts were within the acceptable limits considering that the specimen was just at the onset of yielding. A maximum interstorey drift index of 1.06% ($0.0106h$) was recorded at the fourth floor. The corresponding maximum interstorey drift index of the first floor was 0.86% ($0.0086h$).

The above discussion highlights the fact that the design of the UBC prototype structure could very well be governed by the drift limitations and not by strength. As the specimen had considerable reserve capacity to undergo significant yielding, the limit in the interstorey drift of the storey floors would likely exceed the code limit at a ductility factor of about 2 times the yield displacement. It is noted that in a prototype steel plate shear wall structure, it is not generally desirable to let the interstorey drift ratios govern the design. It is, therefore, of paramount importance for a designer to accurately estimate the stiffness of tall slender multi-storey thin unstiffened steel plate shear walls to ensure satisfactory seismic performance.

The behaviour of the test structure was studied to investigate the distribution and dissipation of the energy in the structure during the Tarzana Hill 140% shake table test. Representative input energy and energy absorption time-histories for the test structure at the first and second storey floors are shown in Figure 5. This figure shows that the first floor dissipated a significant portion of the input energy. Although the absolute interstorey drift was larger at the fourth floor compared to the first and second floors, the overall energy dissipated was minimal because the storey shear was the smallest at the top floor. More importantly, a large portion of upper storey drift was due to rotation, which did not contribute to resisting input motion, and thereby did not dissipate energy.

Figure 6 presents the major and minor principal strains at the centre of bottom storey infill plate and near the base of the column together with the angle of principal strains. In general, the magnitude of principal strain increased with an increase in the intensity of shaking. It can be observed that the maximum computed tensile principal strain was in the order of 60% of the conventional yield strain of the steel. Compressive strains (negative) were produced in the infill plates, particularly at the corner locations near the column bases. Compressive principal strains were about one-third of the tensile principal strains at the centre of the bottom storey infill plate.

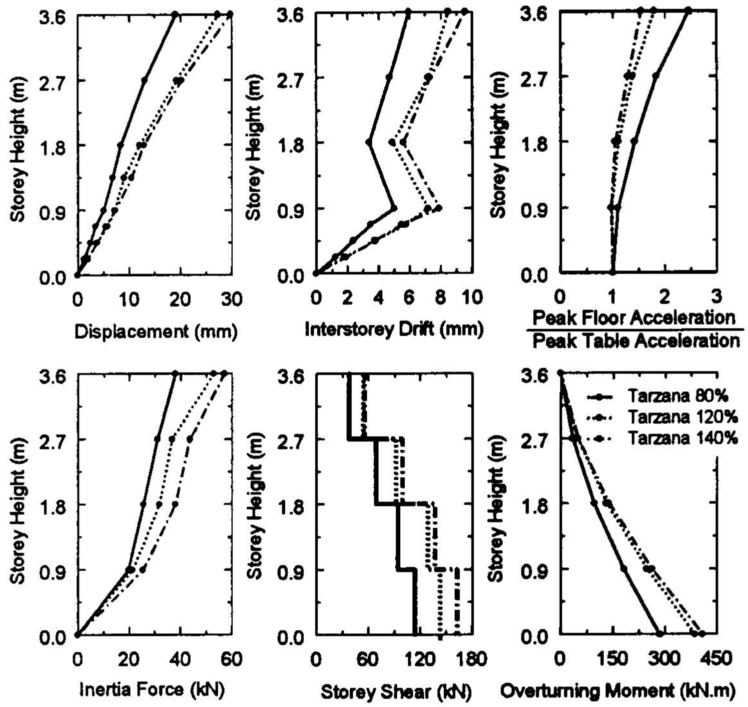

Figure 4: Maximum response envelopes of the test specimen for Tarzana Hill runs

The angle of principal strain alternated between 40° and 44° from vertical at the centre of the first storey infill plate. Near the base, the angle of principal strain varied between 36° and 40° for the loading direction where tension field was anchored at the base. These observations imply that the angle of principal strain was closer to the vertical at the corners. The considerable disturbances observed for the angle of principal strain near the column bases during reversed cycles, where the adjacent column was in compression, is an indication of the complex state of strains at the corners.

