Effect of reinforcing steel bond on the seismic performance of lightly reinforced concrete walls

2014 NZSEE Conference

V.J. Patel, B.C. Van & R.S. Henry

The University of Auckland, Auckland, New Zealand.

ABSTRACT: During the Canterbury earthquake series, several reinforced concrete (RC) walls formed a limited number of cracks at the wall base as opposed to the expected distributed cracking in ductile plastic hinge regions. The ductility of a lightly reinforced concrete wall is dependent on the distribution of cracks, as well as the reinforcement bond and yield penetration at each crack. A series of experimental tests were conducted to investigate how the bond characteristics of reinforcing steel would influence the yield penetration and crack distribution in lightly reinforced concrete members. To vary the bond characteristics, reinforcement with three different deformation patterns were investigated, including a standard deformation pattern and two modified bars with either half the rib height or double the rib spacing of a standard bar. Pull out tests were conducted to quantify the bond strength of the reinforcement with different deformation patterns, followed by direct tension tests of prisms that represented the end region of an RC wall with minimum vertical reinforcement. The pull out tests indicated that halving the rib height and doubling the rib spacing had similar effects of reducing the reinforcement bond strength. The direct tension tests showed similar crack patterns for the two modified bar types, but increased secondary cracking for the standard bar due to the higher bond strength. However, only the half rib height bar displayed a higher ductility than the standard bar, with significantly greater yield penetration at each crack. Using half rib height bars as vertical reinforcement would potentially improve the ductility of lightly reinforced concrete walls.

1 INTRODUCTION

A ductile reinforced concrete (RC) wall is expected to form distributed cracks within the plastic hinge region when subjected to earthquakes. The distributed cracks encourage the reinforcing steel to yield over a significant length, which allows the wall to achieve a greater plastic rotation prior to the reinforcement reaching its elongation capacity. During the 2010/2011 Canterbury earthquake series, several lightly reinforced concrete walls (LRCWs) did not behave in a ductile manner, instead forming only a single or limited number of cracks at the base of the wall. For example, a single crack was observed at the base of one of the RC walls in the Gallery Apartments building and the concentrated inelastic demand resulted in premature fracture of the vertical reinforcement (Smith and England, 2012). LRCWs are unlikely to form distributed cracks when the vertical reinforcement content is such that the cracking moment exceeds the nominal moment of the cracked section (Davey and Blaikie, 2005; Henry 2013).

For a wall with a single flexural crack in the plastic hinge region, the ultimate plastic rotation is proportional to the ultimate crack width and the elongation of the steel at the flexural tension edge (Davey and Blaikie, 2005). The elongation of the reinforcing steel is governed by the steel strain and the depth that inelastic strains can penetrate into the concrete, known as yield penetration (Davey and Blaikie, 2005). Numerical models developed by Haskett et al. (2009) show that bars with lower bond strength will allow inelastic strains to penetrate further into the uncracked concrete section. Additionally, the Structural Engineering Society of New Zealand (2011) recommends debonding the vertical reinforcement at precast concrete panel joints, which could alternatively be achieved by using reinforcing steel with reduced bond strength.

The objective of this research was to investigate whether reducing the bond strength of the reinforcing would increase the yield penetration and potentially increase the ultimate crack width and ductility of LRCWs. The reduced bond strength was achieved by altering the deformation patterns on the reinforcing steel. Trial bars were manufactured with different deformation patterns, and a series of experimental tests were conducted to assess the bond characteristics and potential impact of the deformation patterns on the ductility of LRCWs.

2 EXPERIMENTAL SETUP

Reinforcing steel with three different deformation patterns were investigated using D12 bars, including standard rib deformations, deformations with double the standard rib spacing, and deformations with half of the standard rib height. Details of the rib height and spacing for each of the three deformation patterns are shown in Table 1. Reducing the projected rib area is expected to weaken the bond strength (Kimura and Jirsa, 1992). Although the half rib height and double rib spacing bars possessed the same projected rib area, both variations were included to investigate whether the rib geometries also affected bond strength and RC member ductility.

Table 1 Deformation pattern specification

Deformation pattern	Rib height (mm)	Rib spacing (mm)
Standard bar	1.2	8.5
Double rib spacing bar	1.2	17
Half rib height bar	0.6	8.5

To compare the effect of the altered reinforcement deformation patterns on the ductility of LRCWs, pull out tests and direct tension tests were conducted. The pull out test followed a standard test method that was used to compare the bond characteristics of the three deformation patterns. The direct tension tests were conducted to evaluate the crack development, crack spacing, crack width, and overall ductility of the RC prisms.

