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Preliminary Investigation of the Effects of Out-of-Plane Loading on the In-Plane Behavior of SC Walls

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ABSTRACT

A research project focussing on the effects of out-of-plane loading on the in-plane behaviour of SC walls is being conducted. This paper presents the initial findings from the research project, including: a numerical investigation of the effects of out-of-plane force on in-plane response, the influence of parameters such as aspect ratio, magnitude of out-of-plane force, tie bar spacing on behavior, the test matrix, the test fixture, and sample results from the first test.

Numerical investigations indicate that significant out-of-plane loading cracks the concrete through the wall thickness and potentially reduces the in-plane shear strength. The reduction is primarily due to the longitudinal stresses in the steel faceplates caused by the out-of-plane bending moment produced by the out-of-plane loading. Experimental investigations were conducted using a specially designed and built test setup, which functioned as designed during the first test. The test results for the control and the first specimen indicate that out-of-plane loading equal to the design shear strength ($V_n = V_c$) does not reduce the in-plane shear strength of the SC wall pier specimen with height-to-length aspect ratio of 0.6.

INTRODUCTION

Modular steel-plate composite (SC) walls consist of steel faceplates and infill concrete. The faceplates are tied to each other by tie rods and joined to the concrete by steel anchors. Prior research on SC walls has focused on structural behaviour, analysis, and design when subjected to in-plane shear, out-of-plane shear, and combinations of membrane forces and out-of-plane moments. This research forms the basis of Supplement No. 1 to AISC N690 [AISC N690s1 (AISC, 2014)] for the design of steel structures in nuclear facilities. The supplement includes Appendix N9 for designing SC walls. Currently, there is lack of information and data on the in-plane behaviour of SC wall piers while simultaneously subjected to out-of-plane loading. This is an expected loading environment for SC walls at horizontal connections to foundations and floors and vertical connections to other walls.

This paper presents initial findings from a research project focussing on the effects of out-of-plane loading on the in-plane behaviour of SC walls. The paper includes: (i) background information regarding the in-plane shear behaviour of SC wall piers and out-of-plane shear behaviour of SC walls, (ii) results of preliminary numerical studies conducted to design the experimental investigations and specimens, and (iii) the experimental test matrix, the test setup, preliminary data from the tests conducted. The numerical models will be benchmarked and improved after the testing program is completed.

BACKGROUND: BEHAVIOUR OF SC WALLS UNDER IN-PLANE AND OUT-OF-PLANE LOADING

Labyrinthine structural systems are used for safety-related nuclear facilities. The in-plane behaviour of the walls in such construction is governed by the presence of flanges (transverse cross walls) and boundary elements. In contrast, the in-plane behaviour of SC wall piers (i.e., walls without flanges or cross-walls) depends on the ratio of height-to-length (aspect ratio) of the wall.

For wall piers, Kurt et al. (2013) and Epackachi et al. (2014) have shown that:

- For larger aspect ratios, greater than or equal to 1.5, the in-plane behavior is governed by the flexural capacity of the wall at its base.
- For lower aspect ratios, between 0.6 and 1.0, the in-plane behavior is governed by interaction between in-plane flexure and in-plane shear.

As shown in Figure 1 by Kurt et al. (2013), the in-plane shear strength (or lateral load capacity) of SC wall piers increases with decreasing aspect ratio, but never reaches the shear strength of steel plates alone ($0.6A_sF_y$). For aspect ratios greater than 1.0, the lateral load capacity exceeds the flexural capacity (M_n) of wall cross-section at the base.

The out-of-plane shear behavior of SC walls has been investigated by several researchers, for example, Ozaki et al. (2001), Hong et al. (2009), and Varma et al. (2011). Sener and Varma (2014) have compiled the database of tests done around the world, and compared the experimental results with ACI 349 (ACI, 2006) code shear strength equations. Figure 2 presents this comparison, and indicates that ACI 349 provides a conservative estimate of the out-of-plane shear strength of SC walls. The figure also indicates that the out-of-plane shear strength reduces with increasing shear span-to-depth ratio, and with increasing depth of the cross-section. As discussed in Sener and Varma (2014), experimental results indicate that out-of-plane shear force causes diagonal shear cracking through the concrete thickness. If the tie bar spacing is less than the wall thickness, the concrete cracks engage the tie bars (that they pass around) in axial tension. The tie bars eventually yield, and the out-of-plane shear strength (V_n) is reached.

