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PERFORMANCE OF PARTIALLY DAMAGED SQUAT SHEAR WALLS UNDER REVERSED CYCLIC LOADING

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Abstract: Reinforced concrete (RC) shear walls can resist lateral loads effectively in a high rise structural system. Squat shear walls are encountered in nuclear power plant structures to resisting earthquake forces. This paper reports on some experimental investigations on behaviour of RC squat shear walls partially damaged under quasi-static loading (monotonic and reversed cyclic). Six large scale RC squat shear walls with boundary elements were tested to failure. Important variable of this study include height of wall maintaining all other parameters constant. Three sets of shear walls, containing two in each set, with aspect ratios 1.5, 1.25 and 1.0. In each set one wall was tested under monotonic lateral load, whereas the second walls with displacement reversals under lateral push and pull. These walls were tested under quasi-static loading up to failure. The tested shear walls with partial cracking were repaired and again subjected to above loading. The repair methodology involves removal of damaged concrete, straightening of steel bars (no rupture of reinforcement) and filling of cracks with cement grout. Testing of repaired shear walls was terminated after significant strength loss and deterioration in subsequent cycles. Load vs. displacement hysteresis loops, mode of failure, and crack pattern and overall response of the shear walls have been described. The seismic performance of all the shear walls has been analysed in terms of shear strength, stiffness degradation and ductility. Shear walls showed significant increase in ultimate lateral load at small aspect ratio. At small aspect ratios, shear walls exhibited brittle shear failure with horizontal cracking in the web.

1 INTRODUCTION

Shear walls are structural components that provide resistance to a building or a structure against lateral loading due to wind or earthquake. If the performance of the existing wall does not satisfy the requirements of design code its strength has to be enhanced by retrofitting (repair, strengthening or both). When the performance of the shear walls does not satisfy the requirements of strength, retrofitting techniques can be adopted to upgrade its performance level by any one of the methods practicable.

1.1 Classification of Shear Walls

The ratio of applied moment-to-applied shear force or aspect ratio (A/R) can be replaced with the ratio of wall height-to-horizontal wall length. The lateral loads can be resisted in shear walls either by cantilever action in slender walls or high rise walls (A/R > 2) or by truss action in squat/short walls or low rise walls (A/R < 2). High-rise shear walls predominantly fail in flexure, whereas low-rise walls fail in shear. Squat shear walls are used in nuclear plants where safety of structure is a concern.

1.2 Failure in Shear Walls

Yielding of tension or compression steel or crushing of concrete at ultimate stages causes flexure cracks near the bottom part of the wall. Inadequate horizontal or diagonal reinforcement causes diagonal tension cracks. Sufficient shear reinforcement with formation of compression strut can lead to diagonal compression failure. Provision of sufficient horizontal reinforcement with inadequate vertical reinforcement can cause sliding shear.

Local buckling of web can be prevented by adding boundary elements such as columns or flanges at ends, and minimum thickness of web. Proper confinement of wall can prevent in-plane splitting failure. Squat shear walls should be heavily reinforced to avoid shear failures. During an earthquake there may be a partial collapse (or damage) of the structure, which can be restored to the required level. Many strengthening schemes are being adapted to bring performance levels of deteriorated structures as per the current design codes of practice. Most of the retrofit techniques have resulted in various levels of upgradation. However, issues like effectiveness, resources, invasiveness, cost and practical implementation are major challenges to be overcome.

2 REVIEW OF LITERATURE

2.1 Behavior of RC Shear Walls

Barda et al. [1] investigated the behaviour of walls with flanges and boundary elements with transverse reinforcement. It has been observed that horizontal reinforcement was ineffective for shear resistance in walls with shear span-to-length ratios less than 0.5. Vertical reinforcement was found to be mostly effective for aspect ratio of 1.0.

Paulay et al. [2] reported that the aspect ratio differentiates them as low-rise walls and slender walls. Diagonal tension failure, diagonal compression failure (or web crushing and/ or splitting failure) and sliding shear failure at the base of wall are typical failure modes observed in low-rise walls under monotonic or cyclic lateral loading.

2.2 Transverse Reinforcement

Many researchers highlighted the importance of transverse reinforcement and also anchorage of wall reinforcement into boundary elements. **Oesterle et al. [3]** mentioned that dimensions of hoop and spacing serve four primary functions. Limiting strain capacity of concrete core has been improved by lateral confinement, and improved shear capacity and stiffness. Anchorage of wall horizontal reinforcement need to be extended across the boundary element and terminated with standard 90° bend for low shear walls and 135° hook for high shear walls.