2.1 Pull out test

To compare the bond characteristics of the three different deformation patterns, lengths of reinforcing steel were embedded into concrete blocks and a tensile load was applied to one end of the bar. The bond characteristics were evaluated by plotting the pull out load against the free end slip. The pull out test setup, shown on Figure 1, was partially adopted from the pull out tests conducted by Kimura and Jirsa (1992). Three pull out tests were conducted for each bar type.

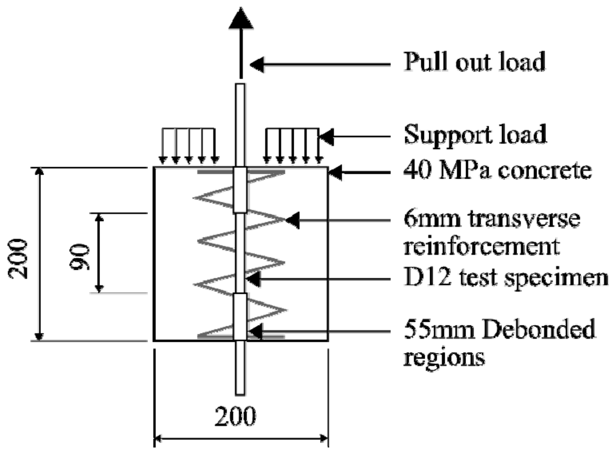
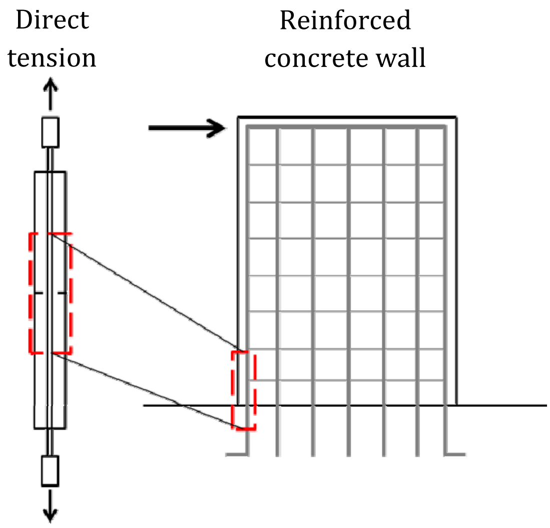


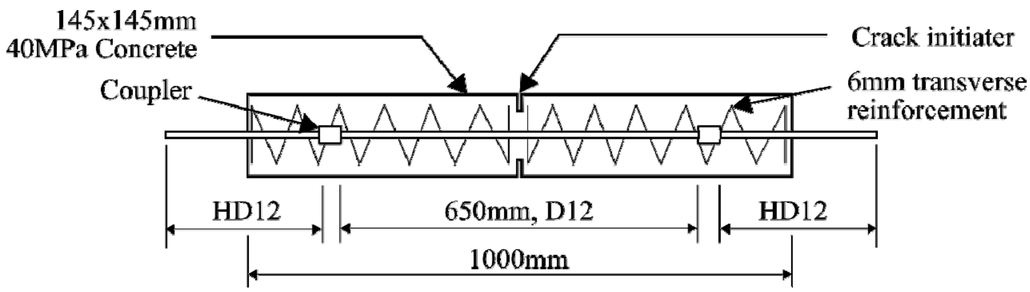
Figure 1. Pull out test setup

2.2 Direct tension test

The direct tension prisms were designed to represent the outer most tension steel and the surrounding concrete at the base of a LRCW, as shown in Figure 2a. A pseudo-static monotonic tension force was applied at each end of the prism to simulate the flexural tension in the end region of the LRCW. The crack development and ultimate width observed during the direct tension test could then be used to estimate the expected lateral load behaviour of a LRCW. The dimensions of the direct tension specimen are shown in Figure 2b. The D12 bars with the different deformation patterns were coupled to standard HD12 bars at the ends to ensure that fracture occurred within the test specimen. The cross-sectional area of the specimen was calculated to provide a reinforcement content that was constant with minimum vertical reinforcement limit for RC walls in NZS 3101:2006. A target concrete strength of 40 MPa was used for all specimens, and a notch was cast at midpoint to act as a crack initiator. A total of three direct tension tests were conducted for each bar type to account for any variability in the results.



