

Preliminary Investigation of Steel-Plate Composite (SC) Wall-to-Concrete Basemat Anchorage Connections

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ABSTRACT

Steel-plate composite (SC) walls are being used in the third generation of nuclear power plants (AP1000®, US-APWR®, etc.). SC walls are also being considered for small modular reactors (SMRs) of the future. SC walls serve as the primary lateral force resisting system for the structure, and can be designed using Supplement No. 1 to AISC N690, which includes Appendix N9 focusing on the design of SC walls. This Appendix specifies that the SC wall-to-reinforced concrete (RC) basemat anchorage connection can be designed to: either (i) achieve the full strength of the SC wall, or (ii) 200% overstrength with respect to the calculated seismic demands.

This paper focuses on the design of an overstrength anchorage connection for SC walls. The connection consists of steel rebars (dowel bars) that are fully developed in the reinforced concrete (RC) basemat, and embedded in the SC walls. There is significant interest in this type of dowel bar anchorage connection because of ease of construction and similarity with previous connection designs for RC walls. However, there is insufficient information in the literature regarding the lateral load behavior, strength, and ductility of SC walls anchored to the RC basemat using dowel bars. This paper presents the initial results from a research project focusing on this particular connection. The paper includes (i) the numerical analysis and design of SC wall-to-RC basemat dowel anchorage connections, and (ii) the development of test matrix and testing approach.

INTRODUCTION

Steel-plate composite walls consist of two steel faceplates with concrete infill. The steel faceplates are connected to each other using tie bars. These tie bars are embedded inside the concrete infill, and act as transverse reinforcement for the wall. The behavior of SC walls have been studied by different researchers. SC walls are being used for safety-related nuclear facilities due to their structural efficiency (Varma et al. 2011), construction schedule and economy (DOE 2006), and resistance to impactive and impulsive loading (Bruhl et al. 2015). Kurt et al (2015) presented the results of eight SC wall piers (without boundary elements) with different aspect ratios (wall height/wall length), reinforcement ratio (total faceplate area/wall thickness), and connector spacing (tie bars, and shear studs). Researchers concluded that the in-plane capacity of SC wall piers increases with increasing wall thickness and decreasing aspect ratio, and the spacing of the connectors on the faceplates influenced the post-peak behavior of the specimens. It was reported that all SC wall piers exhibited flexural behavior, and the failure mechanism was buckling of the steel faceplates around corners of the wall at base, followed by crushing of concrete under compression, and rupture of the steel faceplates under tension. Currently, there is lack of data for SC wall to basemat connections. This paper presents the initial results from a research project focusing on this particular connection.

SC walls can be anchored to the concrete basemat in three possible ways; (i) embedding dowel rebars inside the concrete basemat, (ii) embedding dowel rebars inside the wall and the basemat, or (iii) embedding the

steel faceplates of the wall inside the concrete basemat. This paper presents results from (i) the numerical analysis and design of SC wall-to-RC basemat dowel anchorage connections, and (ii) the development of test matrix and testing approach.

The first connection method listed above requires welding of the faceplates to a base plate. A series of weldable couplers are welded under the base plate. Dowel rebars are then attached to the welded couplers. These dowels are embedded inside the concrete basemat. This method is practical and currently being used in the design of third generation nuclear power plants. Majority of the connection can be assembled in the fabrication yard. The weldable coupler-dowel rebar connection is generally employed for full-strength connections. A full strength connection is designed to ensure that the SC wall fails before the connection. However, this may lead to large force demands for connection and is very conservative.

Embedding dowel rebars is another way of anchoring the SC walls to the concrete basemat. This anchoring method consists of embedding the reinforcement bars inside the concrete infill of the SC wall and concrete basemat. Dowel bars are embedded into the concrete basemat, followed by placing the SC module (without concrete infill) over the dowels. Concrete is then poured into the SC module. The design of this connection is generally done using the overstrength design approach which requires the connection to resist 200% of the calculated seismic demands. The focus of this paper is to discuss the design, construction, and testing of this type of connection. The third type of connection, which involves embedding the steel faceplates inside the concrete basemat, is not considered practical due to the complicated connection detailing. This connection type is not discussed in this paper.

