TESTS OF REINFORCED CONCRETE SHEAR WALLS WITH CONFINED BOUNDARY ELEMENTS USING WELDED WIRE REINFORCEMENT

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Abstract

Welded wire reinforcement (WWR) grids, prefabricated for structural applications, can be used as an alternative to conventional transverse reinforcement in earthquake-resistant construction. WWR grids provide required transverse reinforcement in boundary elements of concrete shear walls with advantages of easy cage assembly, speed and precision of construction, and increased strength. However, experimental evidence of acceptable seismic performance of shear walls utilizing WWR is scarce in the literature. The paper presents the results of a combined experimental and analytical investigation using a 4.5m tall large-scale reinforced concrete shear wall with WWR grids used in the boundary elements as transverse reinforcement. The wall was tested under a constant axial load and incrementally increasing lateral deformation reversals until a significant strength decay is obtained. The wall cross-section had a barbell shape with 100 mm web thickness and 250 mm square boundary elements. The grids used were produced from 9.5 mm (3/8 in) diameter wires welded together to form 4-cell grids with a square center-center dimension of 205 mm. They were spaced at 75 mm in the boundary elements. Material testing was conducted to obtain the stress strain relationships of concrete and steel used. Extensive testing of WWR grids was carried out to assess the performance of grid welds. It was found that the grids suffered from premature weld failure near the wire yield strength, but at a stress level below the wire yield strength. The wall showed ductile response with a displacement ductility ratio of 4.6 and a drift capacity of 2.3% before it started developing a rapid strength decay during the second cycle at 2.8% drift. The wall was analyzed using finite element software under the same reversed cyclic loading applied during the test. An analytical model was developed using the material stress-strain characteristics obtained from the experimental program. An analytical parametric investigation was carried out with different grid weld failure capacities. The wall response was successfully reproduced using the analytical model, which indicated the onset of grid weld failure at about 90% of the wire yield strength. This observation, along with the weld performance in the WWR material tests, indicated that a caution should be exercised in using WWR in earthquake-resistant shear walls. It is recommended to conduct quality control tests on WWR prior to their use in practice. The burst tests employed in the current research program is recommended prior to their application with the grid strength obtained from such tests. If there is a consistent pattern of steel wire rupturing prior to a weld failure, the full capacity of grids can be used in design. Otherwise, a reduced capacity may be employed in design with a satisfactory outcome in wall behavior.

Keywords: concrete shear walls; ductile shear walls; seismic resistant walls; welded wire reinforcement.

1. Introduction

Seismic resistant reinforced concrete shear walls in buildings are designed to provide strength, stiffness, ductility and lateral drift control during strong earthquakes. Shear wall deformability is largely dependent on adequate confinement of core concrete in boundary elements, prevention of longitudinal bar buckling, and proper web design against brittle shear failure. Conventional transverse reinforcement in shear wall boundary elements consists of hoops, overlapping hoops and crossties. The confinement steel requirement for seismic design often results in congestion of steel cage with potential concrete placement problems.

An alternative to conventional reinforcement is welded wire reinforcement (WWR) grids prefabricated for structural applications. WWR grids can be used to provide required transverse reinforcement in the boundary elements with easy cage assembly, speed and precision of construction, and increased strength. However, research on the use of WWR is limited in the literature. Thompson et al. [1] studied the behavior of twelve concrete column samples representing shear wall boundary elements, incorporating MEDO-MESH modules as transverse reinforcement [2]. The columns were loaded under monolithically increasing axial loads up to failure. The research helped examine the hysteretic behavior of shear wall boundary elements using the same type of reinforcement. The test results indicated that the confinement effectiveness of the MEDO-MESH modules was equal to or greater than that of the traditional deformed closed hoops in strain ranges of up to 0.02. Miranda et al. [3] conducted experimental research on shear walls using prefabricated welded wire hoops as transverse reinforcement in boundary elements. The experiments included six full size shear walls with various transverse reinforcement ratios, sizes and spacings of reinforcement. The volumetric ratio of transverse reinforcement varied between 0.48% and 2.0%. The wall specimens were selected to have a minimum height to length ratio of 3.0. The yield strength of welded wire reinforcement ranged between 540 MPa and 560 MPa depending on the wire size, where smaller wires showed higher yield strength. Four of the samples failed due to global buckling, reaching lateral instability at the end of testing. Based on the observations made during the tests, the researchers attributed these failures to the significant axial load on longitudinal bars prior to the closure of residual cracks remaining from the previous loading cycle in the opposite direction. It was also observed that the deformability of specimens were highly dependent on the strength of the resistance welds at MEDO-MESH joins. The failure of MEDO-MESH modules was observed to occur at the resistance welds or in the heat affected zones near the welds. It was recommended that, since the effectiveness of these modules as transverse reinforcement was dependent on the development of the nominal yield strength of the wires, proper quality control of the manufacturing process should be maintained to ensure that the strength of the welds would be greater than the tensile strength of the wires. Further experimental and analytical research was suggested to establish design requirements for such reinforcement.

Saatcioglu and Grira [4, 5] conducted experimental research at the Structural Laboratory of the University of Ottawa involving WWR grids as transverse column reinforcement in which they used WWR grids developed by BauMesh Company [6]. A total of 10 full-scale 350 mm x 350 mm square concrete columns were tested. The columns had volumetric ratios of transverse reinforcement ranging between 1.00% and 2.66%. The grids had a center to center dimension of 292 mm x 292 mm, and were supplied in 4 and 9 cell configurations. The wires used in the grids had a yield strength of 570 MPa or 580 MPa for wires having 9.53 mm and 6.60 mm diameters, respectively. The columns were subjected to either 20% or 40% of their concentric load capacity, accompanied by incrementally increasing deformation reversals. They showed ductile response, equivalent to those reinforced with conventional tie reinforcement. The material tests conducted indicated that the strength and ductility of steel wires forming the grids were not adversely affected by the resisting weld process. The column tests showed no grid failure prior to rupturing of the longitudinal bars in tension.