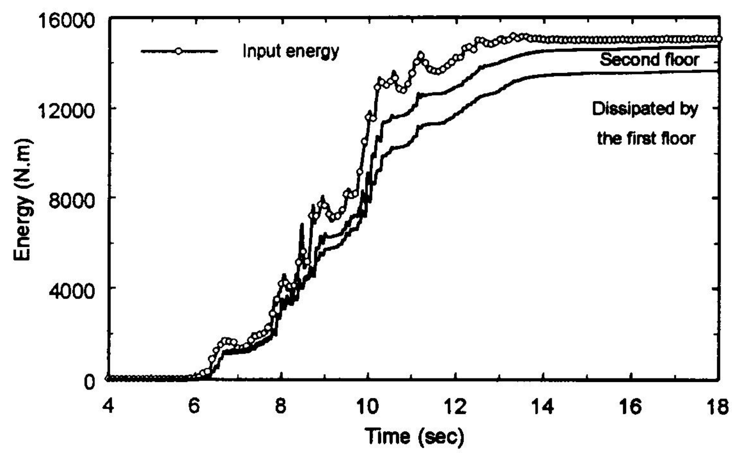

Figure 5: Input energy together with the first and second floor dissipated energy time-histories for Tarzana 140% test

An important observation from the angle of principal strain plots is that while the angle of inclination of principal strains near the base of the boundary frame altered quickly as soon as the load reversed, the angle of principal strain at the centre of the plate remained unchanged until the load passed through a zero base shear position. This observation is consistent with the slow cyclic test results. The gradual change in the angle of inclination of the tension field at the centre of the plate for reversed cycles is the main reason for the inherent pinched hysteresis loops observed for the UBC steel plate shear wall test specimens.

CONCLUSIONS

The results of experimental studies have shown that steel plate shear panels have the advantage of being a redundant continuous system exhibiting relatively stable and ductile behaviour under severe cyclic loading. This beneficial behaviour of steel panels along with the high stiffness contributed by the infill plates acting like tension braces greatly qualifies steel plate shear walls as an ideal energy dissipator system in high risk seismic regions, while providing an efficient system to reduce lateral drift. The results of this study signified an important attribute of a tall multi-storey steel plate shear wall frame in which the bottom storey panel attracts a major portion of the input energy. As a result, the demands on the column bases were shown to be extremely high. Therefore, it is of utmost importance that in a steel plate shear wall structure, the base of the columns be detailed carefully to ensure a robust ductile behaviour.

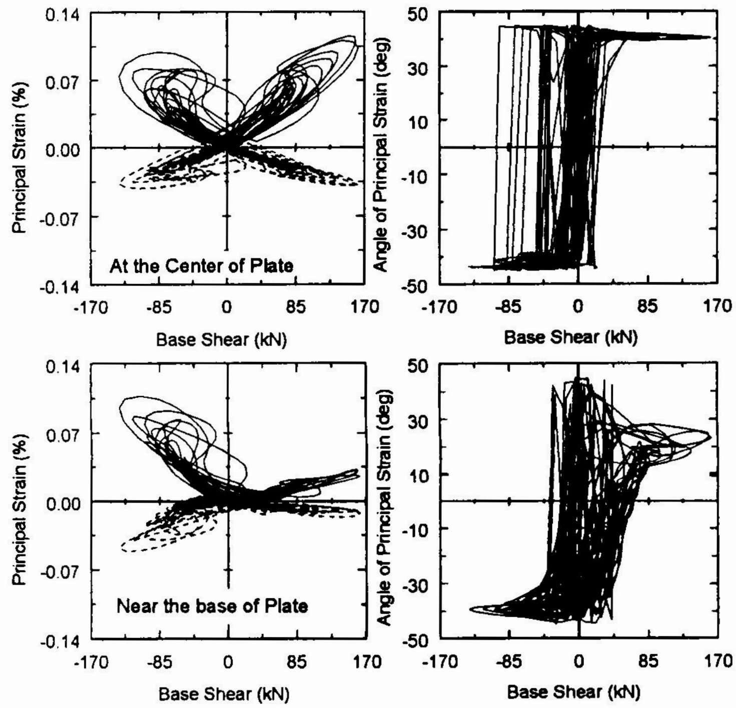

Figure 6: Magnitude and angle of principal strains at the centre and near the base of the first storey infill plate for Tarzana 140% run

The UBC steel plate shear wall specimens utilized the smallest panel width to height aspect ratio so far. The overall behaviour of the four-storey specimens were mainly dominated by flexural behaviour at the top floors. The results of shake table and quasi-static studies at UBC indicated that the design of the prototype steel plate shear wall frame would be governed by the drift limitations and not by strength. This was mainly related to the low stiffness characteristic of the specimens. It is, therefore, of paramount importance for a designer to accurately estimate the stiffness of tall slender multi-storey thin unstiffened steel plate shear walls to ensure satisfactory seismic performance.

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