DESIGN OF DOWEL REBAR CONNECTION SPECIMENS

The purpose of the tests is to furnish information on the in-plane behavior of dowel rebar-SC wall connections. Two large size specimens were constructed to investigate the change in in-plane behavior of SC walls with different size, and number of the dowel rebars. There are three main components of the test specimens; (i) reinforced concrete (RC) foundation, (ii) SC wall, and (iii) dowel rebars. The details of the specimens are discussed below. The specimens have vertically aligned reinforcement bars embedded inside the concrete foundation, and the modular SC walls are placed in such a way that the portion of the dowel reinforcement bars projecting from the RC foundation are encased by the SC wall pier. SC wall specimens in this study have typical rectangular cross section as discussed in Kurt et al (2015). The shear stud and tie bar configuration has been specifically designed for this type of connection. The bottom portion of the wall pier uses only tie bars to i) prevent potential delamination failure, and ii) achieve the composite action between the faceplates and concrete. Aspect ratio of the specimens was selected as 0.6 to maximize the shear forces experienced by the wall at the base.

RC foundation is designed to resist the expected shear forces, and resulting bending forces during the test. The foundation reinforcement is detailed to ensure that it does not interfere with dowel rebars. There are several 1-1/2 in. vertical through holes in the foundation for post-tensioning the foundation to the laboratory strong floor and preventing the foundation from sliding during the tests. The first specimen had 30-#5 dowel rebars, and the second specimen had 30-#6 dowel rebars. Figure 1 shows the layout of the dowel rebars and the cross section details of the SC wall pier specimens. The spacing of the dowel rebar was 4 in. for both specimens. The in-plane length of the wall piers was 60 in, and the thickness of the wall piers was 12 in.

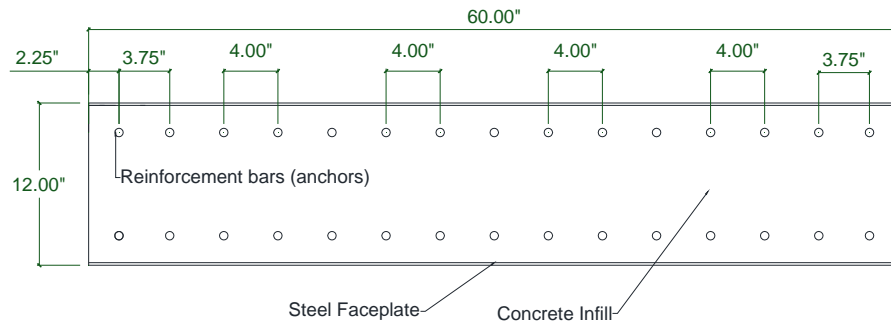


Figure 1. Layout of the reinforcement bar anchors on the cross section of Specimen#1 and Specimen#2.

The in-plane behavior of the SC wall specimens was assumed to have two different mechanisms; (i) SC wall pier behavior as explained in Kurt et al (2015), and (ii) typical RC wall behavior. The aspect ratio, wall thickness, and connector element spacing influence the SC wall pier behavior. Dowel rebar size and spacing, and wall thickness and length influence the in-plane behavior of the RC portion of the specimens. The yield strength of the steel faceplates was 52 ksi. SC wall pier infill concrete strength was taken as 6 ksi in the design calculations. The in-plane flexural capacity of the SC portion (V_{n-SC}) of the wall pier was calculated as 2020 kip-ft. using the empirical equation developed by Kurt et al (2015).