The current research has the objective of investigating the effectiveness of WWR grids in flexure-dominant shear walls as boundary element transverse reinforcement using the same type of WWR used in the column tests by Saatcioglu and Grira [4, 5]. A large-scale reinforced concrete shear wall was designed, built and tested under constant axial load and incrementally increasing reversed cyclic deformations.

Material tests were conducted to assess the strength and ductility of the WWR grids used in the boundary elements. Analytical research was also undertaken to expand experimental results by considering different grid capacities. The details of the experimental and analytical research are presented and discussed in the following sections.

2. Experimental Research

A barbell shaped reinforced concrete shear wall with a height to length aspect ratio of 3:1 (4.35 m height: 1.45 m length) was designed, constructed and tested under simulated seismic loading. The boundary elements had 250 mm square sections. The web thickness was 100 mm. High-strength concrete with f'_c = 83 MPa and Grade 400 MPa conventional deformed reinforcement was used (with a measured yield strength of 460 MPa), except for the boundary element transverse reinforcement, which consisted of BauGrid [6] as WRG. The grids used were produced from 9.5 mm (3/8 in) diameter wires welded together to form 4-cell grids with a square center-center dimension of 205 mm. The wall elevation and cross-sectional views are shown in Figs. 1 and 2, respectively.

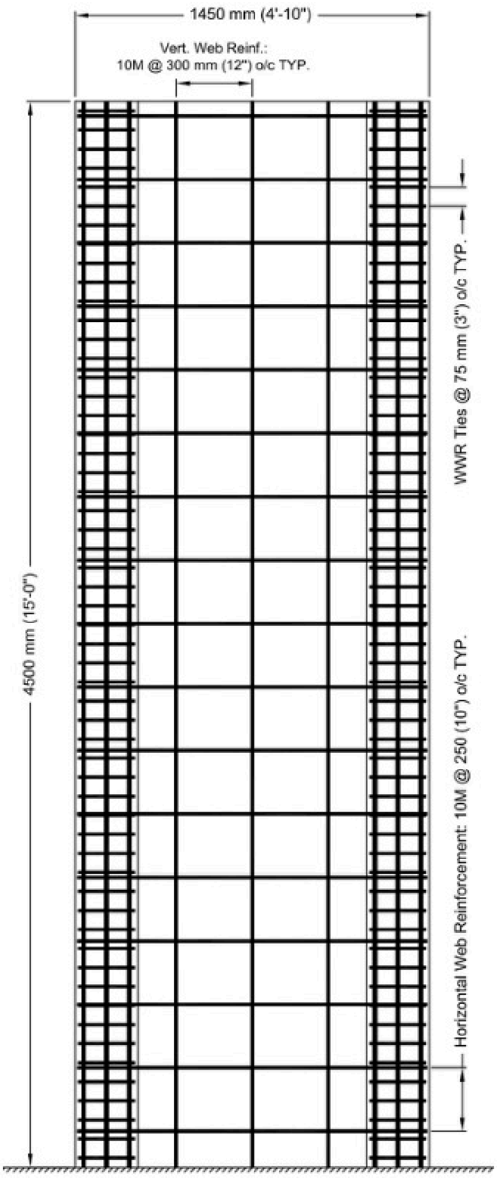

Fig. 1 Elevation view of wall specimen

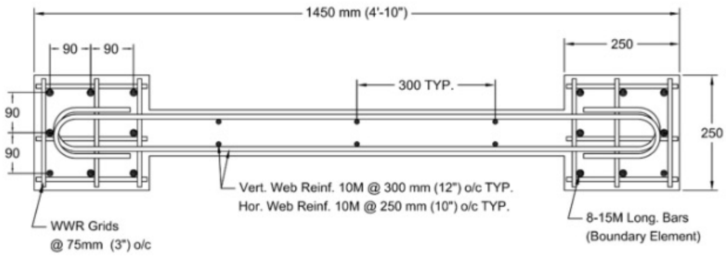

Fig. 2 Wall cross-section

The wall was designed to prevent premature shear failure and promote flexural response, as per current modern building code requirements. It was tested under a constant axial load 1500 kN, which was applied using vertical prestressing strands. The wall was fully fixed to the laboratory strong floor by means of a 500 mm thick rectangular concrete foundation with nominal dimensions of 3300 mm x 1500 mm x 500 mm, which was attached to the floor by means of 75 mm bolts. The test set-up and the wall prior to testing are shown in Fig. 3.

Fig. 3 Test setup

Extensive testing of the grid material (BauGrid WRG) was conducted to assess its strength and deformability. The 9.5 mm (3/8 in) wire used for manufacturing the grids was first tested using a standard ASTM coupon test procedure. Two different types of coupons were used; samples taken between the welds and those included a welded joints in the test region. Both types of coupons produced similar results indicating an average yield strength of 550 MPa (based on 0.2% offset method) and an ultimate rupturing strength of 640 MPa. The maximum elongation measured at rupture ranged between 4.15 % and 4.55 %. Fig. 4 shows the recorded stress-strain relationship and the failure modes for the two types of coupons tested.