a) Representative end region of a LRCW



b) Specimen details

Figure 2. Direct tension test specimen

The direct tension specimens were loaded until the reinforcement fractured. The crack width at the notched section was measured using two LVDT's, and photogrammetry was used to measure the width of any additional cracks that formed along the member. Strain gauges were not used to measure the steel strains because of the risk of interference with the bond characteristics. Instead, hardness testing was used to determine the strain profiles of the reinforcement. At the conclusion of each test, the D12 bars were removed and Rockwell hardness tests were performed at approximately 50 mm intervals along the bars. The hardness results were correlated to plastic strain by comparing the hardness to a predetermined hardness to strain profile, which is described in more detail below.

3 MATERIAL CHARACTERISTICS

3.1 Concrete cylinder tests

To determine the concrete compressive and tensile strength, cylinders were tested in accordance with NZS 3112-2:1986. The cylinders and test specimens were cast using the same 40 MPa concrete mix, and cured together under a damp cloth and polyethylene sheet. The average measured compressive strengths at the time of testing of the pull out and direct tension tests were 35 MPa and 44 MPa, respectively. The split cylinder tensile strengths at the time of testing of the pull out and direct tension tests were 4.0 MPa and 3.7 MPa, respectively.

3.2 Steel tensile tests

Tension tests were performed to evaluate the reinforcing steel material properties in accordance with AS 1391:2007. Each bar type was tested to strains of 5%, 10%, 15%, and bar rupture with the strained specimens prior to rupture used for determining the hardness-strain relationship. The steel tensile test results are shown in Figure 3. It is important to note that the standard and the custom bars were produced from different batches of steel with slightly different material properties. The standard bar lost the yield point as it was stored in coils and the ultimate stress of the custom bars appeared to be 10 MPa higher than the standard bar. Furthermore, the ultimate strain capacity of the custom bars was on average 3% greater than the standard bar. Despite small differences between the standard and custom bars, the performance of these bars was considered comparable.

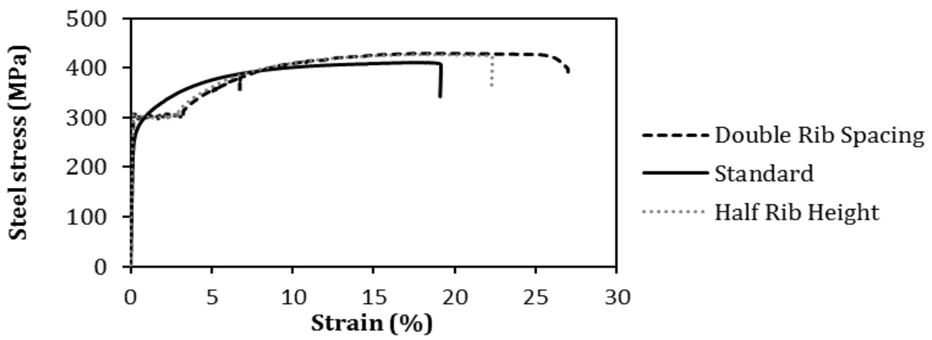
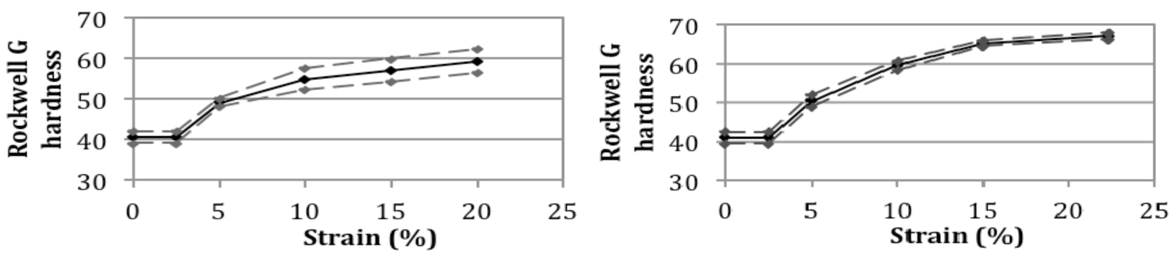