Grade 60 reinforcement bars were used as dowel rebars. Expected yield strength of the dowels (75 ksi, 1.25x60ksi) was used in the moment-curvature analysis of the specimens. Tension coupon tests of the reinforcement bars showed an average tensile yield strength of 76 ksi and 75 ksi for #5 and #6 dowel rebars respectively. The in-plane flexural capacities of the RC portion of the specimens were calculated using moment curvature analysis. The lateral load capacities from the moment curvature analyses (V_{n-RC}) are 520 and 670 kips for Specimen#1 and Specimen#2 respectively. ACI 318-08 (ACI, 2008) Ch. 21 Section 9.4.4 limits in-plane shear strength of the individual wall piers to $10A_c\sqrt{f'_c}$. The limiting lateral load capacity (V_{n-318}) from ACI-318 was calculated as 540 kips with the given cross section and material properties.

ACI 318-08 Ch.11 Section 6.4 discusses the shear friction method in RC members. Shear-friction assumes that the reinforcement bars are extending across a joint or shear plane, and shear transfer occurs across two planes via friction between the surfaces. The friction force is a result of the clamping forces created by the reinforcement bars keeping the two surfaces together. Shear friction capacities (V_{n-sf}) of Specimen#1 and Specimen#2 were calculated as 560 and 790 kips respectively. The friction coefficient was taken as 1.0 as recommended for intentionally roughened surfaces. The shear friction capacities of the both the specimens were greater than the in-plane flexural capacities of the RC portions (V_{n-RC}) of the specimens. ACI 318 Section 11.6.5 limits the shear-friction capacity to $(480 + 0.08f'_c)A_c$, where f'_c is considered as 6 ksi, and A_c is the total infill concrete cross section area. The limiting capacity for shear-friction method (V_{n-sfl}) was calculated as 560 kips, and it was greater than the strength of the individual piers (V_{n-318}).

Table1 shows the comparison of theoretical in-plane capacities for Specimen#1. This specimen was designed to have V_{n-RC} to be lesser than the other theoretical capacities. Table 2 shows the comparison of theoretical in-plane capacities for Specimen#2. The specimen was designed to have V_{n-sfl} lesser than the other theoretical capacities. The failure mechanism of Specimen#2 is assumed as sliding shear.

Table1. Comparison of lateral load capacities for Specimen#1

Limit state	Lateral load capacity	Force (kips)
Yielding of the rebars	V_{n-RC}	521
Yielding of the faceplates	V_{n-SC}	673
Sliding failure	V_{n-318}	540
Shear friction	V_{n-sf}	552
Shear friction upper limit	V_{n-sfl}	560

Table2. Comparison of lateral load capacities for Specimen#2

Limit state	Lateral load capacity	Force (kips)
Yielding of the rebars	V_{n-RC}	669
Yielding of the faceplates	V_{n-SC}	673
Sliding failure	V_{n-318}	540
Shear friction	V_{n-sf}	795
Shear friction upper limit	V_{n-sfl}	560

ACI 318-08 Ch. 12 Section 2 describes the development of deformed bars in tension. The general development length equation (12-1) is given as follows:

$$l_d = \frac{3}{40} \frac{f_y}{\sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)} d_b$$

Bars with minimum clear cover not less than $2d_b$ and minimum clear spacing not less than $2d_b$ and without any confining reinforcement have a $(c_b + K_{tr})/d_b$ value of 2.5. ψ_s is taken as 0.8 for #6 and smaller deformed bars. ψ_e , and ψ_t were taken as 1.0. Considering the yield strength of the reinforcement bar, f_y , and compressive strength of concrete, f'_c , as 60 ksi and 6 ksi respectively, the development length is $23d_b$.

Using Equation 12-1 of ACI 318, the development lengths for #5 and #6 reinforcement bars were 14 in and 17 in. respectively. ACI Ch. 21 Section 9 recommends to apply a 1.25 multiplier in calculating the development length of the reinforcement in structural walls. Thus, the development lengths were increased to 18 in. and 22 in. for Specimen#1 and Specimen#2 respectively.