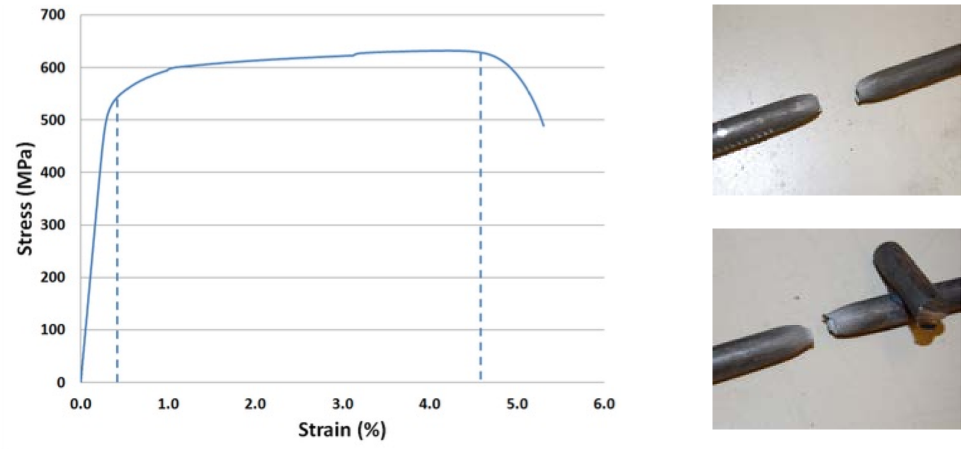

Fig. 4 Stress-strain relationship of the wire used in WWR grids and their failure locations

Additional material tests were conducted on grids having different number of cells to assess the weld capacity. They included joint shear capacity tests following the ASTM 1064 [7] procedure, and a series of tests that are believed to emulate the loading condition that can be generated by the longitudinal compression reinforcement in concrete, restrained by grid corners as they force the grids laterally with a tendency to buckle. The latter tests were conducted on grids embedded in concrete prisms. They were either “direct shear tests” or “burst tests.” The samples had 25M re-bars (having 25.2 mm diameter) cast in concrete to engage in exterior grid corners as shown in Figs 5 and 6. The direct shear tests involved pulling the 25M re-bars at opposite corners apart, in the longitudinal grid direction. The burst tests involved pulling the grids diagonally. Figs. 5 and 6 show the details of direct shear and burst tests, respectively.

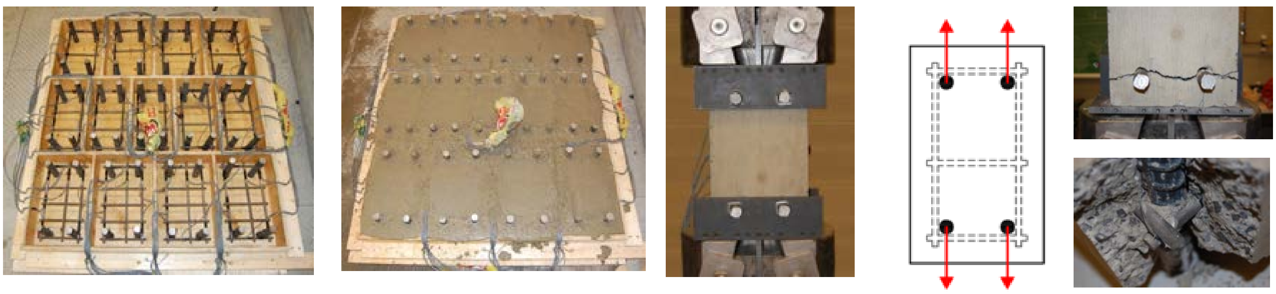

Fig. 5 – Direct shear tests on grids embedded in concrete prisms and pulled parallel to the longitudinal grid direction and the failure of grid welds

The results indicated that the grid welds in all cases failed prior to the yielding of the wires. The strain gauge readings and visual observations confirmed that the applied force was not shared equally by the two parallel grid legs, and the failure occurred in one grid corner only. Assuming equal distribution of forces, the failure occurred between 52% and 64% of the wire yield strength. At failure, it is possible that one of the two grid legs may be resisting higher percentage of yield strength. In these tests, the force was not applied directly to a joint in the diagonal direction, with the 25M bars mostly bearing on transverse grid legs, which may have created some eccentricity of load on the weld, thereby reducing the grid capacity. Somewhat higher failure forces were recorded in burst tests with the load directly applied to the welded joints. Fig. 6 shows a typical burst test. The forces resisted by grid legs in the longitudinal direction (axial force component in the direction of longer grid legs) indicated failure of grid welds at 66% to 90% of the steel yield force. The weld capacity was not high enough to cause yielding in the wires. These findings are different than the earlier batch of BauGrid WRG [6] used in the column study conducted by Saatcioglu and Grira [4, 5]. The grids used in the columns consistently failed by rupturing of the grid legs (wire), rather than the grid welds. Extensive in-air burst tests (without the surrounding concrete) showed that the welds were strong enough to ensure the yielding of the grid legs. Fig. 7 shows examples of in-air burst tests, which are believed to create more severe conditions on the welds because of lack of restraining concrete. The details of grid behavior under different test conditions are discussed elsewhere [8].

