MINIMUM FLEXURAL REINFORCEMENT IN REINFORCED CONCRETE WALLS

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Abstract

Current design practices for reinforced concrete (RC) flexural members to resist bending moments promote the use of "under-reinforced" sections. It is believed that sections with less reinforcement are more ductile and have better deformation capacity. The availability of stronger steel (requiring smaller amounts of reinforcement), failures in buildings with walls with small reinforcement ratios, and laboratory evidence showing that less reinforcement does not always lead to more deformability suggest that the deformation capacity of elements with small reinforcement ratios (ranging from 0.07% to 0.25%) needs to be reexamined.

Tests of four RC walls were conducted to investigate the minimum amount of conventional (yield stress ≤ 60 ksi (414 MPa)) or high-strength (yield stress ≥ 100 ksi (690 MPa)) longitudinal reinforcement needed so that bar fracture does not limit drift capacity to an intolerable value. The walls were 8 in. (203 mm) thick and 40 in. (1016 mm) long. Their aspect ratio was 1.8. They were tested monotonically up to failure and had no axial load. All test walls failed because of fracture of longitudinal reinforcement at drift ratios less than 1%.

Keywords: Minimum Reinforcement; Walls; High-Strength Steel Reinforcement; Longitudinal

1. Introduction

Current design practices for reinforced concrete (RC) flexural members to resist bending moments promote the use of "under-reinforced" sections. It is believed that sections with less reinforcement are more ductile and have better deformation capacity. According to the current American Concrete Institute building code provisions (ACI 318-14) [1], the minimum gross longitudinal reinforcement ratio (ρ) for ordinary walls should be between 0.12% and 0.15% and that for special walls should be 0.25%.

At low ρ, cracking moment of a cross-section can be larger than the nominal yield moment. This could lead to sudden brittle failure at low drift ratios because of concentration of strains at a single crack or a few cracks. Observations made in the field after earthquakes in New Zealand and Chile have shown that walls with low amounts of reinforcement can fail because of bar fracture in the region with one or few cracks [2, 3]. Past experimental investigations [2, 4] have also shown that walls and beams with low reinforcement ratios are susceptible to failure caused by fracture of longitudinal reinforcement.

Using past field and test data, a method for screening lightly reinforced walls that might be vulnerable to fracture of longitudinal reinforcement was proposed by Wood [2]. And walls with low reinforcement ratios and conventional steel (ρ= 0.53%, fy= 43 ksi (300 MPa)), designed based on the provisions in the New Zealand Concrete Structures Standard (NZS 3101:2006) [5], are being tested at The University of Auckland in New Zealand [6].

In walls with low reinforcement ratios fracture of longitudinal bars can take place at first fracking. If that is not the case, the wall can reach its flexural capacity, but bar fracture may follow at a small drift. To our knowledge, there are no methods available to estimate this drift because the profession has focused on failures caused by compression instead of tension.

The potential introduction of high-strength steel reinforcement (HSSR) with yield stress (f_y) larger than 100 ksi (690 MPa) also requires us to reconsider the requirement for minimum longitudinal reinforcement. Use of high-strength steel would lead to a reduction in the quantity of steel required in a section designed to have strength comparable to that of a section with conventional reinforcement ($f_y \leq 60$ ksi (414 MPa)). If the quantity of longitudinal reinforcement in walls is reduced in inverse proportion to the increase in yield stress (up to 120 ksi (830 MPa)), the required minimum reinforcement ratio would reduce to approximately 0.07% for ordinary walls and 0.13% for special walls. Experimental investigations on the use of HSSR in columns, beams and slabs have been conducted in the past [7, 8, 9, 10, 11, 12]. To our knowledge, available test data do not cover the behavior of walls with low amounts of longitudinal HSSR.

Four reinforced concrete wall specimens were designed and tested to investigate the response of walls with minimum amounts of conventional or high-strength longitudinal reinforcement. A preliminary method to estimate drift capacity of lightly reinforced walls in which failure is controlled by fracture of longitudinal reinforcement is presented.