The steel faceplates are typically thin and susceptible to local buckling when walls are subjected to compressive stresses due to in-plane flexure. Local buckling is restrained by the shear studs and tie-bars. The recommendations of Zhang et al. (2014) and AISC (2014) were used to design the spacing of shear studs to prevent local buckling of the steel faceplates before compression yielding. The spacing of the connectors for the SC portion is calculated as 4 in. based on the recommendations by Zhang et al (2014). Special detailing was applied at the RC portion of the specimen until the termination of the reinforcement bars. All threaded rods were used in the RC portion of the specimens in order to prevent any loss of engaging with the reinforcement bar anchors during the tests under relatively high drift ratios.

FINITE ELEMENT ANALYSES

Finite element models (FEMs) were developed in LS-Dyna for initial assessment of the test specimens. The dimensions of the wall, tie bar spacing and distribution, anchorage detailing were consistent with the details

of the specimens discussed in the previous sections. Reduced integration solid elements for concrete infill, fully integrated shell elements for steel plates, and 2x2 Gauss quadrature beam elements for tie bars, shear studs and rebar anchors were used. The interaction between the wall pier and the concrete basemat was defined using the 'automatic_surface_to_surface' contact model. This contact model permits the sliding of the wall pier over the concrete basemat. The coefficient of friction was taken as 0.8. Elastic-perfectly plastic material model with yield strength of 52 ksi was used for steel plates. Continuous surface cap model (material model 159) was used for concrete infill and basemat. Uniaxial compressive strength of the concrete basemat model was considered as 6 ksi. Figure 2 shows details of the model developed for the Specimen#1.

Figure 3 shows the developed equivalent RC model by substituting the steel faceplates with horizontal reinforcement bars embedded inside the concrete infill. The equivalent RC model was built to compare its in-plane behavior with the proposed SC wall pier specimens. Horizontal reinforcement ratio of the RC model was equivalent to the SC model (Figure 2). The dowel rebars were extended all the way up to the height of the wall, and the depth of the anchor length inside the foundation was same as in the SC wall pier specimens.

