Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1488

LARGE SCALE TEST OF A MODULAR STEEL PLATE SHEAR WALL WITH PARTIALLY ENCASED COMPOSITE COLUMNS

M. Dastfan[1] and R.G. Driver[2]

ABSTRACT

The behaviour of conventional steel plate shear walls has been studied in many research projects and the general behaviour of these walls is now reasonably well understood. Seismic loading introduces large axial forces in the boundary columns, which, with the addition of axial forces due to the gravity loads, require high axial strength. Also, for proper anchorage of the tension field that develops after buckling of the infill plate, the columns require high flexural stiffness. To fulfill these two requirements, i.e., high axial strength and flexural stiffness, without using overly deep members, composite columns are considered an attractive option. Partially encased composite columns have several advantages over fully encased columns such as simpler formwork, simpler connection of beams to columns, and lower cost of construction. Research at the University of Alberta is investigating the potential for making partially encased composite columns a practical option for the vertical boundary elements in steel plate shear walls.

The research program involves the design and testing of three steel plate shear walls with partially encased composite columns as the vertical boundary elements. One of these walls is constructed of modules that are bolted together, thus requiring no field welding. The modular concept facilitates site assembly and reduces erection costs. This modular steel plate shear wall is tested under constant gravity loads and lateral cyclic loads that increase gradually until the specimen eventually fails. This paper describes the specimen, the applied loading pattern and the loading mechanism, some design aspects, as well as key results of the completed test.

Introduction

One of the most effective means of resisting lateral loads, particularly forces applied to a structure during seismic events, is the steel plate shear wall system. It consists of a vertical steel infill plate connected to a surrounding frame of beams and columns to transfer lateral loads to the foundation. Columns of steel plate shear walls usually are required to have a large compressive

[1]Ph.D. candidate, Department of Civil and Environmental Eng., University of Alberta, Edmonton, Alberta, Canada
[2]Professor, Department of Civil and Environmental Eng., University of Alberta, Edmonton, Alberta, Canada

capacity, as they carry both gravity loads and axial loads introduced by overturning moment. In addition, for proper anchorage of the tension field in the infill plate, boundary columns should have a minimum flexural stiffness (Dastfan and Driver 2008). Using composite columns is an attractive option to fulfill these two requirements.

Partially encased composite columns (PEC columns) were introduced in the mid-1990s by the Canam Group to carry gravity loads in mid- and high-rise buildings. They consist of a welded H-shaped steel section with transverse links that are welded between the flanges, close to the flange tips, and spaced at regular intervals to increase the local buckling capacity of the thin flanges. Fig. 1a shows the cross-section of a PEC column. At each floor level, side plates are welded to the column flange tips to provide a means for connecting the perpendicular beams framing into the column. Concrete is cast between the flanges at the same time as the floor slab above is cast. Contrary to columns with standard wide-flange sections, there is no limitation on the size of column and with the various plate thicknesses available, columns can be sized to meet exactly the requirements under both construction and service conditions. As the plates used in construction of the H-shaped steel section are relatively thin, the weight of the column at the time of erection is low, which reduces the size of the crane required on site. The simple formwork also makes this type of composite column more economical. Fig. 1b shows the sequence of construction of these columns.

Several experimental studies have been conducted on the behaviour of isolated PEC columns under axial loads (Tremblay *et al*. 1998; Chicoine *et al*. 2000; Bouchereau and Toupin 2003; and Prickett and Driver 2006). Design equations were proposed for the axial capacity of this type of column (Tremblay *et al*. 2000; Chicoine *et al*. 2002). Muise (2000) did experimental work on the behaviour of simple frame connections to PEC columns. A finite element model that accounts for nonlinear material properties, initial imperfections, and residual stresses was developed by Chicoine *et al*. (2002) to predict the ultimate capacity of PEC columns. Begum *et al*. (2007) developed another finite element model that was able to give the full response history, including the rapid expansion of the concrete near and after the ultimate load. This model was capable of tracing the stable post-peak behaviour.

One of the advantages that make PEC columns more desirable than other types of composite columns is that they provide steel surfaces (i.e., flange surfaces), for welding the infill plate to the columns. As the infill plate is welded to the center of the flange, where the column web is welded to the other side of flange, as shown in Fig. 1a, it provides a direct transfer of force to the column.