Depending on the magnitude of the applied out-of-plane loading, concrete cracking through the wall thickness can challenge the in-plane behaviour of the SC wall pier. Additionally, the steel faceplates are subjected to vertical tensile and compressive stresses produced by the combination of out-of-plane bending moment and in-plane bending moment at the base of the wall. These stresses cause potentially asymmetric yielding, local buckling and fracture of the steel faceplates. The SC wall pier is vulnerable to twisting and lamellar or splitting failure through the concrete, which may be resisted by the tie bars.

Based on the discussion above, the parameters important for the combined in-plane and out-of-plane loading tests are: (i) the steel faceplate reinforcement ratio, (ii) the tie bar size, spacing, and reinforcement ratio, (iii) magnitude and location of out-of-plane shear force, and (iv) in-plane shear performance, which is affected by the wall aspect ratio.

The parameters that will be held constant in the experimental investigations are: (i) the steel faceplate thickness and yield stress, (ii) concrete compressive strength (assuming little gain in strength between tests), and (iii) the wall aspect ratio. A36 grade steel faceplates are chosen as their expected yield stress is close to 50 ksi. This yield stress is representative of the faceplates used in nuclear structures. The concrete compressive strength has been set equal to 4000 psi for the same reason. Based on observations by Kurt et al. (2013), the specimen aspect ratio has been kept equal to 0.6 in order to ensure that the wall pier is shear critical for in-plane loading. The plate slenderness ratios (ratio of steel anchor spacing, s, to faceplate thickness, t_p) have been setup to meet the limit recommended by Zhang et al. (2014) to develop faceplate yielding in compression before local buckling, that is, $s / t_p \leq (29000 / 36)^{0.5} = 28$. Steel faceplate thickness has been set at 3/16 in. (0.1875 in.) because it is the smallest plate thickness in the structural plates category. Smaller thicknesses will result in sheet metal properties and associated waviness imperfections. Steel headed stud anchors 0.375 in. in diameter are used. This will result in steel anchor diameter-to-plate thickness (d/t_p) ratio of 2, which are representative of nuclear structures.

The parameters that will be varied are i) magnitude of out-of-plane force applied, ii) tie bar spacing, and iii) steel anchor spacing. Preliminary numerical analysis are performed to finalize the specimen details and the out-of-plane loading location. The analysis forecast the expected behaviour of the specimen.

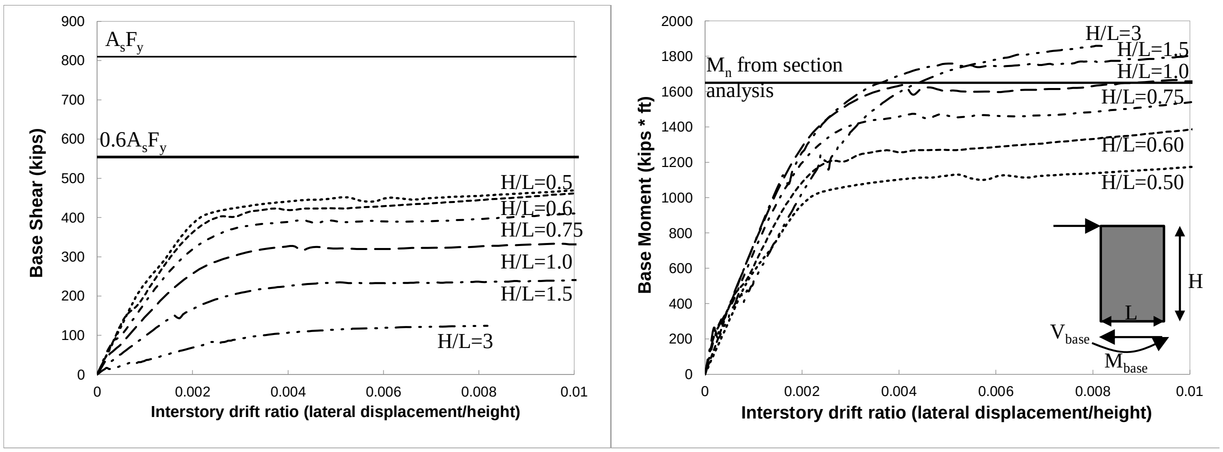
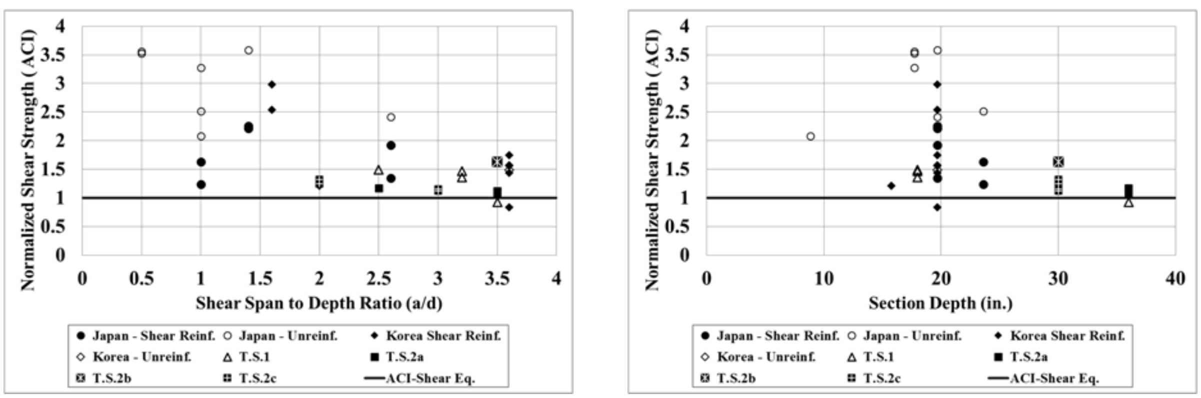