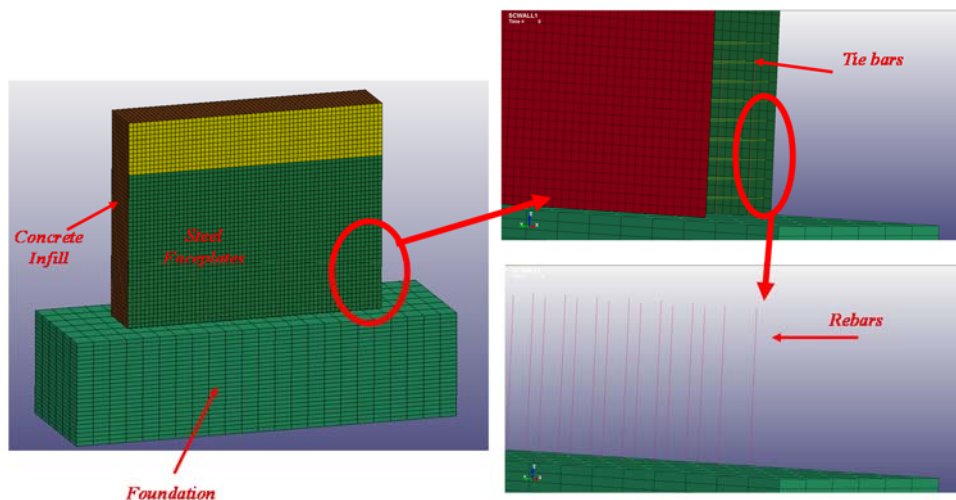


Figure 2. Finite element model details developed for Specimen#1

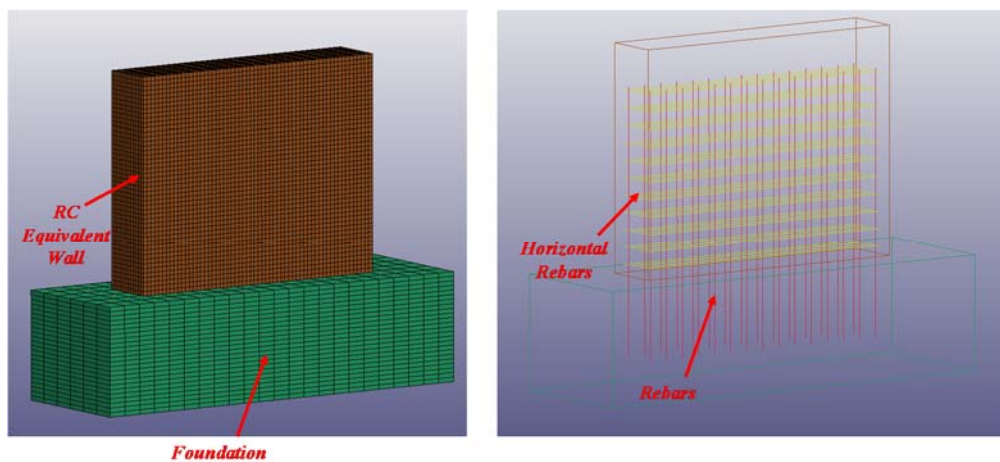


Figure 3. Details of the equivalent RC model developed in LS-Dyna

Figure 4 shows the lateral load-drift ratio comparison of the Specimen#1 and its equivalent RC model under monotonic loading. The walls do not reach their expected lateral load capacities obtained from moment-curvature analysis. They both undergo sliding shear failure. FEMs for Specimen#1 predicted the failure of the extreme tension dowel rebar (vertical reinforcement) to occur at 450 kips. The dowel rebar exceeds 15% plastic strain at this load level.

Figure 5 show the lateral load-drift comparison for Specimen#2 and its equivalent RC model. The Specimen#2 FEM lateral load capacity matches V_{n-RC} . The FEM capacity exceeds V_{n-318} . However, extreme dowel rebar under tension reaches 15% plastic strain at 650 kips lateral force. Sliding shear failure is expected for this specimen in the test.