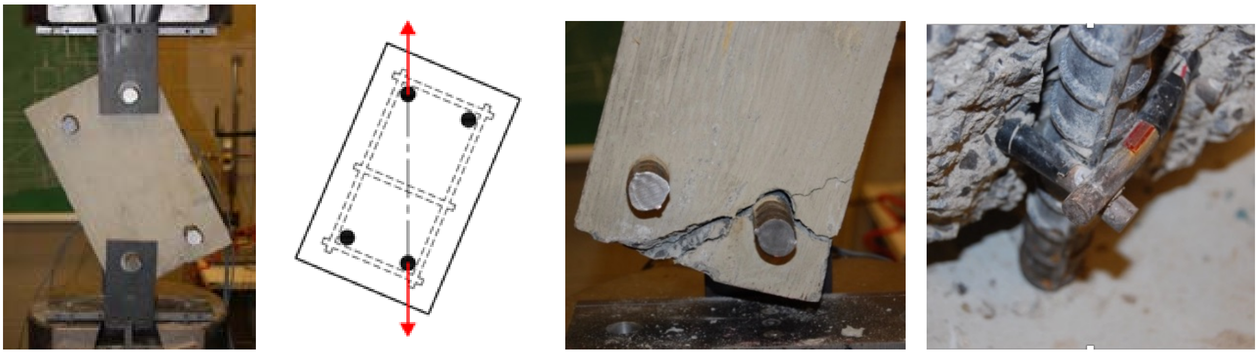

Fig. 6 – Typical burst test and grid weld failure

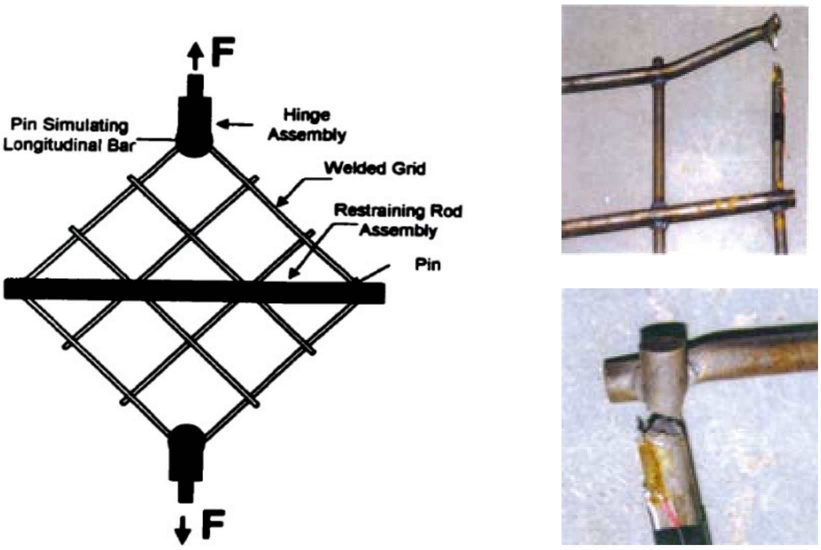

Fig. 7 – Burst tests of BauGrid WWR used in column tests with failure by rupturing of wire outside the weld[4, 5, 9].

The gird joints were also tested to establish the joint shear capacity as per ASTM 1064. In this case a joint was subjected to direct shear stresses as depicted in Fig. 8. During testing, the machine head provided continuous support underneath the horizontal member of the joint while the other head applied direct shear on the joint as illustrated in Fig. 8. The same figure also shows a typical test sample before and after failure. The results of ASTM tests indicated weld shear failure at 77% to 82% of the wire yield capacity. This level of resistance met the ASCE requirement for joint shear capacity (241 times the area of wire in Newtons) and exceeded it by approximately 50%. The ASTM joint capacity corresponds to between 77% and 82% of the wire yield strength. This is consistent with the earlier burst tests, and may be used as grid capacity for the design of confinement reinforcement in place of the yield strength of steel as typically done, unless it is shown by quality control tests that the grid wire is able to develop its yield strength prior to weld failure.

Fig. 8 – ASTM welded wire reinforcement weld tester [7, 9]

The wall with the WWR grids as boundary element transverse reinforcement showed ductile behavior when tested under inelastic deformation reversal. Hysteretic lateral force - top lateral displacement (and drift) relationship is shown in Fig. 9. The forces plotted are the net forces resisted by the wall, account for the effect of the horizontal force component of vertical prestressing cables used for axial load application. The failure occurred due to compression bar buckling in the boundary element during the second cycle towards 2.8% lateral drift. At this stage of loading, the LVDT used for measuring horizontal displacements dislodged from the instrumentation frame. The dotted line in the figure indicates the observed decay in strength as recorded by the actuator load cell.

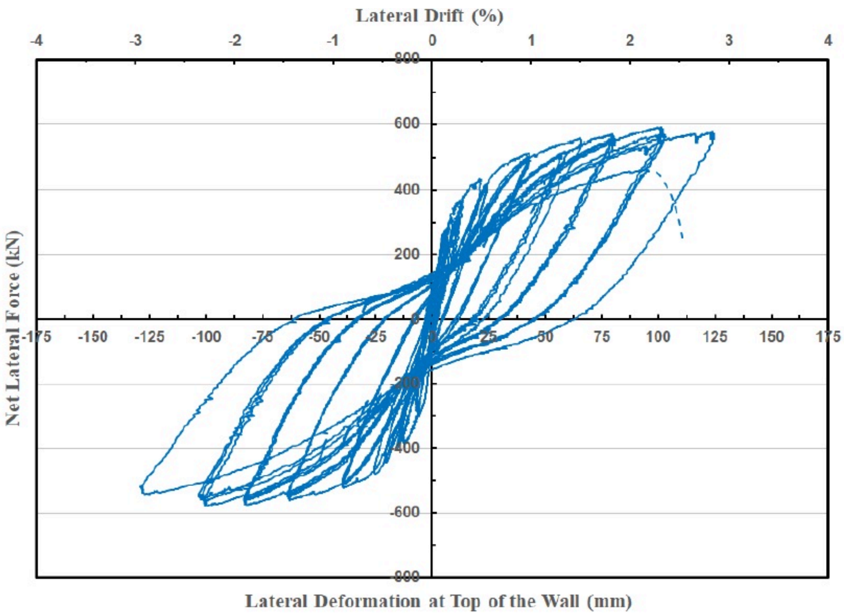

Fig. 9 – Hysteretic relatiship of wall as recorded during the test

The wall was able to resist three cycles at 2.3% drift without any loss of strength. The strain gauges placed on grids indicated a maximum strain of 0.34%, with some of the grids near the base showing variations between 0.2% and 0.3% strains. This is at or near the grid wire yield strength. It is clear that the grids performed well and experienced near yield stresses. At the bar buckling deformation level one of the grid welds failed. Subsequently, as the wall continued experiencing strength decay beyond 3.5% lateral drift two more grids experienced weld failure. The wall at the end of the test is shown in Fig. 10.