Figure 1. Lateral load capacity and behavior of SC wall piers (Kurt et al. 2013)



(a) *(b)*

Figure 2. Variation of out-of-plane shear strength of SC walls (Sener and Varma, 2014)

NUMERICAL MODELS

Benchmarked LS-Dyna (Hallquist, 2006) finite elements models by Kurt et al. (2013) were used to study the effect of out-of-plane loading on the in-plane capacity of the SC walls. The wall pier thickness is 12 in., the length and height of the pier are 60 in. and 36 in. respectively (to maintain an aspect ratio of 0.6). The finite element model parameters are presented in Figure 3. The material properties for the models discussed here are based on the first test conducted recently (initial model runs were made with material properties based on Kurt et al, 2013). The compressive strength for concrete infill is 7700 psi and is based on the specimen cylinder test data on the day of testing. The faceplate material model has yield strength of 47 ksi and tensile strength of 80 ksi and failure strain value of 15%. These values are based on the tension coupon tests for the faceplates. The foundation is modelled with elastic concrete properties. The studs, tie bars, steel dowels and anchors are modelled as beam elements with material properties consistent with benchmarked models.

The effect of wall height-to-length aspect ratio on the in-plane shear behaviour of SC wall piers has been studied by Epackachi et al. (2014) and Kurt et al. (2013), and not repeated here for brevity. The numerical studies presented here focus on the effects of out-of-plane loading (shear and moment) on the in-plane shear (lateral load capacity) of SC wall piers. Additionally, the effects of changes in parameters are also discussed.