To investigate the behavior of PEC columns in the steel plate shear wall system, three large-scale tests have been conducted at the University of Alberta. Deng *et al*. (2008) conducted the first test in this series on a steel plate shear wall with rigid beam-to-column connections to explore the overall behavior of the system and to highlight areas where details can be improved to optimize the system in terms of both performance and economics.

To reduce the overall cost of the SPSW system for use in low to moderate seismic regions where maximum ductility is not required, a modular fabrication method has been proposed wherein the modules are connected to each other by bolting, as shown in Fig. 2.

Modules are fabricated and inspected in the shop and then shipped to the site for assembly. The infill modules are connected to the PEC columns and beams by bolting to a fish plate that has been welded to the columns and beams in the shop. To connect the infill modules to each other, double lap plate splices are used at mid-height in the story. Modules may be fabricated in one or more story high assemblies to suit the fabrication, lifting, and assembly conditions of the individual project.

This modular method of construction completely eliminates the need for field welding and as a result, the cost of the shear wall system is expected to be reduced significantly. Also, since the floor beam flanges are not connected to the columns, the connection will be able to accommodate somewhat larger rotations, which may reduce the chance of hinging in the columns close to the beam-to-column connections. The simple beam-to-column connections will in effect eliminate any contribution to the performance of the wall through frame action and, as a result, the modular system will likely qualify for a somewhat lower seismic force modification factor for design than a shear wall with moment-resisting beam-to-column connections.

Experimental Design

The specimen tested had an overall height of 4.09 m and an overall width of 2.69 m, excluding the base plate. Stories were 1.90 m high and the column centerline spacing was 2.44 m. Plates of 3 mm thickness were used for the infill panels. The overall test set-up is depicted in Fig. 3. Gravity loads were applied to the tops of the columns equally through jacks connected to gravity load simulators, which kept the gravity loads vertical when the specimen experienced large lateral displacements. Each column sustained approximately 600 kN of axial force, which was around 25 to 30 percent of its axial capacity.

The test specimen was connected to the strong floor by 12 high-strength pre-stressed anchor rods. Additional anchored plates were installed on each end of the base plate to prevent specimen sliding. The columns were braced near each floor level to prevent out-of-plane displacement of the columns.

The maximum out-of-plane imperfection of the first floor infill plate from the plane of the wall centerline was 38 mm and was located close to the splice plate at the mid-height of the first story. All the fasteners connecting the modules were pre-tensioned according to the provisions of the Canadian steel design standard (CSA 2001).

To increase the ductility of the columns at the base, where the hinges were expected to form, the spacing of the links was decreased and longitudinal rebars were used. The details at the column base are shown in Fig 4. The first floor of the specimen before the column concrete was cast is shown in Fig. 5.

Lateral loads were applied to each floor equally through the top flanges of the floor beams to avoid local failure of the PEC column due to the loading mechanism itself. The lateral loading procedure was based on the method outlined in ATC-24 (ATC 1992). Since the first floor was of primary interest, the first floor deflection (δ) was chosen as the “deformation control parameter” and the base shear was chosen as the “force quantity”. In the elastic range, nine cycles were carried out, in which the “force quantity” was used to control the lateral loads, while the “deformation control parameter” was used to control the lateral loads in the remaining cycles. The load and deflection history during the test is listed in Table 1.

General Observations During the Test

During the first nine cycles, in which the test was conducted by controlling the base shear force, several flexural-type cracks developed in the concrete close to the bases of the columns. The infill plate in the first floor buckled in one half-wave in the first six cycles and in two half-waves in the following three cycles. In cycles 7 to 9, one wave of buckling was visible in the infill plate at the second floor. In cycle 7, a small flexural crack was observed in the concrete at the top of the first-story column, indicating that the column was deforming in double curvature. The connection between the first floor beam and column had not yet slipped and was thus carrying significant moment. During cycle 9, white wash on the first-story infill plate started to flake off close to the upper corner of the panel, which indicated yielding of the infill plate.