2. Experimental Program

Four reinforced concrete wall specimens were tested at Bowen Laboratory for Large-Scale Civil Engineering Research at Purdue University to investigate the minimum amount of conventional or high-strength longitudinal reinforcement needed so that fracture of longitudinal reinforcement does not limit drift capacity to an intolerable value.

2.1 Test Specimens

The specimens were 8 in. (203 mm) thick, 40 in. (1016 mm) long and 14 ft. (1.22 m) tall. They were rotated 90 degrees for testing convenience. Load was applied at mid-height such that the effective height was equal to 6 ft (Fig. 1 and 3). Wall aspect ratio was 1.8. The variables in the tests were the grade and quantity of longitudinal steel reinforcement. The specimens were reinforced in the longitudinal direction with 2, 4 or 7 #3 ASTM A615 Gr. 60 or ASTM A1035 Gr. 120 deformed reinforcing bars. The bars were spaced equally with a 4 in. (102 mm) cover on the sides and the ends as shown in Fig. 1. The longitudinal reinforcement ratio, defined as the ratio of total area of steel in the longitudinal direction to gross cross-sectional area, varied between 0.07% and 0.24%. The specimens with Gr. 120 steel (W1-120-0.07 and W1-120-0.14) had approximately half as much longitudinal reinforcement as the specimens with Gr. 60 steel (W1-60-0.14 and W1-60-0.24 respectively). No transverse reinforcement was used. The specimens were designed such that the shear capacity of the section without any transverse reinforcement was larger than the maximum expected shear demand. Table 1 presents the properties of the test specimens. Fig. 2 shows the measured stress-strain curves for the two types of longitudinal reinforcement used. Cracking moment (M_{cr}) of the cross section was estimated using gross section properties and measured concrete properties (reported in Section 3). Nominal moment capacity (M_n) was estimated at a limiting concrete compressive strain of 0.003 using the measured properties of steel presented in Fig. 2.

Additional details including material properties, specimen descriptions, test setup, instrumentation, drawings, and test data are available at datacenterhub.org/resources/284.

Table 1 – Test specimens and variables

Specimen	Nominal Yield Stress, f_y, ksi (MPa)	No. of #3 Bars	Longitudinal Reinforcement Ratio, ρ = As/Ag, %	ρf_y, ksi (MPa) [f_y=Nominal Yield Stress]	M_{cr}/M_n
W1-120-0.07	120 (830)	2	0.07	8.3 (57)	1.8
W1-120-0.14	120 (830)	4	0.14	16.5 (114)	1.0
W1-60-0.14	60 (414)	4	0.14	8.3 (57)	1.8
W1-60-0.24	60 (414)	7	0.24	14.4 (100)	1.2