Salonikios et al. [4] tested eleven reinforced concrete walls designed according to Eurocode 8. The parameters included aspect ratio, use of bidiagonal bars with and without gap at intersection at the base, web reinforcement ratio, longitudinal reinforcement ratio, with and without axial loading, and presence of construction joints. The walls failed predominantly in flexure, with intersection of flexural cracks originating from opposite edges. Concrete crushing and buckling of reinforcement at confined edges was observed. Sliding was controlled in walls with bidiagonal bars.

2.3 Repair of RC Shear Walls

Vecchio et al. [5] tested repaired RC shear walls with conventional technique of removal and replacement of concrete. The repair procedure included recasting of web with new concrete up to top 180mm from soffit of top slab and completed using a high-strength, non-shrink epoxy grout. Punching shear failure was observed in web-flange joint, whereas shear slip failure at the base near repair interface in the reconstructed walls.

Selective techniques for repairing damaged walls were brought out by **Elnashai et al. [6].** Stiffness, ductility and shear strength can be increased individually without altering other two parameters depending on arrangement of steel strips. Plates were bonded to the wall with epoxy mortars. Addition of external unbounded reinforcement bars increased strength without affecting stiffness of walls. It has been recommended that strengthening of RC structural members with externally bonding steel strips for improving the seismic resistance of walls.

The technique of external bonding with steel strips was extended to RC walls subjected to earthquake loading by **Altin et al. [7].** Walls are provided with alignment of steel strips such as diagonal, lateral and a combination of lateral and vertical. Application of epoxy adhesive to surface of wall and insertion of anchor rods prevented debonding of steel strips and delayed premature buckling of steel strips.

Bass et al. [8] tested push off specimens consisting of column referred as “base block” and the infill wall constructed above the base block referred as “new wall”. Specimens were cast in vertical, horizontal overhead positions. Different interface characteristics were adopted, which include; untreated, heavily sand blasted, chipped to achieve surface roughness, provided with shear key and coated with epoxy agent before pouring new concrete. It has been concluded that increase in embedment depth, amount of reinforcement, and use of deep surface preparation techniques at the interface results in high shear strength at higher slip levels.

Marini et al. [9] assessed the effectiveness of seismic strengthening of RC shear walls with thin high performance jackets. High performance jacket includes a steel mesh with tensile strength of 1200MPa covered with a thin layer of concrete of compressive strength greater than 140MPa. The steel mesh consisted of bent weaved wires of 2mm diameter and 20mm spacing. The tested walls experienced a uniform crack pattern with limited opening extended from the wall base to the critical zone of failure. The failure of strengthening walls occurred due to crushing of concrete jacket at the base, exhibiting high ultimate strength, structure deformation capacity and ductility.

Lombard et al. [10] studied the effectiveness of externally bonded carbon fibre tow sheets for seismic strengthening and repair of reinforced concrete walls. The damaged wall was repaired by one vertical layer of FRP. The undamaged walls are also strengthened with FRP. The failure in control wall was due to crushing of concrete at the toe of the wall. The repaired wall regained 90% of the initial stiffness and experienced ductile failure in flexure mode. The strengthened walls had increased secant stiffness and load carrying capacity.

Antoniades et al. [11] retrofitted squat RC shear walls with FRP jackets in combination with FRP strips in order to increase the strength of heavily damaged walls during earthquake. The original walls were designed according to Eurocode 8 provisions. One wall was conventionally repaired and remaining damaged walls were strengthened with FRP. Conventionally repaired wall could achieve original strength but showed less initial stiffness and energy dissipation capacity when compared to original wall. The hysteresis behaviour and initial stiffness of FRP strengthened wall was more than the conventionally repaired wall but less than that of the original wall. The propagation of existing shear cracks was prevented by FRP jackets.

The effect of detailing deficiencies in RC walls like lap splicing of the longitudinal reinforcement in the plastic hinge region, poor confinement of boundary elements and anchorage of transverse reinforcement, and insufficient shear strength to develop hinging was studied by **Paterson et al. [12]. S**pecimen provided with lap splice at the base failed due to failure of lap splice and in the region of flexural hinging exhibited more ductile capacity. Retrofitting of walls included wrapping of carbon fibre on collar head and also additional confinement reinforcement over the lap splice region along with headed bars and CFRP wrapping. The combination of headed reinforcement with CFRP reduced the shear distress and was effective in confining the boundary elements.

Ghobarah et al. [13] rehabilitated RC walls with fibre composites to enhance the wall capacity to earthquake loads and also ductility. Three walls were tested; control wall, upgraded with bi-directional fibre-reinforced sheets and bidirectional sheets anchored with additional steel bolts. The boundary elements were also provided with FRP anchors in the upgraded walls. The control wall which is deficient in shear and ductility developed tension cracks at the bottom and mid height of the wall. The rehabilitated walls recorded more strain in concrete, less strain in horizontal bars, higher displacement ductility than the control wall. The alignment of fibres at 45 degrees prevented shear mode of failure. FRP anchors failed in shear but steel anchors failed due to yielding. The confinement of the boundary elements with CFRP sheets delayed concrete crushing and also enabled in developing full tension in longitudinal bars.