Fig. 10 – Shear wall at failure and observed damage to concreet and WWR grids

3. Analytical Research

Analytical research was conducted to expand the experimental results and develop a design procedure for using WWG as boundary element transverse reinforcement. The shear wall specimen tested was modelled in VecTor2 [10] finite element analysis software. The software provided capabilities for modelling non-linear behavior of reinforced concrete elements under reversed cyclic loading while incorporating shear-flexure interaction. This capability of the analytical tool was of utmost importance, as shear walls are typically subjected to significant shear stresses even if the walls are designed to fail in flexure. The analytical research was divided into two phases: i) Validation of the analytical model and ii) Analytical parametric investigation.

The shear wall was modelled in VecTor2 using the material properties established through standard material tests. The model included three segments: i) Foundation, ii) Loading beam, and iii) Shear Wall. The steel reinforcement was introduced as smeared reinforcement with the exception of longitudinal bars, which were modelled individually. Web horizontal reinforcement was modelled as smeared reinforcement having 0.8% steel and extended into the boundary elements. Web vertical reinforcement was modelled as individual unconfined bars with an unsupported length to diameter (b/t) ratio of 25. The horizontal and vertical WWR grid legs were modelled to have 1.14% and 1.58% steel, respectively. The boundary element longitudinal bars were modelled as confined bars with an unsupported length to diameter (b/t) ratio of 5. The cover concrete was modelled as unconfined concrete, whereas the core concrete in the boundary elements were modelled as confined concrete using the Modified Park-Kent model [11]. Compression and tension softening of concrete were considered as recommended by Vecchio [12, 13]. Fig. 11 illustrates the finite element mesh for the wall. The wall was analyzed under the loading history applied during the test. This generated inelastic static analysis results under reversed cyclic loading. The initial analysis was based on the assumption that the grids were able to yield and develop ductile behavior as established by the stress-strain relationship of the wire, without the weld failure (yiled strength of $F_y = 550\ MPa$, ultimate strength of $F_u = 640\ MPa$ and ultimate rupturing strain of 4.35%). Experimental and analytical hysteretic relationships are compared in Fig. 12. The experimental results showed maximum recorded forces of +592 kN and -578 kN in the push and pull directions, respectively. The specimen yielded at 0.52% drift and failed during the 2.84% drift cycle after completing the 2.39% drift cycle successfully. In comparison, the VecTor2 analysis showed +618 kN and -592 kN maximum forces in the push and pull directions, respectively. The VecTor2 model yielded at 0.46% drift and failed during the 3.25% drift cycles after successfully completing the 2.82% drift cycles. The wall behaved in a ductile manner and failed by buckling of the longitudinal bars in compression. The analysis output showed a maximum concrete stress value of 90 MPa and 133 MPa for the unconfined skin concrete element and the confined core concrete element, respectively. The analysis also showed a maximum horizontal tensile stress of 460 MPa in the smeared reinforcement at the wall/foundation interface of the boundary element. The longitudinal bars at the extreme ends of the wall showed a maximum tensile stress of 649.2 MPa and a maximum compressive stress of 634.2 MPa. The analysis indicated slightly more ductile response than the test results, essentially because the weld failure was not allowed in the analytical model.