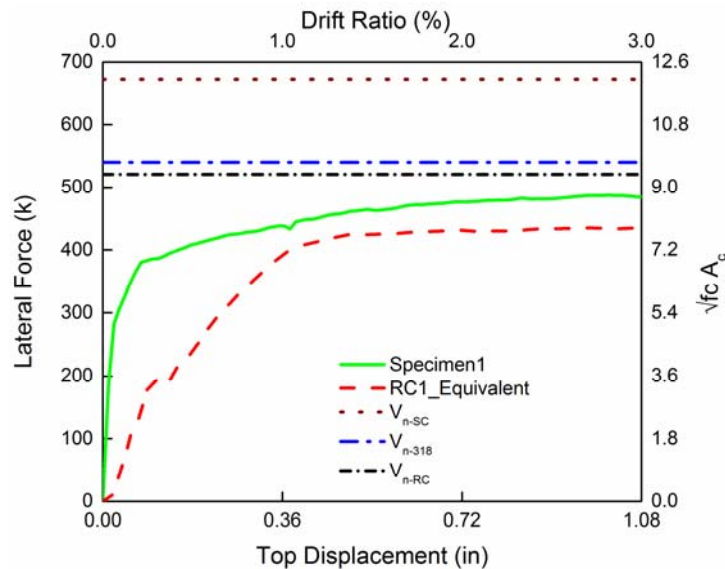


Figure 4. Lateral load-drift comparison for Specimen #1 and equivalent RC wall

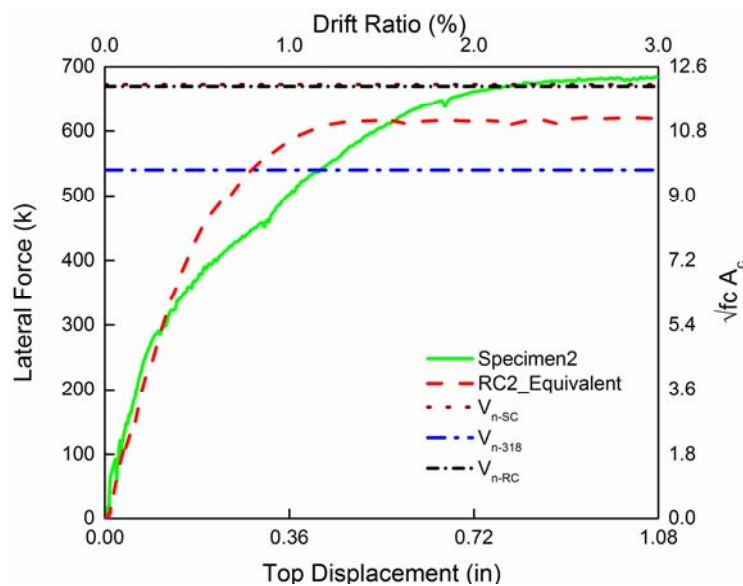


Figure 5. Lateral load-drift comparison for Specimen #2 and equivalent RC wall

CONCLUSIONS

This paper discussed the design of dowel rebar connection for SC wall-to-concrete basemat. The flexural capacity of the RC portion of the specimens was calculated by moment-curvature analysis. ACI 318 limitations, V_{n-318} , and shear friction capacity, V_{n-sf} , for Specimen#1 were greater than the flexural capacity of the specimen, V_{n-RC} . ACI upper limit for the individual wall pier strength, V_{n-318} , was lesser than the shear-friction, V_{n-sf} , and flexural capacity, V_{n-RC} , of Specimen#2. Although the first specimen was theoretically a flexure dominated wall, both specimens are expected to fail in shear. The connector elements (shear studs and tie bars) inside the specimens were detailed to prevent premature buckling of the steel faceplates.

3D finite element models of the designed specimens were developed in LS-Dyna. The results show that the Specimen#1 does not reach its flexural capacity, Specimen#1 fails by sliding and crushing of infill concrete. Specimen#2 reaches its flexural strength. However, it is expected that the wall pier will fail by sliding, and the capacity of the specimen will be very close to V_{n-318} .

REFERENCES

- ACI 318 (2008), "Building Code Requirements for Structural Concrete and Commentary" American Concrete Institute, Farmington Hills, MI
- U.S. Department of Energy. Accident analysis for aircraft crash into hazardous facilities (DOE-STD-3014). Washington, D.C.: U.S. Department of Energy;
- Bruhl, J., and Varma, A.H. (2015). "Missile Impact Behavior and Design of Composite SC Walls." International Journal of Impact Engineering, Elsevier Science, Vol. 75, pp. 75-87, <http://dx.doi.org/10.1016/j.ijimpeng.2014.07.015>, 1st Tier.
- Kurt, E.G., Varma A.H., Epackachi S., and Whittaker A.S. (2015). "Rectangular SC Wall Piers: Seismic Behavior and Design." ASCE Structures Congress, Portland, Oregon, USA.
- Varma, A.H., Zhang, K., Chi, H., Booth, P. and Baker, T. (2011). "In-plane shear behavior of SC composite walls: theory vs. experiment." Proceedings of the 21st IASMiRT Conference (SMiRT 21), New Delhi, India, Paper ID #764.
- Zhang, K., Varma, A.H., Malushte, S., and Gallocher, S. (2014). "Effects of Shear Connectors on the Local Buckling and Composite Action in Steel Concrete Composite Walls." Nuclear Engineering and Design, Special Issue on SMiRT-21 Conference, Vol. 269, pp. 231-239, Elsevier Science.<http://dx.doi.org/10.1016/j.nucengdes.2013.08.035>