Figure 3. Steel tensile test, steel stress (MPa) against steel strain (%)

3.3 Rockwell hardness tests

When reinforcing steel plastically deforms, the steel strain hardens. By using this behaviour, the plastic strain of reinforcing steel can be approximated by correlating a hardness-strain relationship. The hardness-strain relationship was determined by conducting hardness tests on reinforcing steel that was strained to 2.5%, 5%, 10%, 15%, and failure, and one sample that was unstrained. Rockwell G hardness tests were conducted according to ASTM E18-12. The pre-strained lengths of reinforcing steel were cut into 60 mm segments and one side was filed down to a flat surface to allow a workable surface for the indenter.

Figure 4 shows the variation in steel hardness with strain, with a 95th percentile confidence interval of the true mean hardness reading for a given strain. The hardness-strain characteristic profiles of the custom bars and the standard bars were separated due to the differences in material properties. The data for the standard rib bar was considerably more scattered than the half rib height or double rib spacing bars, leading to difficulty in assessing true strain values. The hardness to strain characteristics obtained were used to estimate the strain of the direct tension specimens.



(a) Standard bars **(b) Half rib height and double rib spaced bars**

Figure 4. Hardness-strain relationships

4 PULL OUT TEST RESULTS

Figure 5 presents the data obtained from the pull out tests of standard (S), double rib spacing (D), and half rib height (H) specimens. All bars achieved a pull out failure as opposed to bar rupture or splitting failure. Figure 5 shows the initial force-slip behaviour for each rib pattern, which was obtained by calculating the average force of three specimens of each type of bars at discrete values of slip. The standard bar was able to maintain its initial stiffness for approximately twice the load compared to the half rib height or double rib spacing at approximately 20 kN. Furthermore, at 0.12 mm slip, the pull out load for half rib height and double rib spacing specimens were approximately 60% of the pull out load of the standard specimen. This suggests that the bond strength of the custom bars with half the projected rib area was approximately half the bond strength of a standard bar. These results are consistent with the findings of Kimura and Jirsa (1992) which concluded that bars with smaller projected rib area had weaker bond strength. These bars were therefore appropriate to test whether weaker bond strengths could increase yield penetration and crack widths.

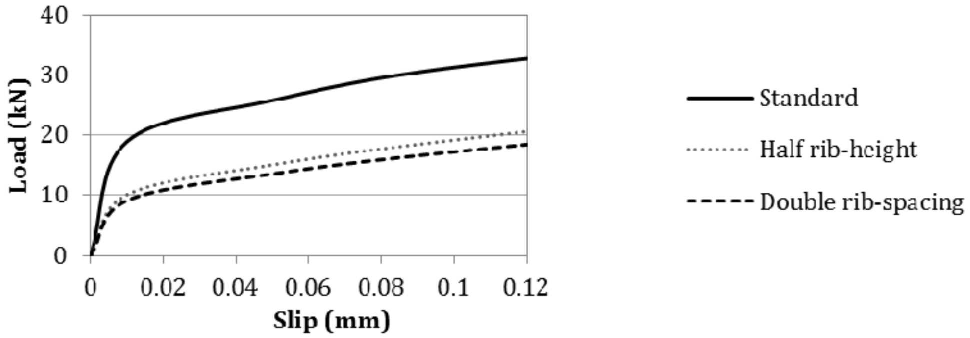


Figure 5. Pull out load (kN) versus free end slip (mm)

5 DIRECT TENSION TEST RESULTS

Figure 6 shows the representative samples of each deformation pattern immediately prior to failure. For all specimens, the first crack initiated at the notch at a load of approximately 20 kN. As the load increased above the yield point of the steel, secondary cracks formed approximately 250 mm above and below the central crack and the crack widths grew as the load increased. Uniquely for the standard specimens, additional secondary cracks and lateral bursting occurred near the centre notched crack. The additional secondary cracks formed because the standard bar had a shorter development length. Lateral burst occurred because the transverse spiral reinforcement terminated at midspan and the bursting stress caused the ends of the spiral to slip and become ineffective. Bursting did not occur for the half rib height or double rib spaced bars because the bursting stresses develop farther from the ends of the transverse spiral reinforcements due to the longer development length.