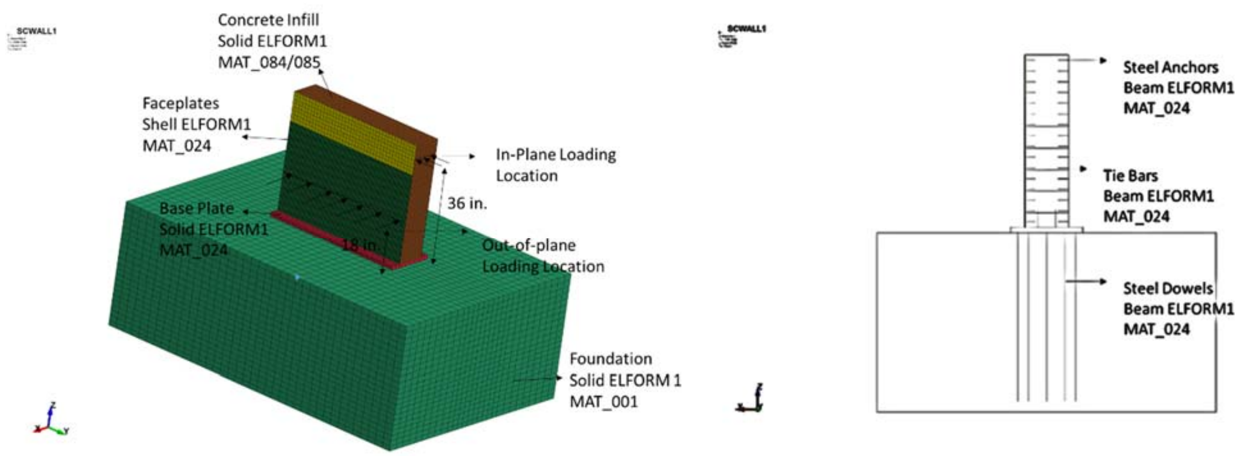


Figure 3. LS Dyna Model Parameters

Behaviour of SC Wall Piers

The behaviour and strength of SC wall piers subjected to in-plane and out-of-plane forces is compared to that of SC walls subject to in-plane forces only. The out-of-plane force is applied at mid-height of the wall pier. As shown in Figure 4, the failure of the SC wall piers is no more symmetric when out-of-plane (OOP) force is applied in addition to in-plane (IP) force. The resulting out-of-plane moment causes longitudinal tensile and compressive stresses in the two faceplates. The in-plane force adds to the tension on one corner of one faceplate and compression on the opposite corner of the other faceplate. This leads to buckling and tensile rupture failure in the opposite corners of the two faceplates (as can be seen in Figure 4a). Since the longitudinal stresses due to out-of-plane forces are caused by the resulting moment, the location of the out-of-plane force plays an important role.

Additionally, twisting of the SC wall piers subject to in-plane and out-of-plane loading is observed. With a constant out-of-plane force applied, the application of incremental in-plane force causes an increase in out-of-plane displacement along with the in-plane displacement. As observed in Figure 5, the collapse of the SC wall pier subjected to in-plane and out-of-plane loading is due to lateral overturning of the wall after the faceplate fails in tensile rupture and compression buckling.

Effect of Location of Out-of-plane Force

As discussed above, the location of out-of-plane force plays an important role in determining the in-plane capacity of SC wall piers. Figure 6 shows the percent reduction in the in-plane capacity of the SC wall pier as a function of out-of-plane loading location (measured from base of wall pier) per unit height (OOP/H). The OOP force is of magnitude $2\sqrt{f'_c}$. Since, the in-plane capacity is affected by the stresses due to out-of-plane moment, aspect ratio of 1 is used for this study. The effect of stresses due to moments is more prominent in higher aspect ratios. As the height of out-of-plane loading location increases, the associated moment increases, leading to higher longitudinal stresses in the faceplates. These stresses reduce the additional in-plane loading that can be applied on the wall before failure. The change in the location of out-of-plane force also affects the behaviour of the SC wall piers. As observed in Figure 7, for OOP/H value of 0.2, the stresses due to out-of-plane moment are less in magnitude and the compressive stress due to in-plane moment is able to initiate compression buckling in both the faceplates. For OOP/H value of 1, the stresses due to out-of-plane moment are higher and the compressive stress due to in-plane moment is unable to cause buckling in one of the faceplates.

Since the in-plane capacity of the specimen decreases significantly when the out-of-plane loading is closer to the top of the wall, the out-of-plane loading location for the experimental investigations is chosen to be mid-height of the wall pier. This is done to ensure that the moment due to out-of-plane loading is not very high (OOP loading at top) or very low (OOP loading close to the base).

Effect of Parameter Variation

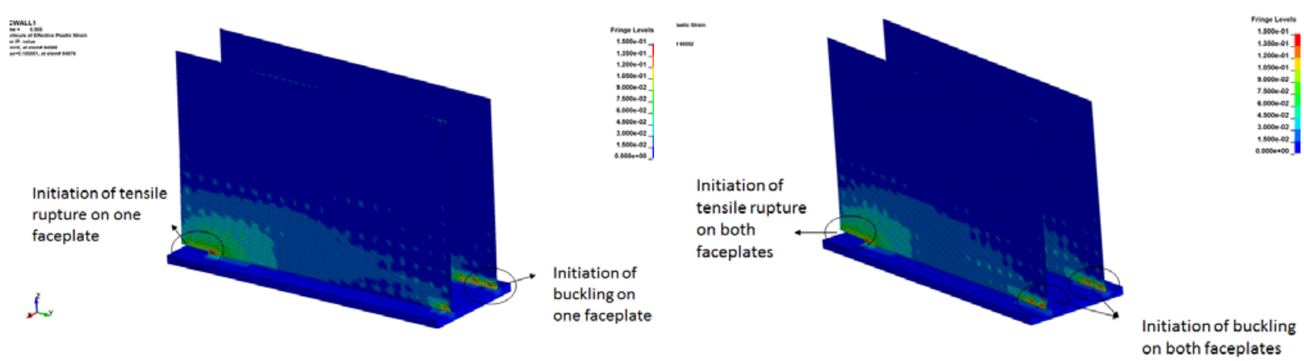
Numerical studies have been performed to study the effect of variation of parameters such as tie bar spacing and out-of-plane loading magnitude. The observations from these studies are presented in the following sections.