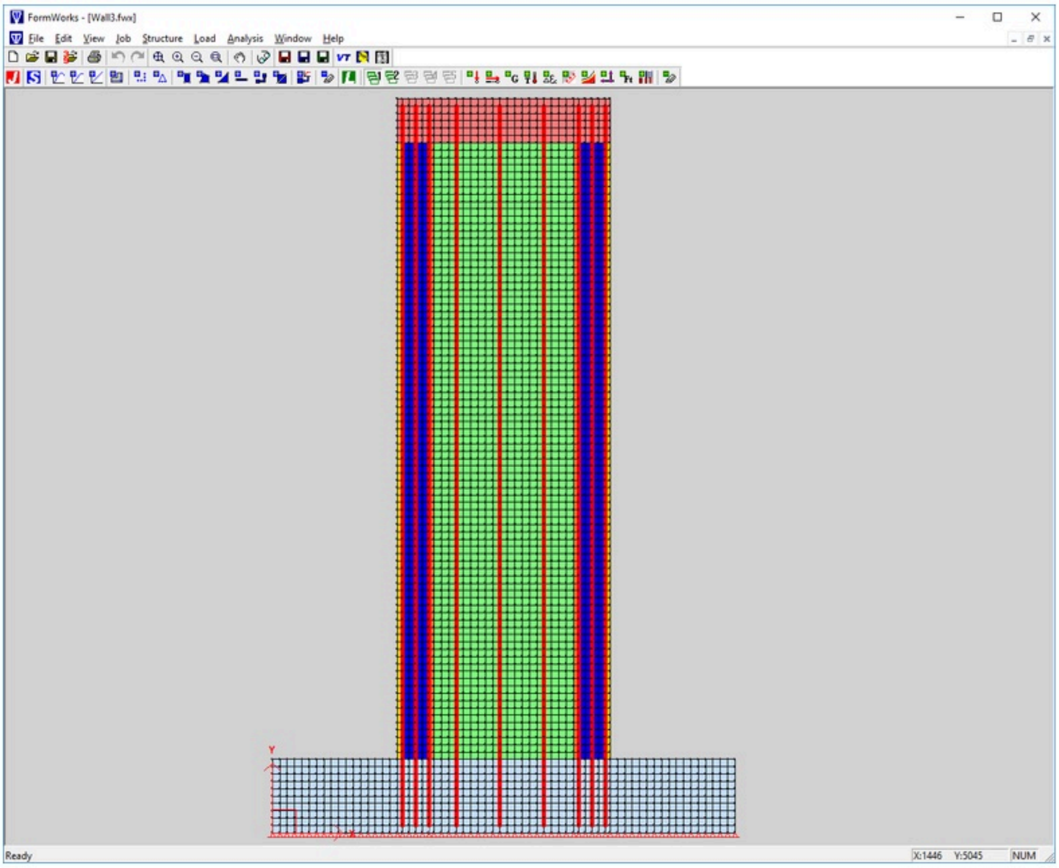

Fig. 11 – VecTor2 finite element model

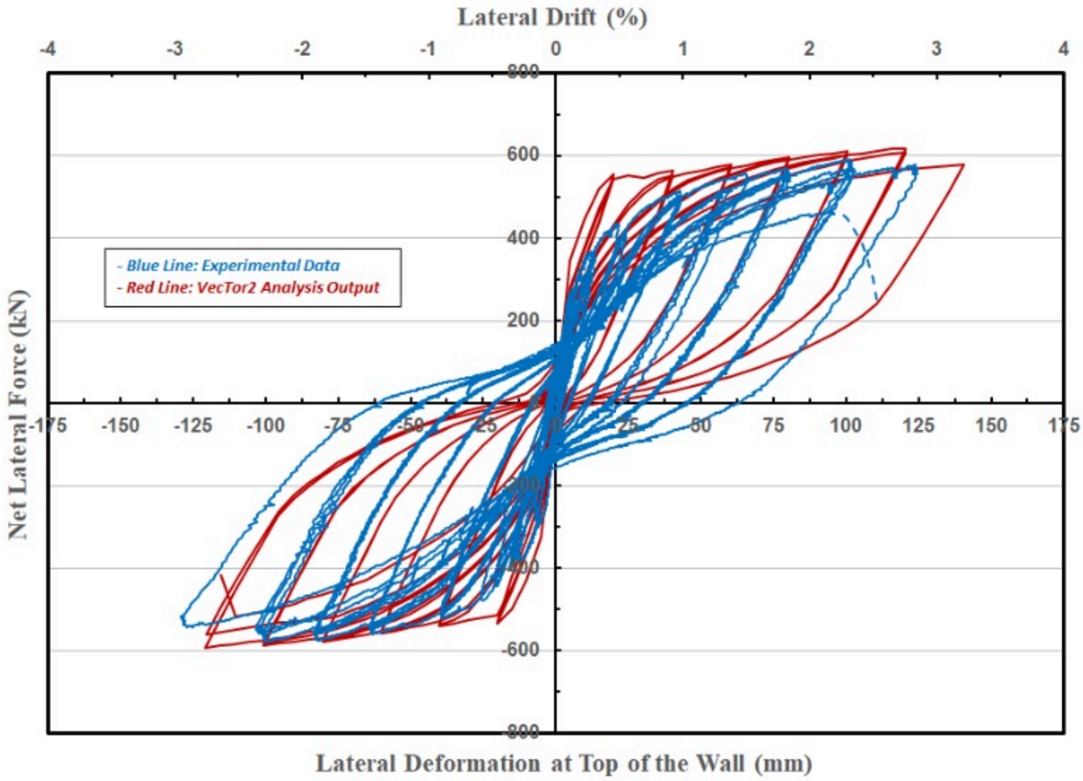

Fig. 12 – Hysteretic responses as obtained from experimental data and analysis results based on WRG yield strength of F_y =550 MPa, F_u = 640 MPa (no weld failure)

The stress-strain characteristics of WWR grids were varied in the subsequent three analyses to assess the effect of grid strength on wall response. The grids were modelled by capping their ultimate capacity at assumed weld failure strength. This was done by reducing the yield strength of transverse steel in the analytical model (to reflect premature weld failure), and setting the ultimate capacity of steel equal to the yield strength. ASTM 1064 considers 241 MPa as the minimum weld strength for WWR to be acceptable for use in structural applications in earthquake resistant structures. This strength limit was used as the lower bound strength in the parametric investigation. In addition, grid capacities of 400 MPa and 500 MPa were modelled to provide an understanding of the sensitivity of wall behavior to grid strength. The upper bound strength used in the parametric investigation corresponded to the strength of the wire, assuming no premature weld failure. The results of the latter analysis are presented in Fig. 12. The analyses results based on the reduced grid strength are presented in Figs. 13 through 15. The shear wall models with WWR strengths as governed by weld failures at 241 MPa, 400 MPa, and 500 MPa indicate gradually improving wall ductility rations with failuring during 1.39%, 1.89%, and 2.81% drift cycles, following successfully completion of three cycles at 0.92%, 1.42% and 2.32% drift ratios, respectively. These drift capacities can be compared with the ductility of the first model with WWR without the weld failure, which failed at 3.25% drift after succesfully compeleting three cycles at 2.84% drift ratio. In comparison, the test wall failed aat 2.84% drift after completing three succesful cycles at 2.39% drift ratio. These results indicate that the experimental response matched with the analytical model that had grid weld failure at 500MPa, prior to reaching the wire yield strength of 550 MPa. This observation is consistent with the strain gauge data recorded during the test, indicating that the transverse reinforcement design for the boundary element may be based on the capacity of the welds in the grids, as established by tests.