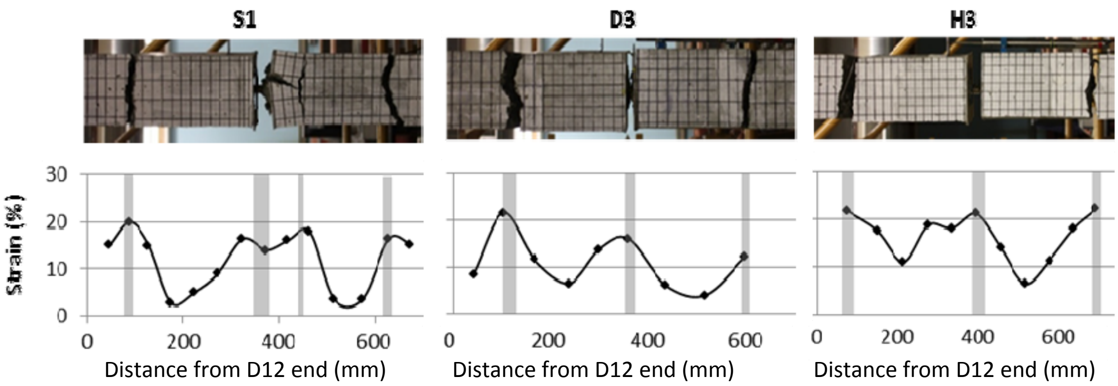


Figure 6. Hardness and strain profile of representative samples

The behaviour of the direct tension tests were further investigated by examining the three types of instrumentation used, consisting of photogrammetry, LVDT's, and hardness tests. Because the LVDT's typically detached from the specimen prematurely, the photogrammetry provided more reliable measurement of crack widths prior to failure. Table 2 shows the ultimate crack widths for each specimen calculated from the photogrammetry. The subsequent cracks caused by lateral bursting for the standard bar specimens were also included in the centre crack width. It was observed that one of the H1 secondary cracks occurred at the location of a coupler. Since the reinforcing steel is stiffer at this location the crack width was smaller. H1 was therefore treated as an outlier and was not considered when interpreting the results. The average total elongation of the half rib height specimens were 44% larger, and the double rib spacing specimens were 30% smaller, than that of the standard deformation specimens. The half rib height specimens performed as expected according to the hypothesis, that weaker bond allowed for greater yield penetration. However, the double rib spacing specimens contradict this hypothesis because larger yield penetration was expected as it had lower bond strength according to the pull out test results. It is possible that there is a slipping mechanism that the pull out test was unable to capture, as discussed in detail below.

Table 2 Ultimate crack widths measured by photogrammetry

Specimen	Top crack width (mm)	Centre crack width (mm)	Bottom crack width (mm)	Total elongation (mm)	Average total elongation (mm)
S1	16.5	43.3*	16.1	75.9	
S2	22.8*	32.5	19.4	74.7	
S3	12.8	35.9*	10.3	59.0	69.9
D1	7.8	18.4	24.1*	50.3	
D2	25.3*	16.8	3.3	45.4	
D3	21.2*	16.3	14.4	51.9	49.2
H1	8.6	25.9*	19.3	53.8	
H2	30.0	23.0	50.8*	103.8	
H3	36.2*	36.5	24.5	97.2	100.5

** Location of reinforcing steel fracture*

Figure 7 shows the plot of the tensile load against the centre crack width for the representative samples measured by the LVDT's. The centre crack width remained relatively small until the yield strength of the reinforcing steel was reached at 34 kN. As reinforcing steel plastically strained, the crack widths increased. The double rib spaced bar fractured at the smallest crack width and standard bar fractured at the largest crack width. However, the larger crack width of the standard specimen was due to the LVDT measuring multiple cracks. The two custom bars appeared to follow similar crack width growth rates up to 12 mm. This similarity suggests that at lower steel strains, there is little difference in how the steel slips and strains within the concrete. However at large steel strains, the double rib spacing bars are unable to slip in the concrete yet the half rib height bars could.