From cycle 10 to the end of the test, the controlling parameter was the first floor deflection (δ). In cycles 10 to 12 (cycles to reach δ = 8.5 mm), some diagonal cracks started to develop in the concrete close to the bases of the columns. The infill plate in the first story buckled in three half-waves, and two waves were visible in the second story. In cycle 10, some bolt slip noises were heard that were from the first floor beam-to-column connections. In cycle 13 (cycle to reach δ = 17 mm), a loud noise was heard from the second floor beam-to-column connections and rotation meters recorded a sudden rotation in one connection at δ = 13.6 mm. In cycle 14, the concrete cracks in the columns propagated and the rotations in the first floor beam-to-column connections were clearly visible. There were some new signs of infill plate yielding in the first story and minor noises were heard from bolts around the infill plates and splice plates at the center of the infill plate. The number of half-waves in the first story infill plate increased to five. In cycle 15, the rotation in the second floor beam-to-column connection was clearly visible and this rotation caused an increase in the number of buckling half-waves in the second floor. The first signs of local buckling of the column flanges were observed in cycle 16. In cycle 17, the flanges of the columns in tension started to tear. The first tearing due to low-cycle fatigue of the thin infill plate from localized kinking was observed in cycle 18. During cycles 18 to 21, the base shear was increasing and in cycle 21 (cycle to reach δ = 42.5 mm) the maximum base shear (1824 kN) was attained. In cycle 22, new tearing started to develop in the first story infill plate. It was in cycle 24 that the flanges of the columns in tension tore completely and the tear began to propagate into the web. As a cumulative result of the damages in the columns and infill plate, the capacity of the specimen dropped gradually after the ultimate capacity was reached.

There were several new tears in the first story infill plate at the end test. Fig. 6 shows the infill plate in the first story after the test completion. The yield lines in the infill plate and change of angle at the beam-to-column connection are visible in the figure. The maximum rotation in the first floor beam-to-column connection was close to 2.3 degrees and the maximum rotation was 3.5 degrees in the second floor beam-to-column connection. The infill plate started to tear at the crest of buckles that formed adjacent to the connections as a result of these connection rotations.

Failure Mode

Tears at the bottom of the columns initiated at the flange tips due to tension from the overall frame action. In general, the column flange tears initiated right at the top of the side plates located at the bases of the columns. They eventually opened through the entire exterior column flange and then tore into the column web, which led to the gradual reduction of the capacity of the specimen. The presence of longitudinal rebars in the columns helped stop propagation of tearing in the column web and thus no sudden drop in base shear was observed, compared to the first test of the series. At the largest first story deformation (δ = 68 mm), the flange tear openings at the center of the flanges of both columns (i.e., web-to-flange junction) were less than that observed in the first test, as a result of the presence of the longitudinal rebar. Base side plates were used primarily to move the position of maximum column demand away from the welds that connect the columns to the base plate.

There were several tears in the infill plate due to low-cycle fatigue failure from localized kinking. At the end of the test, the length of the longest one close to the upper corner of the panel was 235 mm with a 25 mm opening. The presence of these tears did not cause a sudden reduction in wall strength as alternative load paths were available.

Overall Behavior of the Steel Plate Shear Wall

The base shear versus first floor displacement is plotted in Fig. 7. The pinching of the hysteresis loops is mainly because of the shear buckling of the infill plates and is seen in all cycles, as the infill plate was thin and unstiffened and therefore buckled in early cycles. The kinks in the curves identify instances of sudden rotation of the beam-to-column connections due to slip of the pretensioned bolts, as well as buckling and stretching of the panel. The hysteresis loops flatten as test progressed because of the nonlinear behavior of the concrete and steel in cycles with higher deflections.

The maximum base shear was reached when a story deflection equal to $5\delta y$ = 42.5 mm was applied to the first floor. This result is comparable with the result of the first test of the series, although the cost of construction of this specimen is lower. At $8\delta y$ = 68 mm, the base shear force was approximately 85 percent of the maximum base shear force.

Summary and Conclusions

A large-scale two-story modular steel plate shear wall specimen with PEC columns was tested to failure to study its behaviour under cyclic lateral loading and to observe possible local failures. The columns and the system behaved in a ductile manner. The beam-to-column connections behaved well and rotated 2.3 and 3.5 degrees at the first and second floors, respectively. Due to the presence of longitudinal rebars at the column bases, the base shear did not decrease suddenly after the column flanges were torn completely. The stiffness of the columns was enough to provide anchorage for the tension field developed in the infill panels as the whole plate yielded in the first story. The failure mode was identified as tearing of the column flanges at the base, combined with concrete crushing, and several tears in the first story infill plate.