In-Plane Shear Capacity considering Out-of-Plane Loading

The in-plane shear capacity of specimen when no out-of-plane loading is applied is compared with the in-plane shear capacity when an out-of-plane load of magnitude $2\sqrt{f'_c}$ is applied at mid-height of the wall. Figure 8 presents the idealized in-plane force-displacement curves for these cases. It is observed that the reduction in in-plane capacity is below 5%. The reduction in capacity is negligible because the out-of-plane force is applied close to the base and the magnitude of out-of-plane load applied is not high. Therefore, the resulting out-of-plane moment is low.

IP Shear Capacity variation with change in magnitude of OOP loading

IP shear capacity of specimen (CNSC1) with out-of-plane load of magnitude $2\sqrt{f'_c}$ is compared with the capacity of specimen (CNSC2) with out-of-plane loading equal to the out-of-plane capacity of the SC wall pier (which is approximately twice the OOP loading for CNSC1). As observed in Figure 9, the increase in the magnitude of the out-of-plane loading causes a reduction of about 15% in the in-plane capacity of the SC wall pier.



a) IP+OOP Force b) IP Force

Figure 4. Initiation of failure in SC walls

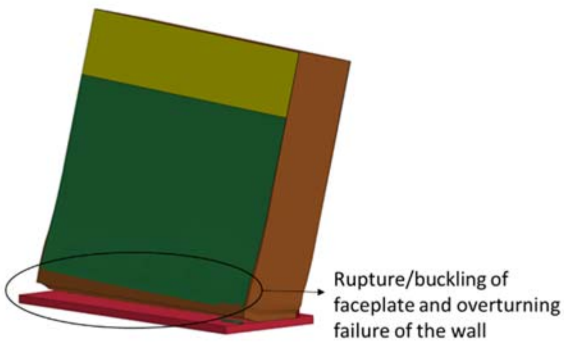


Figure 5. Typical failure of SC wall subject to IP+OOP Loading

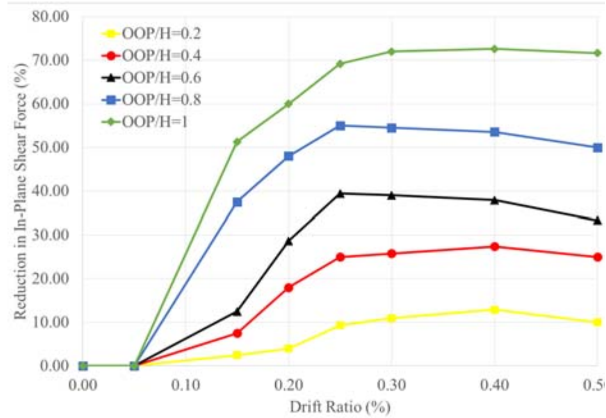


Figure 6. Reduction in IP force with location of OOP force

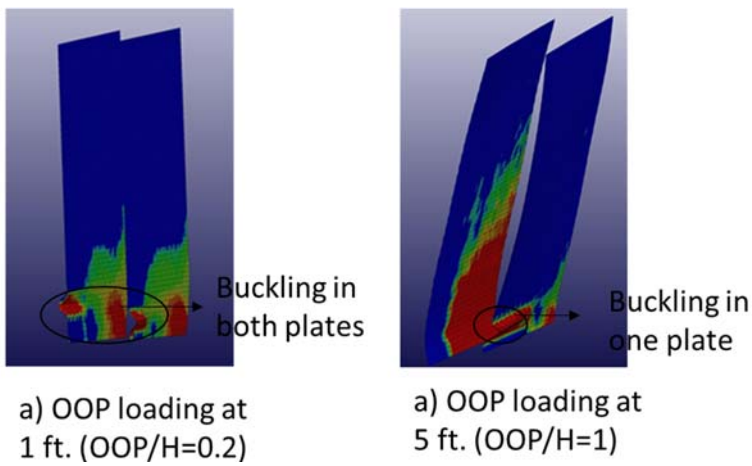


Figure 7. Behavior of SC wall piers at 0.5% drift ratio

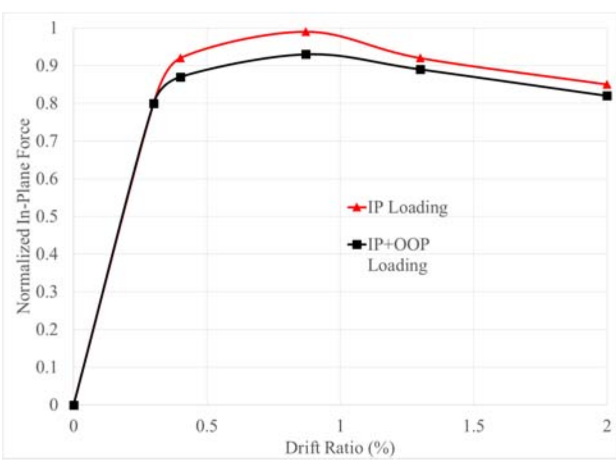


Figure 8. Idealized IP Force-Displacement Curve for CNSC1

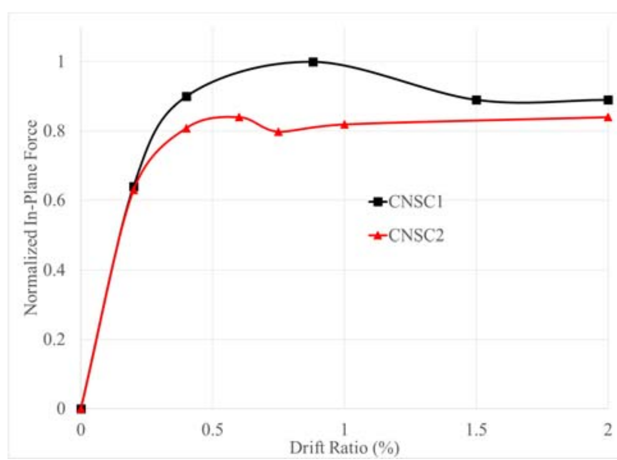


Figure 9. Idealized IP Force-Displacement Curve:CNSC1 &CNSC2