Li et al. [14] designed eight walls with less confinement reinforcement as recommended by New Zealand Concrete Design Code and American Concrete Institute Code. The original walls had boundary elements and exhibited heavy damage at the base with shear and flexural cracks along the height. The repair and retrofit methodology included removal of damaged concrete and replacing with polymer modified repair mortar, injection of epoxy into cracks at the base of the wall and retrofitting with GFRP/ CFRP material. The repaired walls showed less stiffness degradation, more ductility, and recovered almost original strength before post-peak than the original specimens. It has been observed that strength retention of CFRP wrapped specimens is more than that of GFRP specimens.

3 EXPERIMENTAL PROGRAMME

The repair and testing of six RC shear walls that have sustained damage during lateral loading is herein addressed in this work. The repaired walls are designated as RSW-MO-1.5, RSW-CY-1.5, RSW-MO-1.25, RSW-CY-1.25, RSW-MO-1.0 and RSW-CY-1.0. The first three alphabets ‘RSW’ indicate repaired shear wall, accompanied by two letters ‘MO’ or ‘CY’ subjected to monotonic or cyclic loading and then numerical value representing aspect ratio (i.e. 1.5, 1.25 or 1.0). The control walls are designated as SW-MO-1.5, SW-CY-1.5, SW-MO-1.25, SW-CY-1.25, SW-MO-1.0 and SW-CY-1.0. The first two alphabets ‘SW’ indicate shear wall. The geometric and reinforcement details of control (undamaged) walls are discussed in section 3.1. Further, the repair methodology adopted for partially damaged control walls is explained in section 3.2.

3.1 Description of Shear Walls

Control walls were designed with good detailing as per IS:13920 for achieving sufficient ductility. The specimens are provided with stiff beam at the top and heavily reinforced foundation block at the base. The web wall was cast monolithically with top beam and foundation block. The web wall has rectangular section of width of 600mm and thickness (or depth) of 150mm which are maintained constant for all the shear walls. The edges of the web wall were confined with stiff boundary elements having a length of 500mm and thickness of 200mm. Hence the web wall has a total width of 1000mm including the thickness of the boundary wall on both edges. The height of the web is kept varying as 1500mm, 1250mm and 1000mm and respectively corresponding to the aspect ratios of 1.5, 1.25 and 1.0. The dimensions of web wall are shown in Figure 1.

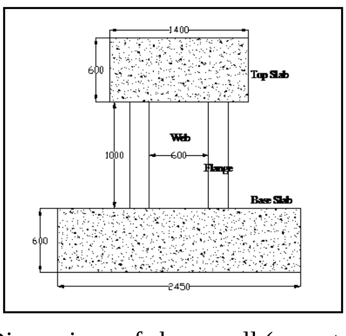

Figure 1: Dimensions of shear wall (aspect ratio 1.0). All dimensions are in mm

3.2 Repair and Strengthening of Specimens

The walls (control) initially tested to failure, experienced heavy damage in the critical region of web wall and bottom edge of the boundary wall. The failure was manifested by crushing of concrete in the web wall, a horizontal crack in the lower portion of web wall and buckling of longitudinal bars in the bottom edge of the boundary wall. It is necessary to repair the heavily damaged areas in the web and to restore the original capacity, before applying any strengthening technique.

The repair methodology involved removal of damaged concrete from the base of the web wall including lower portion of boundary elements and then replacement of damaged concrete as shown in Figure 2. In damaged areas, concrete was removed throughout the thickness of web as shown in Figure 3. The visual inspection of exposed reinforcement bars after removal of damaged concrete concluded that fracture of steel did not occur but buckling of bars was observed. Welding of steel bars was not required, whereas buckled bars were straightened. The damaged region was filled with same grade of concrete. Superplasticizer was added to make the concrete flowable for easy compaction. The maximum size of coarse aggregate was restricted to 12.5mm to enable easy packing.

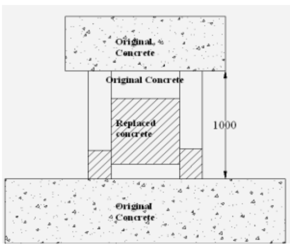

Figure 2: Schematic sketch of repair methodology.

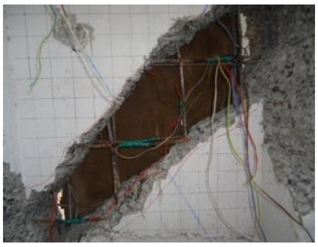

Figure 3: Removal of heavily damaged region of web.