Acknowledgment

Funding for this project is provided by the Canam Group, Saint-Georges, Quebec, the Natural Sciences and Engineering Research Council of Canada, the Alberta Ingenuity Fund Scholarship from the Alberta Heritage Foundation, and the Honorary Izaak Walton Killam Scholarship from the University of Alberta. Helpful discussions with Richard Vincent and Pierre Gignac of the Canam Group regarding the specimen details are gratefully acknowledged.

Tables and Figures

Table 1. Load and deflection history

	Force Control Parameter Base shear (kN)	**Deformation Control Parameter First floor deflection (mm)**
Cycles 1-3	±300	–
Cycles 4-6	± 600	–
Cycles 7-9	± 900	–
Cycles 10-12	–	$\delta y = \pm 8.5$
Cycles 13-15	–	$2\delta y = \pm 17$
Cycles 16-18	–	$3\delta y = \pm 25.5$
Cycles 19-20	–	$4\delta y = \pm 34$
Cycles 21-22	–	$5\delta y = \pm 42.5$
Cycles 23-24	–	$6\delta y = \pm 51$
Cycles 25-26	–	$7\delta y = \pm 59.5$
Cycles 27	–	$8\delta y = \pm 68$

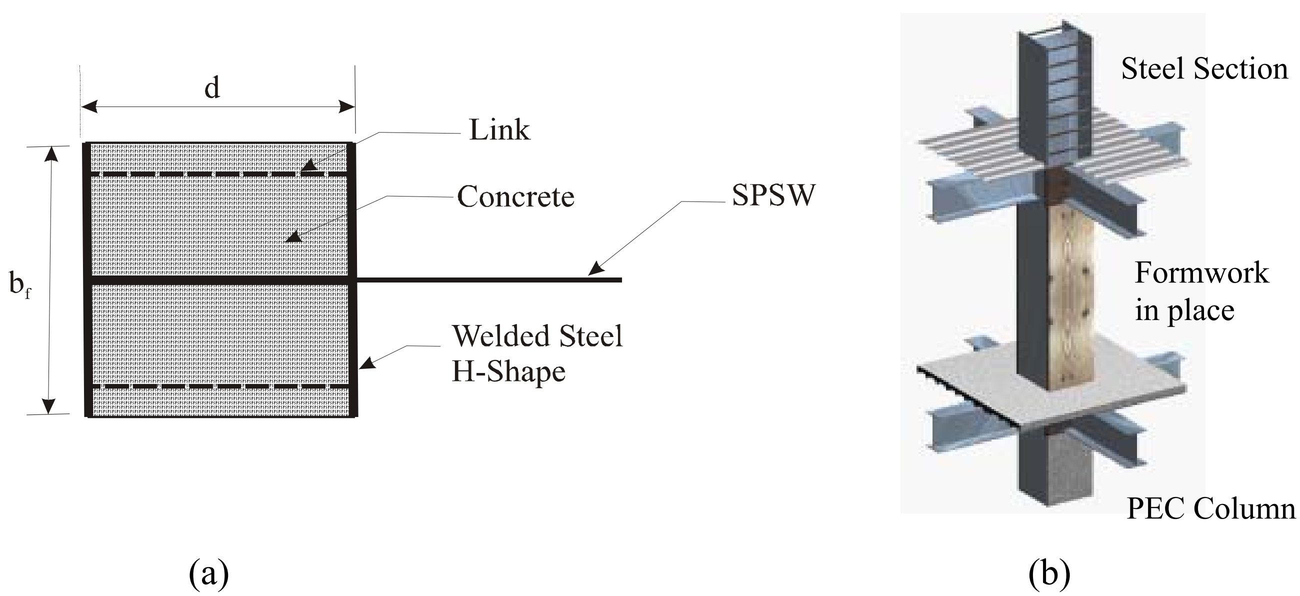

Figure 1. (a) Typical cross-section of PEC column; (b) Sequence of construction (illustration courtesy of R. Vincent, Canam)