TEST MATRIX, SETUP AND LOADING PROTOCOL

Based on the SC wall pier behaviour and numerical evaluations discussed above, a test matrix for the experimental investigations is developed and is presented in Table 1. As shown, the specimens are all 12 in. thick, made with 3/16 in. steel faceplates, resulting in a plate reinforcement ratio of 0.031. All specimens have an aspect ratio of 0.6 and are shear critical walls. Control Specimen and Specimen CNSC1 have tie bar spacing of the wall thickness (12 in.), which can be representative of containment internal structures. Specimens CNSC2 and CNSC3 have tie bar spacing of half the wall thickness (6 in.). This ensures that the out-of-plane capacity of the wall includes contribution from steel and concrete both. Each wall has a foundation connection that will develop the capacity of the wall above and limits inelastic action to the wall. Plate slenderness ratio is kept at 16 for CNSC1 and CNSC2, and 21 for Control and CNSC3.

Control specimen will not be subject to any out-of-plane loading, CNSC1 will be subject to out-of-plane load of magnitude $2\sqrt{f'_c}$, and CNSC2 and CNSC3 will be subject to loading magnitude of out-of-plane shear capacity of the SC wall. Comparison of results from Specimens Control and CNSC1 will present the effect of out-of-plane loading on the in-plane capacity of the wall. The results from Specimens CNSC1 and CNSC2 will be compared to evaluate the influence of the magnitude of out-of-plane shear force on the in-plane behaviour of the wall. Comparing the results of specimens CNSC2 and CNSC3 will provide insight into the effects of shear stud and tie bar spacing on the in-plane capacity of the wall piers.