Figure 4: Replacement with new concrete.

The surface cracks were closed by grouting and surface coating using NITOBOND SBR cement grout. The grouting procedure started with formation of "V" groove on crack surface which are more than 1mm in width and depth was about 3 to 5mm. The width of the groove was about 10mm. The surface was made wet and the groove was filled with RENDROC CS paste. Holes were drilled in the crack and PVC pipes were inserted and then fixed using Rendroc Plug. The holes of width larger than 5mm were closed using Nitobond SBR modified. Hairline cracks were filled by coating with cement slurry mixed with Nitobond SBR which also rendered a finished surface. Finally, pressure grouting using cement slurry admixed with Nitobond S B R was done through the inserted PVC pipes to close the voids in the cracked concrete. After curing projected PVC pipe was trimmed off, and then filled with Nitobond SBR grout.

Figure 5: Shear wall after grouting and surface finishing.

3.3 Testing of Shear Walls

As-built wall specimen of each aspect ratio was tested under monotonic loading while the companion wall was tested under reversed cyclic loading. A total of three specimens were tested under monotonic loading and remaining three to reversed cyclic loading. Similar test set-up and loading protocol were maintained for both control and repaired specimens. The walls were tested by positioning in the vertical direction, fixed to a strong reinforced foundation block which was anchored to a strong floor using six 6-20mm diameter high tension bolts. A 1000kN MTS hydraulic actuator attached to the a strong wall was used to transfer horizontal load. The horizontal loading was applied at the top beam of the walls, wherein two metal plates were attached and connected through four high strength bolts passing through the top beam and actuator. The bolts connecting the actuator and top beam and also foundation block and strong floor were pre-tensioned. A constant axial load of 6T was applied using two hydraulic jacks of 50T capacity mounted on the top beam and in turn connected to the loading frame. The schematic sketch and test set-up is shown in Figures 6 and 7 respectively.

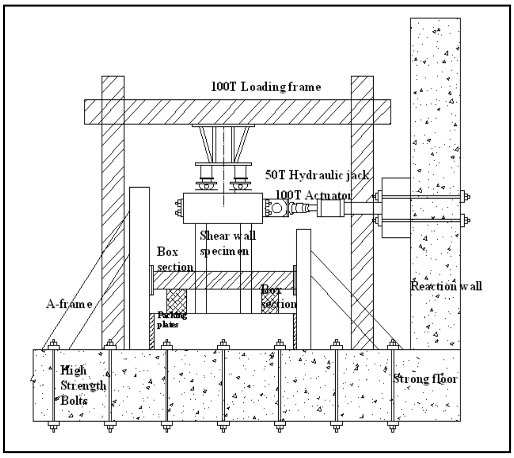

Figure 6: Schematic sketch of test set-up.

3.4 LOADING HISTORY

Displacement control was used throughout the test. The typical loading history included two loading cycles at each displacement level until failure. The application of displacement amplitudes has been followed as per FEMA 461 guidelines. Two cycles were applied at each displacement level. Each reversed cycle of loading consisted of push and pull. Push is “negative” and away from the strong wall and pull is “positive” and towards the strong wall. The specimen experienced same peak displacement value in both push and pull directions. The loading sequence is given in Figure 8.

Figure 7: Test set-up of shear wall.

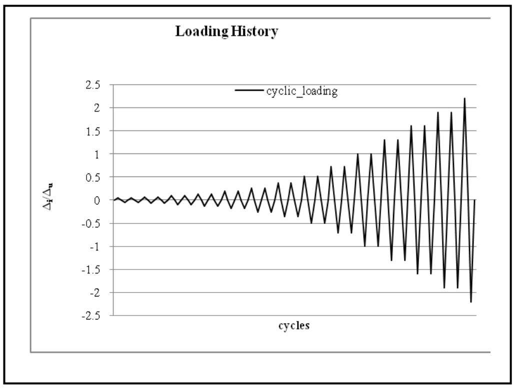

Figure 8: Loading Sequence.

4 INSTRUMENTATION

The instrumentation included a load cell at the end of the hydraulic actuator, and a series of linear variable displacement transducers

(LVDTs). The concrete surface strains were measured using LVDT's (Linear Variable Differential Transducers). LVDTs were mounted to measure the diagonal deformations on both faces of the web wall, uplift and sliding of foundation block, uplift of top beam, web expansion and top lateral displacement.