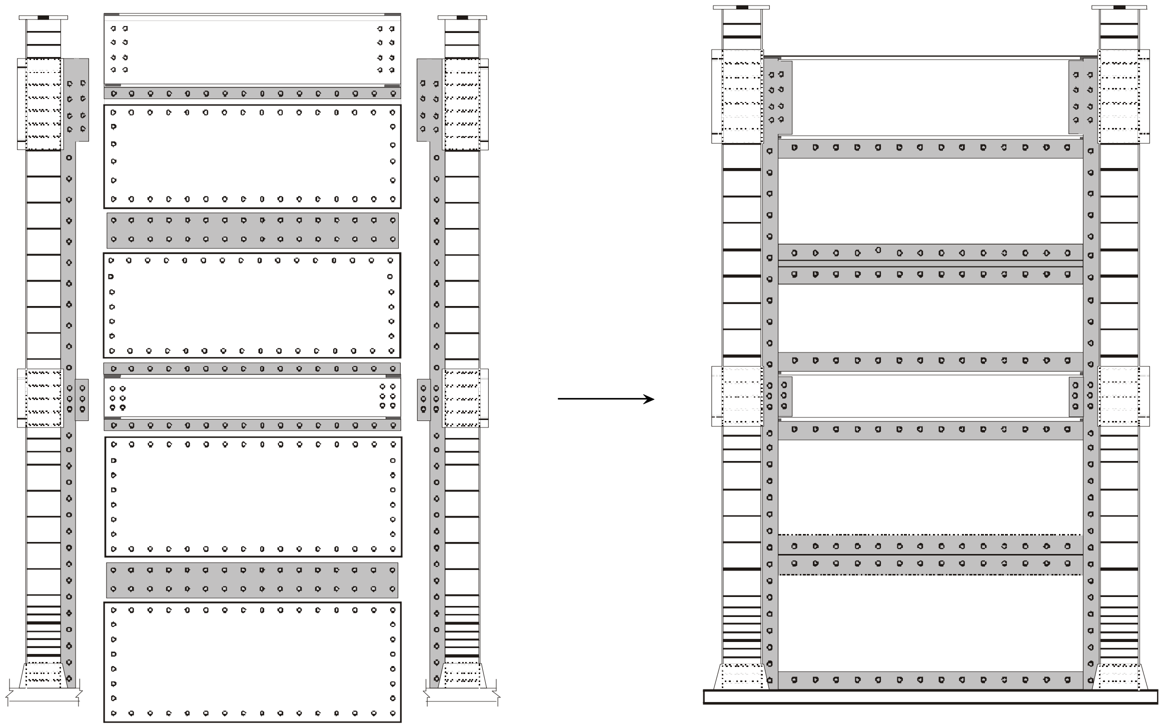

Figure 2. Modular SPSW test specimen with PEC columns:
(Left) Exploded view; (Right) Assembled view