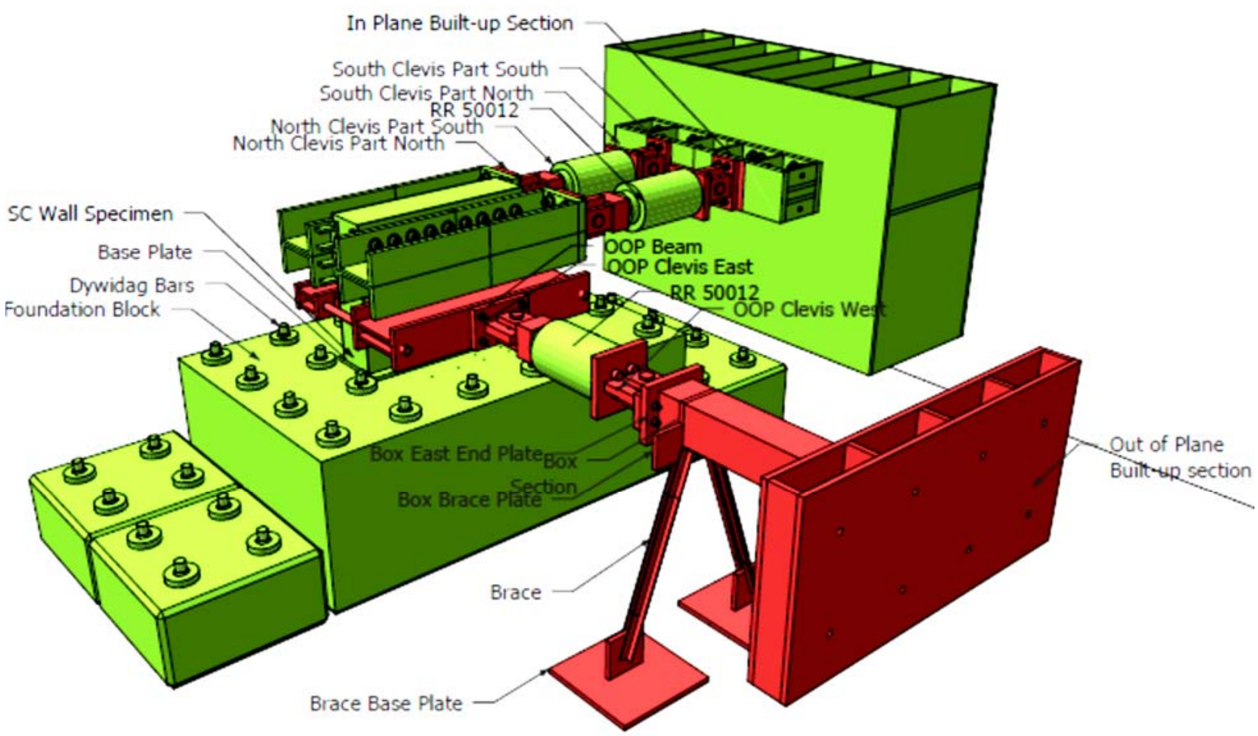
The test setup for the experimental investigations is presented in Figure 10. The out-of-plane set up consists of a 660 kip dual action actuator and out-of-plane beams to apply the loading on the specimen. The in-plane set up consists of two 660 kip dual action actuators and in-plane beams. In-plane (IP) and out-of-plane (OOP) clevis have been designed and fabricated to be able to accommodate in-plane and out-of-plane movement of the specimen. Specimens will be subjected to incremental cyclic out-of-plane loading till the concrete cracks. The out-of-plane force will then be kept constant at the value mentioned in Table 1, and subjected to incremental cyclic in-plane forces till the specimen fails. For brevity, the loading setup, loading protocol and instrumentation plan are not discussed in detail here.

Table 1. CNSC Test Matrix

Specimen #	Height (in.)	Length (in.)	Tie Spacing (in.)	Stud Spacing (in.)	OOP Force
CONTROL	36	60	12	4	N.A
CNSC1	36	60	12	3	$2\sqrt{f'_c}A_C$
CNSC2	36	60	6	3	$2\sqrt{f'_c}A_C + A_t\, f_{ty}$
CNSC3	36	60	6	4	$2\sqrt{f'_c}A_C + A_t\, f_{ty}$