Internal strain gauges were mounted on the steel bars both on longitudinal and transverse reinforcement. Electrical resistance strain gauges of 120ohm were used. The portion of ribbed surface on the reinforcement rods where the strain gauge had to be prepared to achieve a smooth surface. Strain gauge was bonded on the ground surface using adhesive and then strain gauge wire was soldered to the PVC sheet. An external wire was soldered to the sheet using soldering paste. Then strain gauge was then wrapped with an epoxy material to prevent damage during casting. All the strain gauge wires were taken out carefully from reinforcement cage and bonded together during casting. These wires were connected to a data acquisition system during testing.

5 AXIAL LOAD

A constant axial load was applied on the top beam by a pair of hydraulic jacks each of 50T capacity. The applied axial load represented was equal to 2.0% of axial load capacity. The axial load was calculated based on a normalized average axial stress of $0.1f_{ck}A_g$ in the wall, which is normalized as a function of concrete compressive strength. A_g is the cross sectional area of the wall.

6 DISCUSSION OF RESULTS

The push-over curves under monotonic loading and hysteresis loops under cyclic loading are drawn. The crack pattern was also observed during testing. The parameters to be evaluated from experimental investigations include; shear strength, energy dissipation, stiffness degradation and displacement ductility. Moderate sliding at the fixed base was observed, but did not significantly affect the failure mode.

6.1 Crack Pattern

The general observation of crack pattern in all the specimens is described in this paragraph. Crack patterns and failure modes for all walls subjected to monotonic loading and reversed cyclic loading were generally similar. Hairline flexural cracks and shear cracks were developed along the height of the wall in the first stage. These hairline cracks were first observed on the boundary element along the height during first cycles of loading. Flexural failure under the flanges and sliding failure along the wall base were also characterized. The main flexural crack propagated at the base followed by strength degradation and subsequent failure. Crack propagation increased along the height and penetrated through the thickness of the web wall with increasing amplitudes of displacement. A crack was observed at the interface of the repaired zone in the web wall. A horizontal flexural crack was formed at the base of the boundary wall and foundation block at the interface of repair concrete which gives clear evidence that most inelastic behavior occurred at this location. The diagonal cracks on the web wall did not open initially due to the presence of boundary elements. With increasing displacements, these diagonal cracks opened up and increased size leading to crushing of concrete especially during cyclic loading.

Figure 9: Spalling of concrete in web region and compression flange of RSW-MO-1.5.

Buckling of the longitudinal steel reinforcement was observed at the bottom edge of the boundary elements during monotonic loading. The compression flange exhibited significant crushing of concrete in the bottom part of the boundary element. The failure of repaired walls subjected to monotonic loading is shown in Figures 9-11.

Figure 10: Spalling of concrete on compression flange and bottom portion of web wall in RSW-MO-1.25.

Figure 11: Final cracking pattern in shear wall RSW-MO-1.0.

Figure 12: Final failure of shear wall RSW-CY-1.5.

The inelastic flexural deformation took place at the horizontal flexural crack formed at the boundary elements base and is indicated by the buckling of longitudinal steel. The post peak resistance of the walls was drastically reduced (a sudden fall in strength) at the failure stage. Failure was dominated by shear mechanism. Failure of walls subjected to cyclic loading is shown in Figures 12-14.

Figure 13: Complete cracking and concrete crushing of web wall and boundary element in RSW-CY-1.25

Figure 14: Final cracking pattern in shear wall RSW-CY-1.0

The crack widths are generally high on the web wall and at higher displacements led to crushing of concrete when reversed cyclic loading was applied. Buckling of yielded steel bars was accompanied by spalling of crushed concrete in the web region.

6.2 Lateral load vs. Lateral Displacement

Figures 15 to 17 show the variation of lateral load vs. lateral displacement of shear walls RSW-MO-1.5 and SW-MO-1.5, RSW-MO-1.25 and SW-MO-1.25, RSW-MO-1.0 and SW-MO-1.0 subjected to monotonic pushing in one direction. The maximum loads recorded for repaired shear walls RSW-MO-1.5, RSW-MO-1.25 and RSW-MO-1.0 are 462kN, 515kN and 676kN at displacements of 23mm, 29mm and 35mm respectively. The maximum loads recorded for control shear walls SW-MO-1.5, SW-MO-1.25 and SW-MO-1.0 are 481kN, 533.8kN and 694.2kN at displacements of 11.7mm, 15.42mm and 12.45mm respectively.

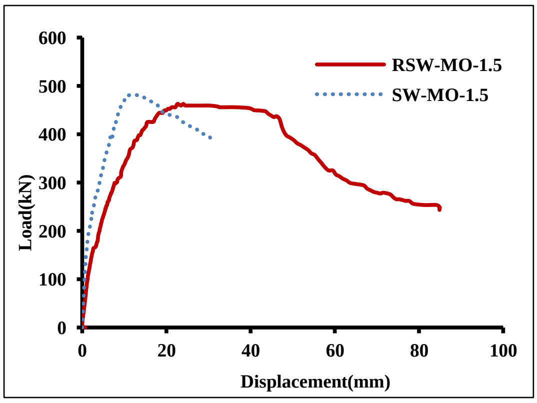

Figure 15: Load vs. Displacement curve for repaired and control walls under monotonic loading (AR 1.5).