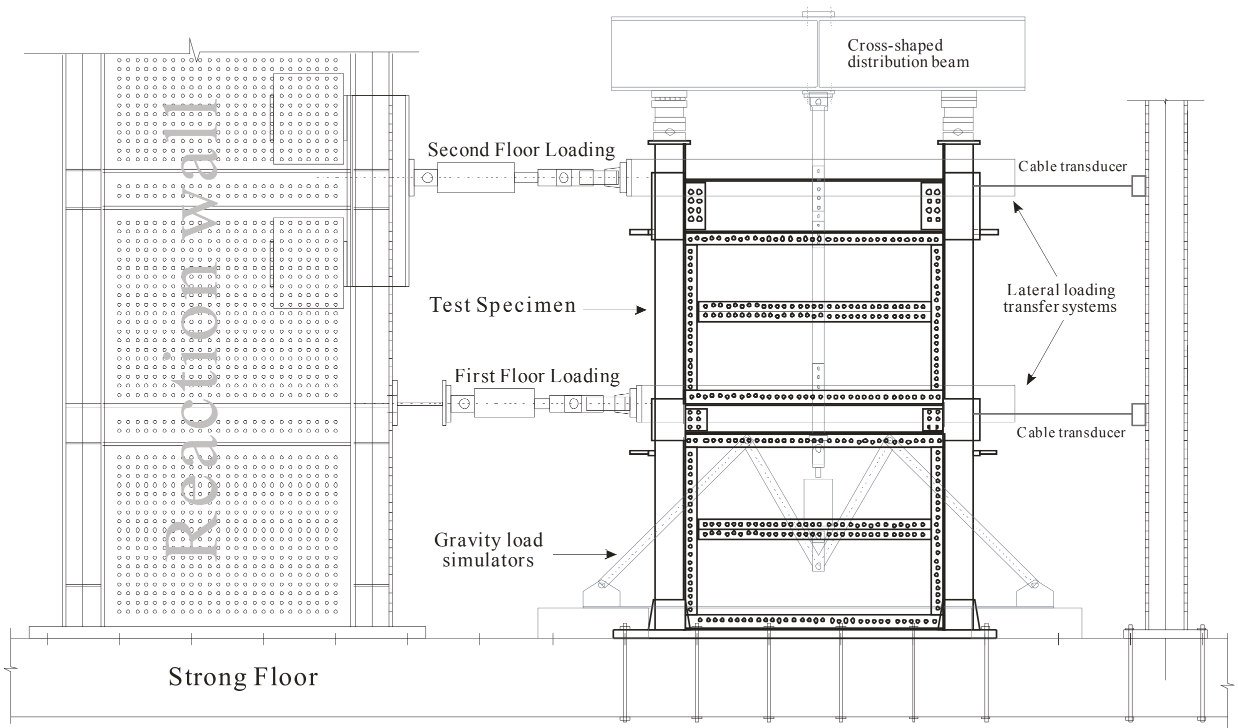

Figure 3. Overall test set-up

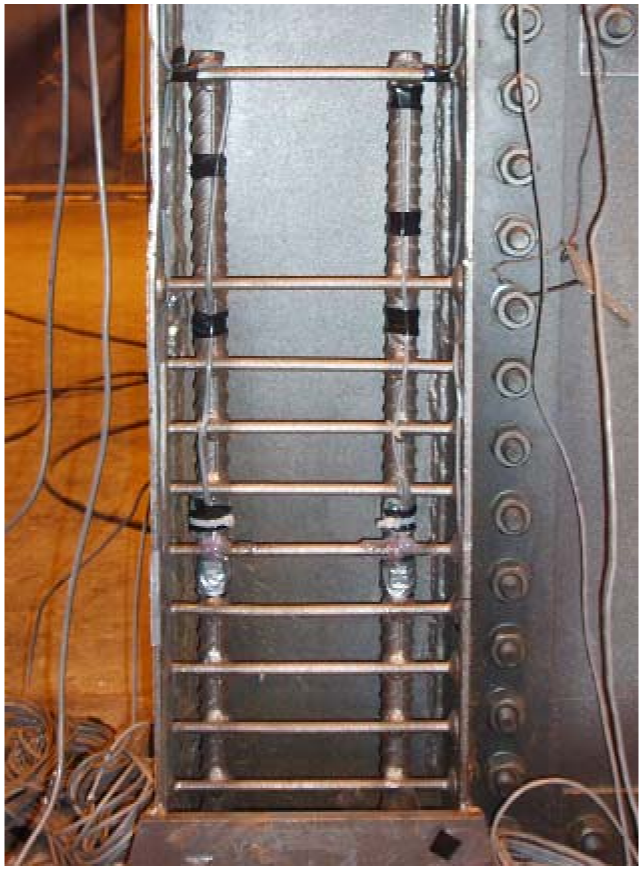

Figure 4. Details of columns at the base

Figure 5. First floor of steel plate shear wall with PEC columns before casting concrete