Preliminary Experimental Results

Specimens Control and CNSC1 were tested recently. CNSC1 was subjected to 13 cycles of incremental in-plane loading while maintaining the out-of-plane loading constant. Cycle 13 applied an in-plane displacement of magnitude three times the yield displacement of the specimen (Fore brevity, the results of Control specimen are not discussed in detail here). Figure 11 shows the comparison of in-plane force displacement behaviour of specimens Control and CNSC1. As shown, the specimens demonstrated good hysteretic behaviour. The IP strength of both the specimen was approximately the same. There was no significant reduction in the in-plane shear capacity of the CNSC1 due to the application of out-of-plane loading. Both the specimens were pushed to a drift ratio of approximately 1.5%. The control specimen failed due to cyclic yielding of the steel faceplates leading to fracture in the base metal close to the weld. CNSC1 failed due to cyclic yielding of the steel faceplates and eventually compression failure and spalling of the concrete. Figure 12 shows the state of damage in the CNSC1 SC wall piers at the end of the test. The local buckling of the steel faceplates, and concrete cracking and spalling are shown at both ends of the wall pier in Figure 12. The response of the specimen CNSC1 is no longer symmetric, there is twisting of the specimen and considerable residual out-of-plane drift in the specimen. On the other hand, Control specimen failed symmetrically.



a) Model of test setup

b) Physical setup

Figure 10. CNSC Test Setup

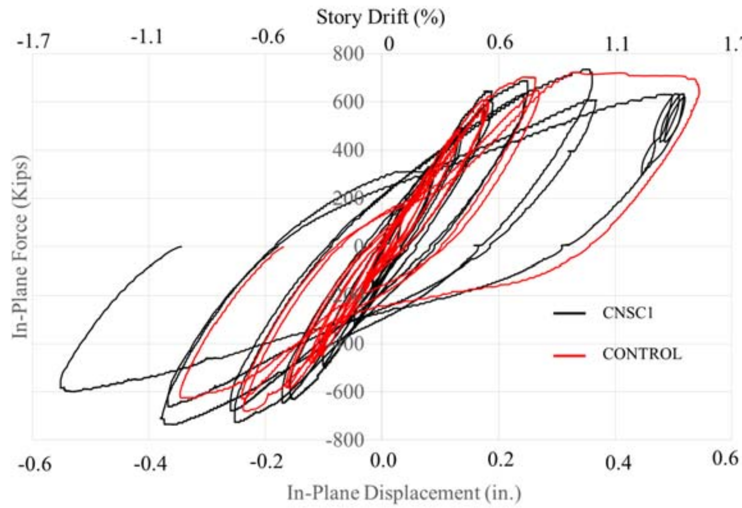


Figure 11. Comparison of In-plane force displacement behaviour (Specimens Control and CNSC1)

a) South Face

b) North Face

Figure 12. Specimen CNSC1 at end of test (zoomed in to bottom portion)

CONCLUSIONS AND DISCUSSION

The test matrix for the experimental study was developed based on the expected behaviour of SC wall piers and the results of preliminary numerical analysis. Experimental investigations were conducted using a specially designed and built test setup, which functioned as designed during the tests. The test results for the first two specimens (Control and CNSC1) indicate that out-of-plane loading equal to the design shear strength ($V_n = V_c$) does not reduce the in-plane shear strength of the SC wall pier specimen with height-to-length aspect ratio of 0.6. However, the response when out-of-plane loading applied is no longer symmetric, there is twisting of the specimen and considerable residual out-of-plane drift.

It is observed that the location and magnitude of out-of-plane loading affect the in-plane shear capacity of the SC wall piers significantly. Numerical results indicate that i) reduction in in-plane capacity can increase significantly (above 50%) as the out-of-plane loading location moves closer to the top, and ii) in-plane shear capacity is reduced by 15-20% as magnitude of out-of-plane loading is doubled (from concrete cracking threshold to out-of-plane shear capacity of the wall). The reduction in in-plane capacity due to change in magnitude of out-of-plane loading will be experimentally verified in the tests that follow.

The in-plane capacity is affected by the out-of-plane moment caused by the out-of-plane loading. The resulting out-of-plane moment causes longitudinal tensile and compressive stresses in the two faceplates. These stresses reduce the in-plane capacity of the wall pier. Therefore, the effect of out-of-plane loading can be considered by considering the additional stresses due to out-of-plane moments. The authors are working on developing an interaction equation for the in-plane and out-of-plane moments acting on the SC wall pier.

ACKNOWLEDGEMENT

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