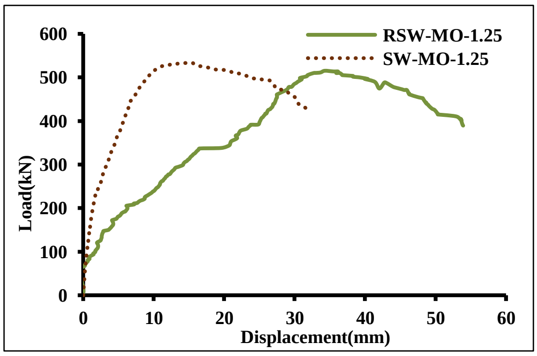

Figure 16: Load vs. Displacement curve for repaired and control walls under monotonic loading (AR 1.25).

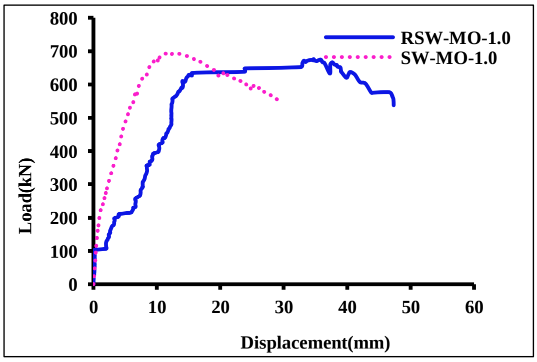

Figure 17: Load vs. Displacement curve for repaired and control walls under monotonic loading (AR 1.0).

The strength and displacement at which ultimate stage is reached decreased as the aspect ratio decreased from 1.5 to 1.0 for both repaired and control shear walls under monotonic loading.

Figures 18 to 20 show the lateral load vs. Lateral displacement as hysteresis loops for the repaired as well as control walls RSW-CY-1.5 and SW-CY-1.5, RSW-CY-1.25 and SW-CY-1.25, RSW-CY-1.0 and SW-CY-1.0 subjected to cyclic pushing and pulling up to failure. The maximum loads recorded for repaired shear walls RSW-CY-1.5, RSW-CY-1.25, RSW-CY-1.0 are 471kN, 551kN, 674kN at displacements of 80mm, 50mm, 60mm respectively in the pushing direction and 477kN, 378kN, 510kN at displacements of 80mm, 40mm and 60mm respectively in the pulling direction. The maximum loads recorded for control shear walls SW-CY-1.5, SW-CY-1.25, SW-CY-1.0 are 461.8kN, 530.2kN, 761.46kN at displacements of 18.77mm, 11.53mm, 12.76mm respectively in the pushing direction and 490.5kN, 396.4kN, 630kN at displacements of 21.58mm, 14.17mm and 7.25mm respectively in the pulling direction. Hence peak load is recorded for the shear wall with aspect ratio 1.0 cyclic loading.

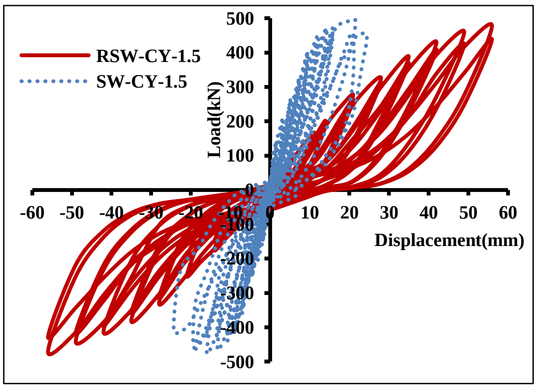

Figure 18: Load versus Displacement Hysteresis Loop for repaired and control walls subjected to cyclic loading with aspect ratio 1.5

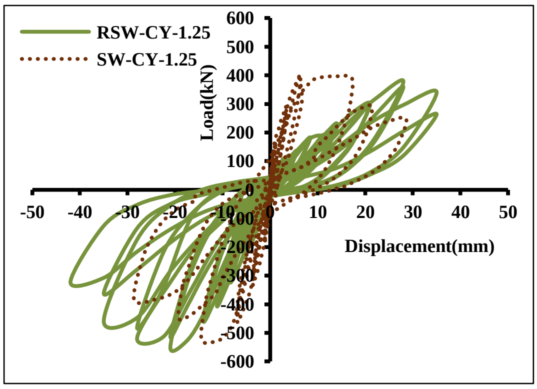

Figure 19: Load versus Displacement Hysteresis Loop for repaired and control walls subjected to cyclic loading with aspect ratio 1.25