Figure 6. First floor infill plate after the test

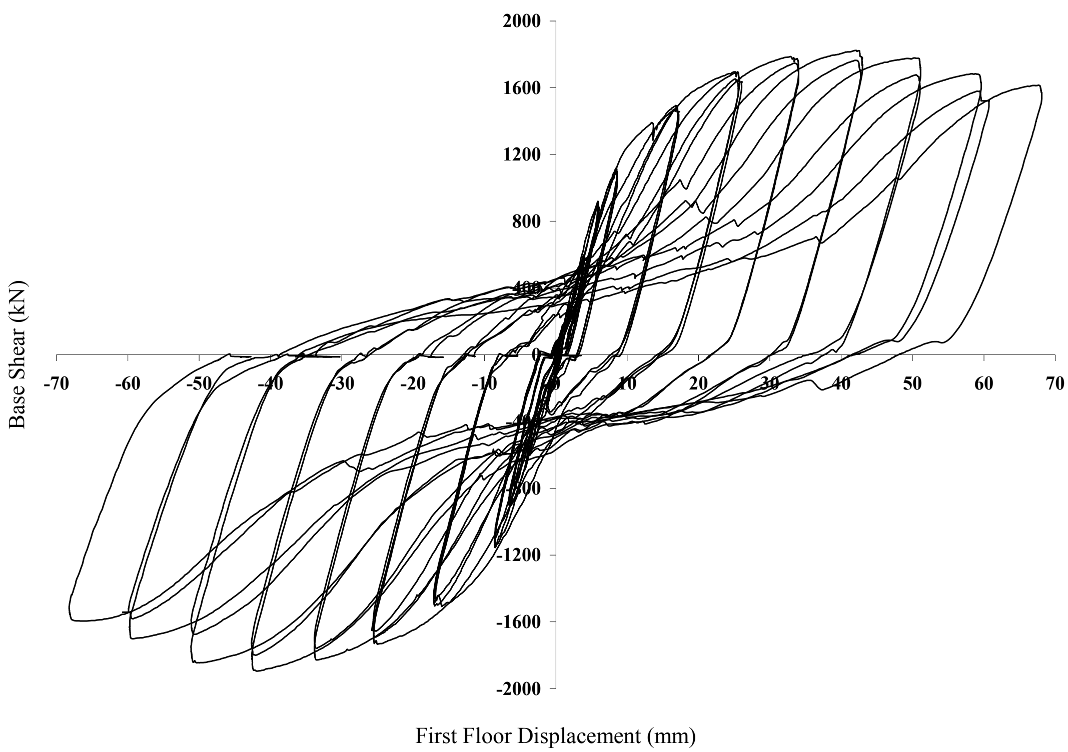

Figure 7. Base shear versus first floor displacement

References

ATC, 1992. Guidelines for Cyclic Seismic Testing of Components of Steel Structures, *Report 24*, Applied Technology Council.

Begum, M., Driver, R.G., Elwi, A.E., 2007. Finite Element Modeling of Partially Encased Composite Columns using the Dynamic Explicit Method, *Journal of Structural Engineering*, American Society of Civil Engineers, 133(3), 326-334.

Bouchereau, R., Toupin, J.-D., 2003. Étude du Comportement en Compression-Flexion des Poteaux Mixtes Partiellement Enrobés. *Report EPM/GCS-2003-03*, Dept. of Civil, Geological and Mining Engineering, École Polytechnique, Montréal, QC, Canada.

Chicoine, T., Tremblay, R., Massicotte, B., Yalcin, M., Ricles, J., Lu, L.-W., 2002. Behavior and strength of partially encased composite columns with built-up shapes, *Journal of Structural Engineering*, 128(3), 279-288.

Chicoine, T., Tremblay, R., Massicotte, B., Yalcin, M., Ricles, J., and Lu, L.-W., 2000. Test programme on partially-encased built up three-plate composite columns, *Joint Report EPM/GCS No. 00-06*, Dept. of Civil, Geological and Mining Engineering, École Polytechnique, Montréal, QC, Canada – ATLSS Engineering Research Centre, *No. 00-04*, Lehigh University, Bethlehem, PA, USA.

CSA, 2001. *CAN/CSA-S16-01: Limit States Design of Steel Structures*, Canadian Standards Association, Toronto, ON, Canada.

Dastfan, M., Driver, R.G., 2008. Flexural Stiffness Limits for Frame Members of Steel Plate Shear Wall Systems, *Proceeding of Annual Stability Conference, Structural Stability Research Council,* 2-5 April, Nashville, TN, USA.

Deng, X., Dastfan, M., Driver, R.G., 2008. Behaviour of Steel Plate Shear Walls with Composite Columns, *Proc., Structures Congress*, American Society of Civil Engineers, April 24-26, Vancouver, BC, Canada.

Muise, J.L., 2000. Behaviour of Simple Framing Connections to Partially Concrete Encased H-section Columns, *M.Sc. thesis*, Department of Civil Engineering, University of Toronto, Toronto, ON, Canada.

Prickett, B.S., Driver, R.G., 2006. Behaviour of Partially Encased Composite Columns Made with High-performance Concrete, *Structural Engineering Report No. 262*, Dept. of Civil and Environmental Engineering, University of Alberta, Edmonton, AB, Canada.

Tremblay, R., Chicoine, T., Massicotte, B., 2000. Design equation for the axial capacity of partially-encased noncompact columns, *Proc. Composite Construction IV*, Engineering Foundation, Banff, AB, Canada.

Tremblay, R., Massicotte, B., Filion, I., Maranda, R., 1998. Experimental study on the behaviour of partially encased composite columns made with light welded H steel shapes under compressive axial loads, *Proc. SSRC Annual Technical Meeting*, Atlanta, GA, USA.