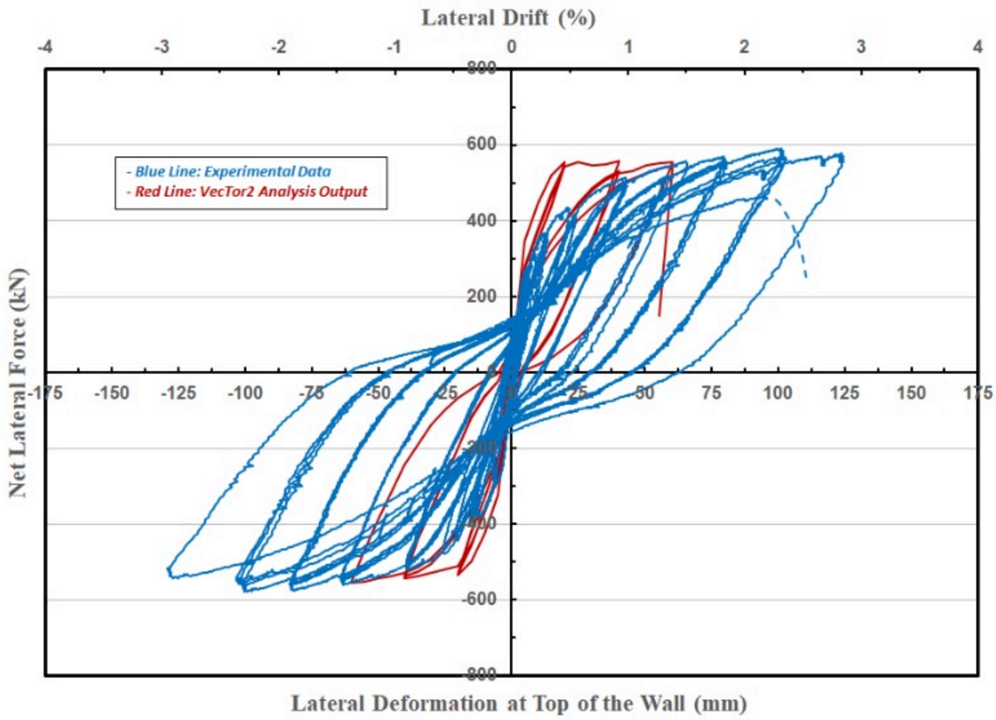

Fig. 13 – Hysteretic responses as obtained from experimental data and analysis results based on WRG yield strength of F_y =241 MPa, F_u = 242 MPa

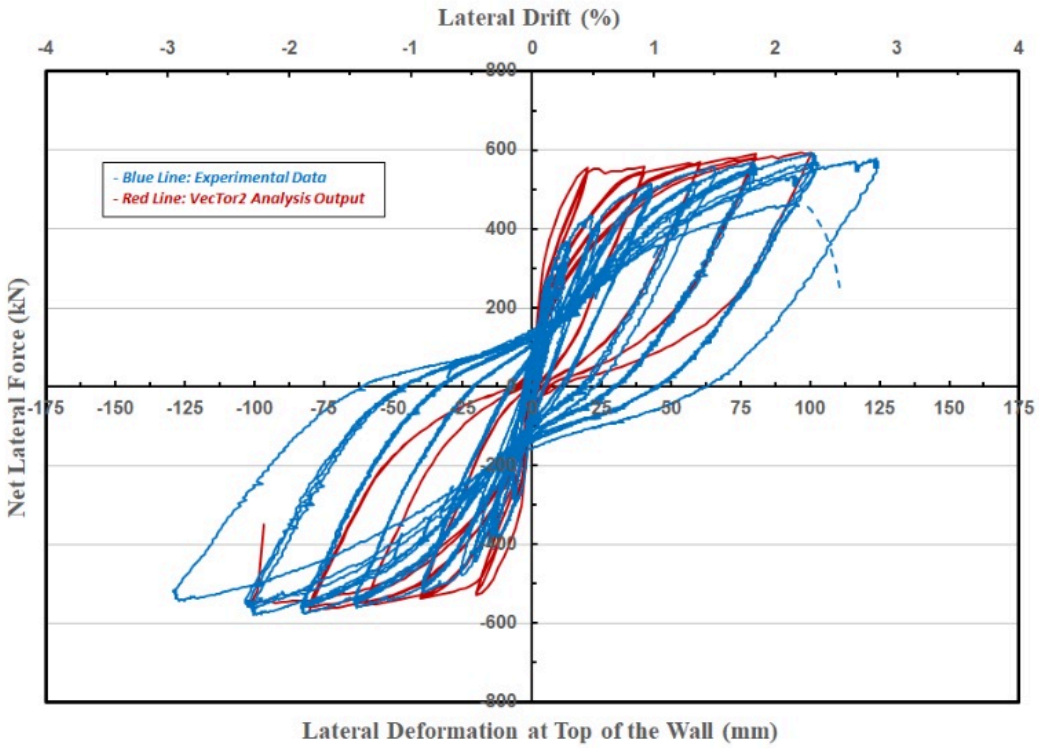

Fig. 14 – Hysteretic responses as obtained from experimental data and analysis results based on WRG yield strength of F_y =400 MPa, F_u = 401 MPa