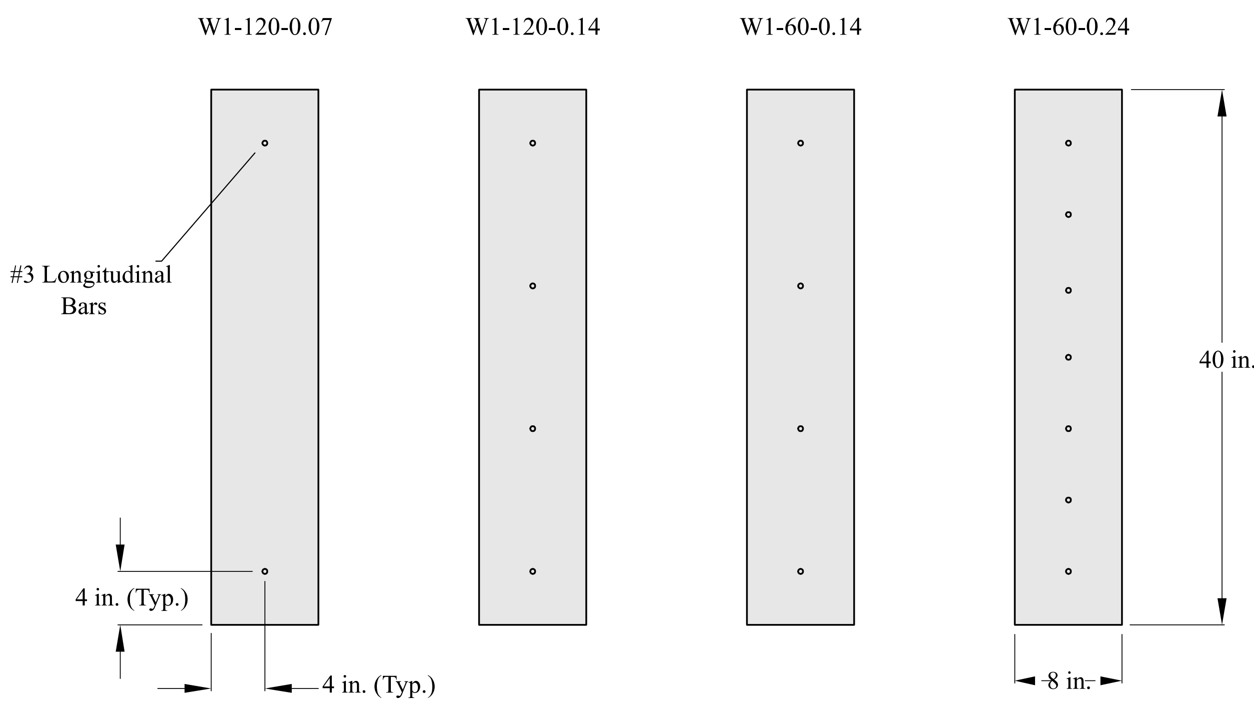


Fig. 1 – Cross-Sections

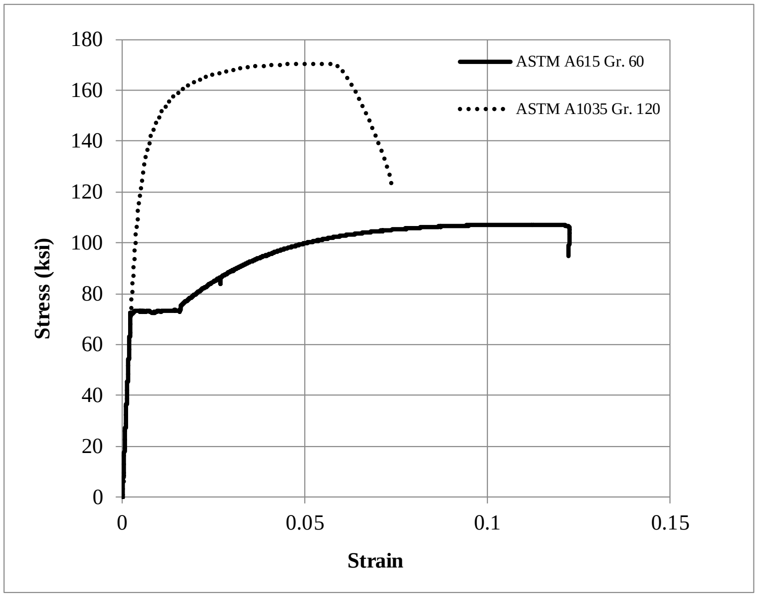


Fig. 2 – Stress vs. strain curves for longitudinal reinforcement

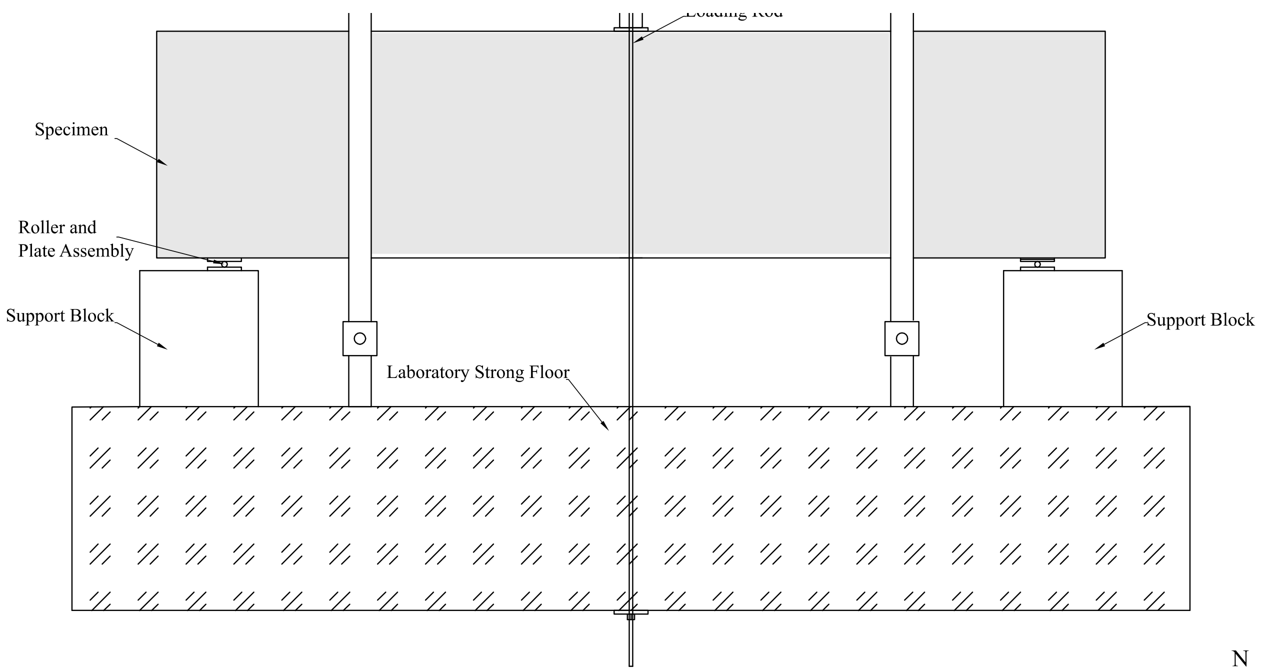


Fig. 3 – Test setup

2.2 Test Setup, Instrumentation and Procedure

The test setup is shown in Fig. 3. Specimens spanned 12 ft (3.66 m) between simple supports with their height oriented in the North-South direction of the laboratory. Four HSS steel tubes were used as out-of-plane bracing. Load was applied at mid-span using two 1 in. post tensioning threaded rods and two 40 kip (20 ton) center-hole hydraulic rams. The hydraulic rams were connected to the same manifold and controlled using a hand-pump. The applied load was measured using two load cells. Displacements were measured at seven locations along the span using linear variable differential transformers (LVDTs). The LVDTs were placed at mid-span and at 2 ft (0.61 m), 3 ft 4 in. (1.02 m) and 6 ft (1.83 m) to the North and South of the mid-span. An optical measurement system (Optotrak Pro Series 600) was used to track the three-dimensional coordinates of infrared targets placed on the specimen. Each specimen was tested monotonically up to failure. No axial load was applied.

3. Results and Discussion

Table 2 presents a summary of the test results. Drift ratio is defined as the ratio of displacement measured at the load point to the distance from the load point to support (which is the effective height of the wall). The displacement and drift ratio at cracking and bar fracture are reported. It should be noted that the maximum applied load reported in Table 2 was larger than the load resisted by the walls at bar fracture. The load vs. drift ratio curves are shown in Fig 4. Table 3 lists the measured concrete properties and steel yield stress (estimated using the 0.2% offset method). The data in Table 3 were obtained in accordance with ASTM Standards [13, 14, 15, 16]. Fig. 5 shows one of the four tested specimens.