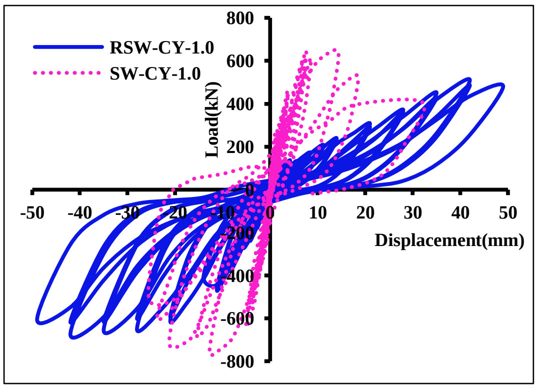

Figure 20: Load versus Displacement Hysteresis Loop for repaired and control walls subjected to cyclic loading with aspect ratio 1.0

6.3 Envelope Curve

Envelope curve is the locus of extremities of the load-displacement hysteresis loops, which contains the peak loads from the first cycle of each phase of the cyclic loading as per ***ASTM E2126-11***. For walls subjected to monotonic loading the observed push-over curve is the envelope curve. The envelope curve for repaired and control walls subjected to monotonic loading is plotted in Figure 21. Repaired shear walls RSW-MO-1.5, RSW-MO-1.25, RSW-MO-1.0 retained 96%, 96% and 97% peak load than control shear wall SW-MO-1.5, SW-MO-1.25 and SW-MO-1.0 respectively.

Repaired shear walls RSW-CY-1.5, RSW-CY-1.25 and RSW-CY-1.0 retained 98%, 104% and 89% in the push direction and 97%, 95% and 81% in the pull direction w.r.t control shear walls SW-CY-1.5, SW-CY-1.25 and SW-CY-1.0.

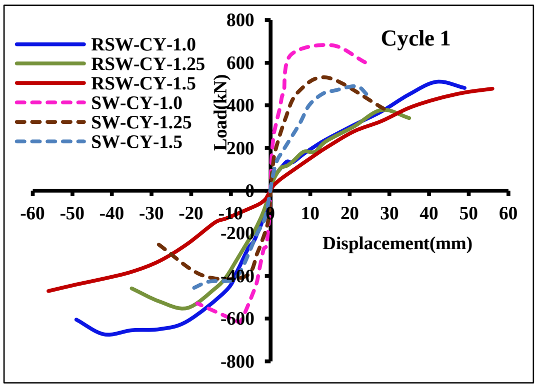

Figure 21: Envelope curve for repaired and control walls subjected to monotonic loading

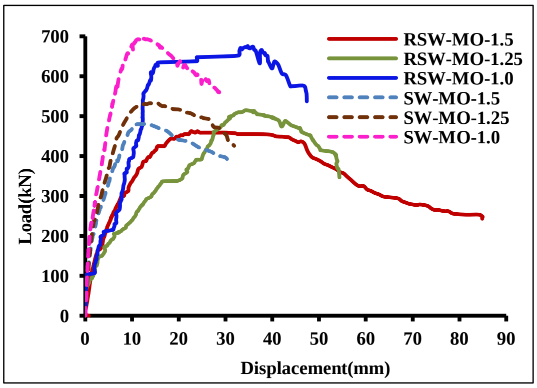

Figure 22: Envelope curve for repaired and control walls subjected to cyclic loading

The hysteresis loops for repaired walls are better than the original walls and hence shows that the repaired walls significantly resisted diagonal shear cracks. Peak load occurred after several cycles in the inelastic range and prior to failure. After peak load, there was strength deterioration and stiffness degradation as seen in the hysteresis loops.

6.4 Stiffness Degradation

The stiffness at every load cycle is calculated as the slope of the line joining the points of peak loads under push and pull in a displacement cycle. Stiffness is normalized by dividing stiffness at any load cycle by the initial stiffness for better comparison. The normalized stiffness degradation curve in cycle 1 of each displacement is shown in Figure 23. The control specimens were tested upto failure showing extensive crushing of concrete and buckling of reinforcement. The initial stiffness of the repaired walls is clearly less than that of the original one. The conventional repair method of removal and replacement of concrete with cement grouting under pressure of the cracks for web wall and boundary elements could not fully restore the original stiffness of walls.