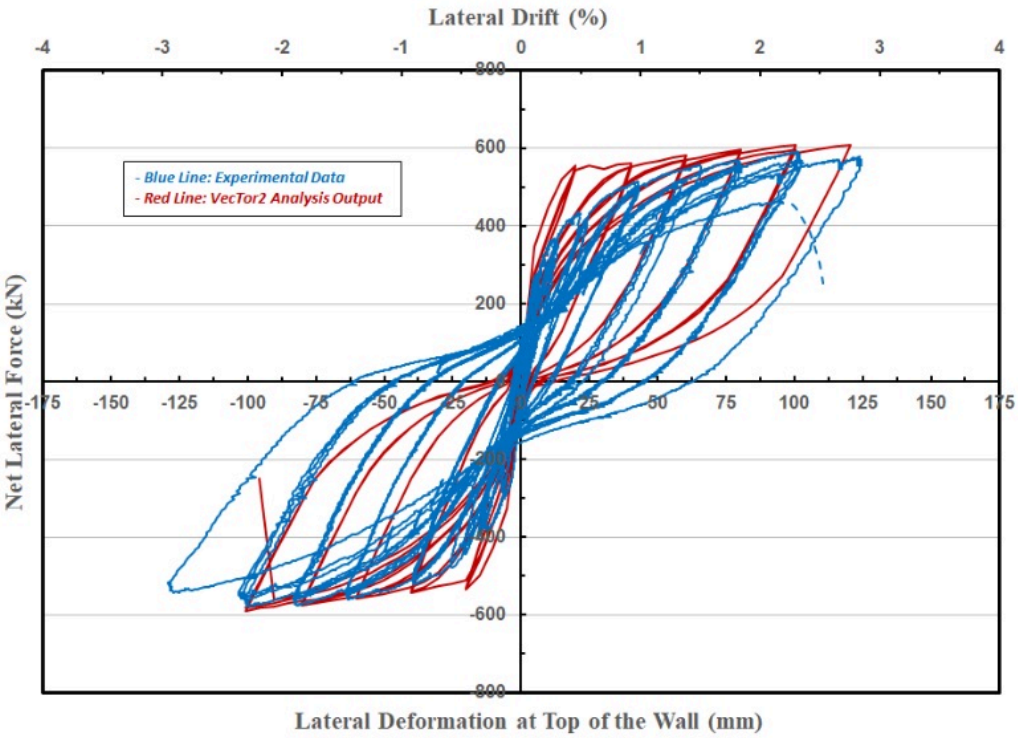

Fig. 15 – Hysteretic responses as obtained from experimental data and analysis results based on WRG yield strength of F_y =500 MPa, F_u = 501 MPa

The comparisons of wall responses shown in Figs 12 through 15 indicate that the experimentally observed behavior was best matched with the hysteretic response of the analytical model shown in Fig. 15 with WWR developing 90% of the wire yield strength. This is consistent with the material tests conducted. The average value of the ASTM 1064 weld tests and the burst tests was approximately 80% of the wire yield strength. Considering the actual grid behavior if the shear wall and the redundancies provided by multiple grids with multiple grid legs restraining the compression reinforcement in the wall boundary element, it is expected to have slightly better performance of grids during the shear wall test. Nevertheless, it is prudent to use lower design yield strength in computing the transverse grid reinforcement requirements for boundary elements unless the wire is shown to yield in burst tests prior to the grid weld failure in the material quality control tests. It is noteworthy that all the analytical predictions of non-linear wall behavior, including the lowest grid failure strength of 241 MPa, indicate that the wall strength capacity attained in the test was achieved. This is also expected as the transverse reinforcement in boundary elements begin engaging in the confinement mechanism of the core concrete and restraining the compression bars against buckling shortly after the development of concrete strains in the extreme compression fiber at peak concrete strength as the wall develops its flexural resistance.

4. Conclusions

The following conclusions can be drawn from the experimental and analytical research conducted:

- The shear wall tested in the experimental phase of research was designed as per ACI 318-14 [14] requirements to perform in the flexure mode of deformations. The wall performed in a ductile manner and developed three cycles at 2.3% drift, virtually without and strength decay, and failed during the second cycle at 2.8% drift in the splice region. The failure was initiated by concrete crushing and spalling in the cover near the splice region 500 mm to 600 mm above the wall foundation interface, followed by the buckling of longitudinal compression reinforcement upon the failure of grid welds. There was no distress observed in the grids until after 2.3% lateral drift cycles, but they could not sustain high transverse strain demands at 2.8% drift and failed through the failure of grid welds as illustrated in Fig. 10. Using 0.5% drift ratio as the global yield level for the wall, the displacement ductility ratio attained was 2.3/0.5 = 4.6 prior to reaching cycles with substantial strength decay.

- The analysis of the wall under the same reversed cyclic loading applied during the test, with different levels of grid failure stresses, indicated that the grid in the critical region of the wall developed weld failure when the grid wire stress level was at 500 MPa. This stress level corresponds to 90% of the wire yield strength. In spite of the premature weld failure, the wall developed a displacement ductility ratio of 4.6, which is consistent with the drift levels expected from a ductile shear wall.
- The burst tests of grids in concrete prisms provided the best estimate of the grid strength measured during the wall test. The results obtained were similar to the ASTM 1064 test results, although the mechanism of force resistance of the WWR grids were different than the stress condition generated during the ASTM 1064 weld tests. It is recommended to use the grid strength obtained from the burst tests for designing transverse reinforcement requirement in earthquake resistant shear walls. Only when the grid wires yield and rupture during the burst tests, the wire yield strength can be used in design.

5. References

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