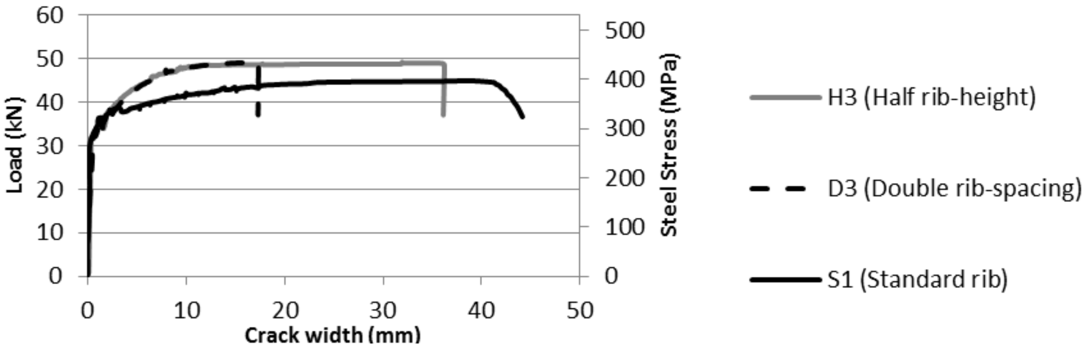


Figure 7. Load vs. centre crack width of representative specimens

The reduction in diameter of the bar as plastic elongation occurs could be a contributing factor for the difference between the half rib height and double rib spacing specimens, despite the pull out test suggesting that the bars had similar bond characteristics. As shown in Figure 8, the reduction in the diameter of the bar as the strain of the bar increases, reduces the effective rib height. At large strains the effective rib height of the half rib height bars could be reduced to zero, allowing the bar to slip freely within the concrete causing the strains to penetrate further. The same effect would be less likely to occur for the standard or double rib spaced bars because the rib heights are two times larger than that of the half rib height bar.

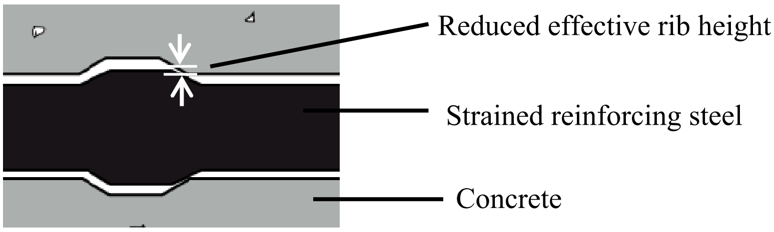


Figure 8. Reduction in effective rib height when bar elongates

The reinforcement strain profiles obtained from the hardness tests for the representative samples are also shown on Figure 6. The ultimate strains occurred at the crack openings and the strain gradually decreased going into the uncracked concrete as the tension was transferred from the reinforcing steel to the concrete. The strains for the half rib height bars appeared to be maintained a relatively high values within the uncracked sections, whereas both double rib spacing and standard bars dropped to approximately 5%. These results indicate that the strains of the half rib height bars were able to penetrate further into the concrete for the member to obtain greater crack widths. A plot of the steel strains for increasing distances from the nearest crack opening is shown in Figure 9. Near the crack face, the strains were similar between the different bar types. However, the half rib height strains were approximately two times larger than the others at locations greater than 50 mm from the nearest crack face, confirming that the half rib height bars has greater yield penetration.