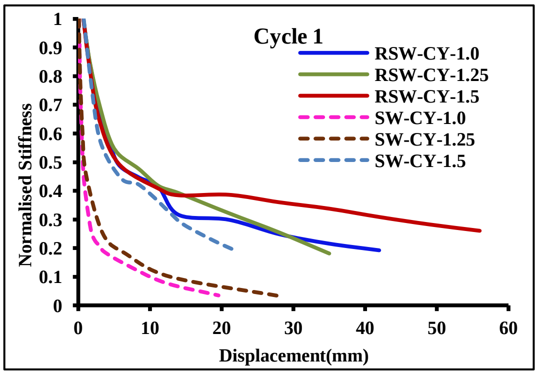

Figure 23: Normalised stiffness degradation curve for repaired and control walls

6.5 Cumulative Energy Dissipated

Energy dissipated in the joint can be calculated as the area under the load vs. displacement curve. The area for each cycle is calculated and then summed up for the cumulative energy dissipation for the entire load and unload cycles. For comparison, the cumulative energy dissipation is normalized by dividing it by the product of the volume of the joint and the compressive strength of concrete.

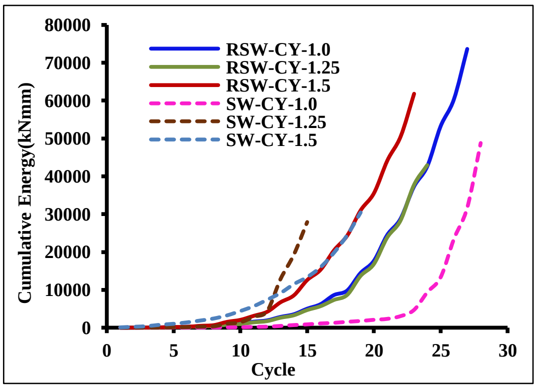

Figure 24: Cumulative Energy dissipation curve for repaired and control walls

The repaired shear walls recorded higher cumulative energy than the control shear walls. RSW-CY-1.5, RSW-CY-1.25 and RSW-CY-1.0 recorded 51%, 54% and 51% higher cumulative energy than the control walls SW-CY-1.5, SW-CY-1.25 and SW-CY-1.0. As a general observation, with the decrease in the aspect ratio, the cumulative energy dissipated increased for both control and repaired walls.

6.6 Displacement Ductility Factor

The displacement ductility factor is calculated using the observed experimental load vs. displacement curves. The displacement ductility values for repaired and control walls are plotted in Figure 25 for walls subjected to monotonic loading and Figure 26 for walls subjected to reversed cyclic loading.

The repaired walls recorded a lesser ductility value than the control walls and also followed similar trend as the control walls.

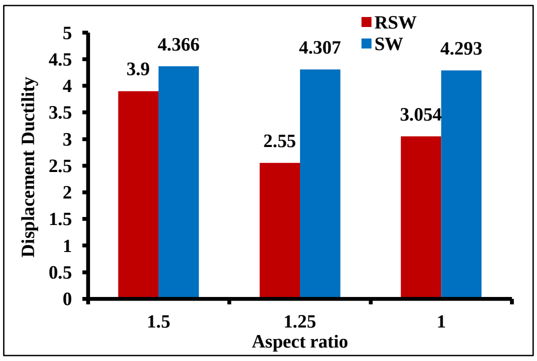

Figure 25: Displacement Ductility values for repaired and control walls under monotonic loading

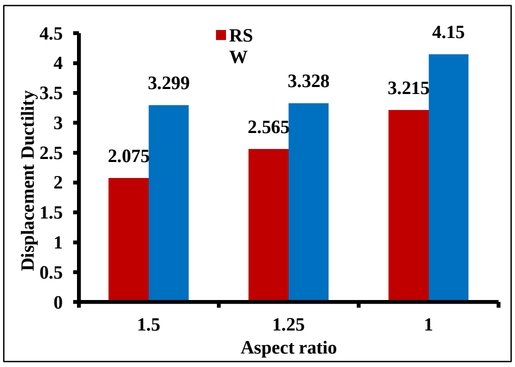

Figure 26: Displacement Ductility values for repaired and control walls under reversed cyclic loading

7 CONCLUSION

The control and repaired walls showed crushing of concrete and predominantly buckling of longitudinal steel. Failure is due to concrete crushing in the web and buckling of longitudinal steel at the confined edges of boundary elements during reversed cyclic loading. Horizontal shear at the interface between old and new concrete resulted in severe cracking between the boundary elements and foundation block, web wall and foundation block especially during cyclic loading. Original strength of the repaired walls was almost restored. Initial stiffness of repaired walls is very less. Geometrically similar walls with low aspect ratio exhibited an increase in shear strength, decreases the ductility and increases the energy dissipation. Also, the repaired walls exhibited similar mode of failure, improved energy dissipation, more stiffness degradation and less displacement ductility.

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