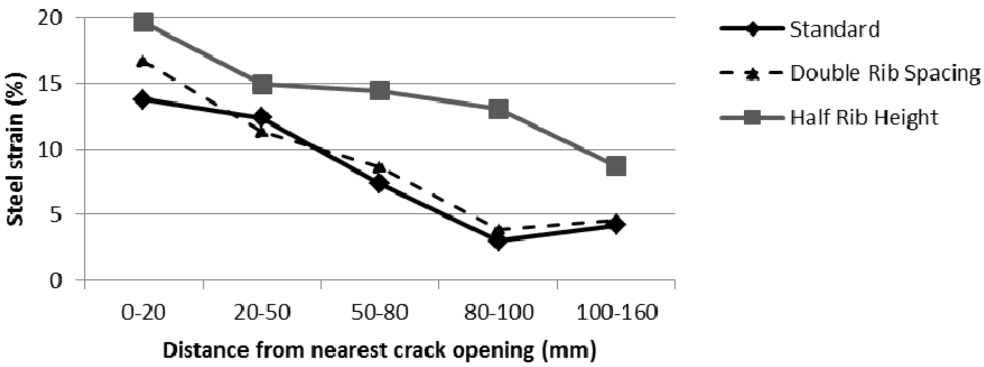


Figure 9. Steel strain relative to the nearest crack opening

6 IMPLICATIONS TO WALLS

Since the ultimate crack width at the base of the wall is proportional to the plastic rotation of the wall (Davey and Blaikie, 2005), the results show that the half rib height bars are able to achieve the greatest plastic rotations if only a single crack formed. On the other hand, the double rib spacing bar showed similar yield penetration and crack widths in comparison to the standard bar, and it is expected that no additional ductility would be gained by adopting double rib spaced bars. The direct tension results also showed that all specimens formed secondary cracks when designed to the minimum vertical reinforcement requirements in NZS 3101:2006. However, since standard bars were able to transfer the stresses to the concrete over a shorter length than the half rib height bar, a greater number of secondary cracks formed. The standard bar may result in smaller individual crack widths than the half height rib bars, but the combined total elongation in the plastic hinge region may be greater due to the increased distribution of secondary cracks. There appears to be a trade-off between achieving a greater number of cracks and achieving greater yield penetration at a single crack. Further research is required to investigate at what vertical reinforcement content it may be appropriate to adopt half rib height bars to achieve greater ductility.

7 CONCLUSIONS

Pull out tests confirmed that the bond strength of the reinforcing steel bars with half rib height and double rib spaced deformations were approximately 60% of the bond strength of bars with standard deformation patterns. However, it was found that low bond strength measured by the pull out test might not correspond to larger ultimate crack widths. The increase in yield penetration of half rib height bars may have been exacerbated because the ribs were able to freely slip from the concrete at high plastic strains.

The ultimate crack width at the base of a lightly reinforced concrete wall could be increased by 44% if half rib height bars were adopted. Potentially, the increase in the ultimate crack width could lead to a 44% improvement in plastic rotation if a single crack formed in the plastic hinge region of the wall. No measured benefit was gained from using deformation patterns with doubling the rib spacing. Adopting reinforcing steel bars with half rib height deformations may result in improved ductility for lightly reinforced shear walls that are susceptible to the formation of a single flexural crack. However, further research is required to investigate at what vertical reinforcement content standard bars may offer improved performance due to the formation of a greater number of secondary cracks.

8 ACKNOWLEDGMENTS

Financial support for this research was provided by the Natural Hazards Research Platform through contract C05X0907. The authors would also like to thank Pacific Steel Group for their support of this project and donation of the specially manufactured bars, as well as Stresscrete Northern Ltd for their help constructing the test specimen.

REFERENCES

AS 1391:1991. Methods for tensile testing for metals. Standards Australia, Sydney, NSW.

AS/NZS 4671:2001. Steel reinforcing materials. Standards Australia, Sydney, NSW,. Co-published by Standards New Zealand.

ASTM E18-12:2012. Standard test methods for Rockwell hardness of metallic materials. ASTM International, West Conshohocken, United States.

Davey, R. and Blaikie, E. 2005. On the flexural ductility of very lightly reinforced concrete sections, *New Zealand Society for Earthquake Engineering Conference*, Taupo, New Zealand, Mar 11-13.

Haskett, M., Oehlers, D., Mohammed Ali, M. and Wu, C. 2009. Yield penetration hinge rotation in reinforced concrete beams, *Journal of Structural Engineering,* Vol 135(2): 130-138.

Henry, R. 2013. Assessment of minimum vertical reinforcement limits for RC walls, *Bulletin of the New Zealand Society for Earthquake Engineering*, Vol 46(2): 88-96..

Structural Engineering Society of New Zealand (SESOC). 2011. Practice note - design on conventional structural systems following the Canterbury earthquakes. http://canterbury.royalcommission.govt.nz/documents-by-key/20111221.1908, Report prepared for the Canterbury Earthquakes Royal Commission.

Kimura, H. and Jirsa, J. 1992. Effects of bar deformation and concrete strength on bond on reinforcing steel to concrete. *International Conference on Bond in Concrete*, Riga, Latvia, Oct.

NZS 3101:2006. Concrete Structures Standard. Standards New Zealand, Wellington, New Zealand.

NZS 3112-2:1986. Methods of test for concrete. Standards New Zealand, Wellington, New Zealand.

Smith, P. and England, V. 2012. Independent assessment on earthquake performance of Gallery Apartments - 62 Gloucester Street. http://canterbury.royalcommission.govt.nz/documents-by-key/20120217.3188, Report prepared for the Canturbury Earthquakes Royal Commission.