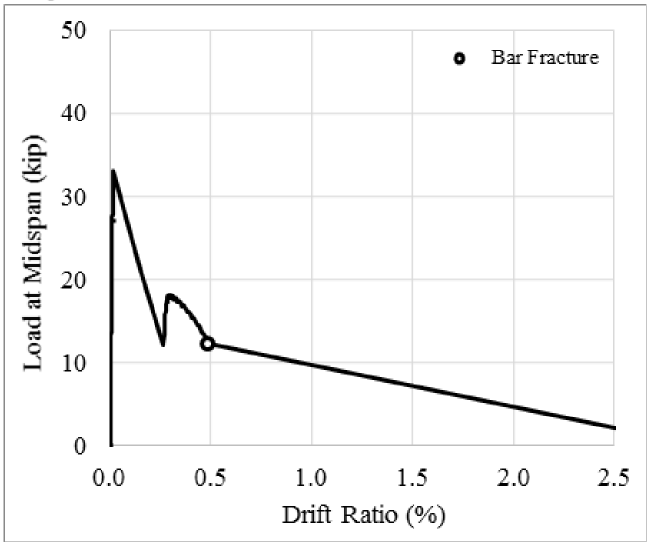
Drift ratio at cracking ranged between approximately 0.01 and 0.02%. There was only one flexural crack in all specimens. Failure in all specimens was caused by bar fracture at the location of flexural cracking (Fig. 5). Bars fractured at a drift ratio of approximately 0.5% for both specimens with Gr. 120 reinforcement and at 0.85% for the specimens with Gr. 60 reinforcement. The drift ratio at which bars fractured (for specimens with the same grade of steel) was not sensitive to longitudinal reinforcement ratio.

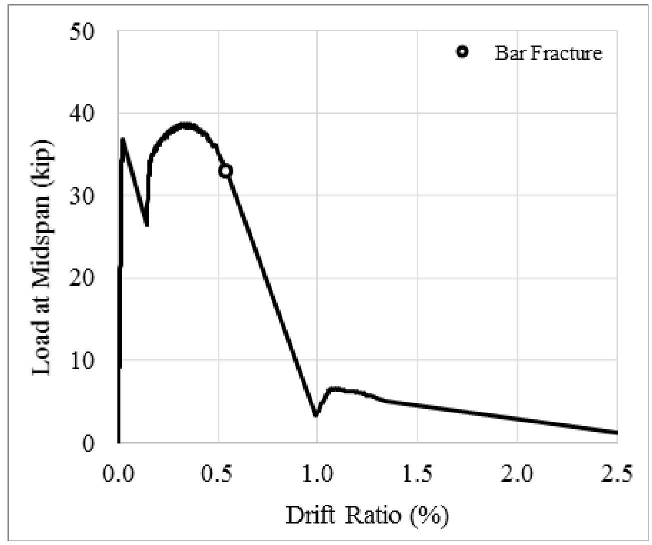
Table 2 – Summary of test results

Specimen Name	Maximum Applied Load *, kip, (kN)	Mid-span Deflection at First Crack, in. (mm)	Drift Ratio at First Crack, %	Mid-span Deflection at Bar Fracture, in. (mm)	Drift Ratio at Bar Fracture, %
W1-120-0.07 (Set 1)	33 (150)	0.013 (0.33)	0.017	0.35 (9)	0.49
W1-120-0.14 (Set 2)	39 (170)	0.015 (0.38)	0.021	0.39 (10)	0.54
W1-60-0.14 (Set 1)	33 (150)	0.012 (0.30)	0.017	0.59 (15)	0.82
W1-60-0.24 (Set 2)	47 (210)	0.010 (0.25)	0.014	0.62 (16)	0.86
* The reported maximum applied load does not include weight of the loading equipment (2.5 kip) and self-weight of the specimen (4.7 kip)					

Table 3 – Measured concrete and steel properties

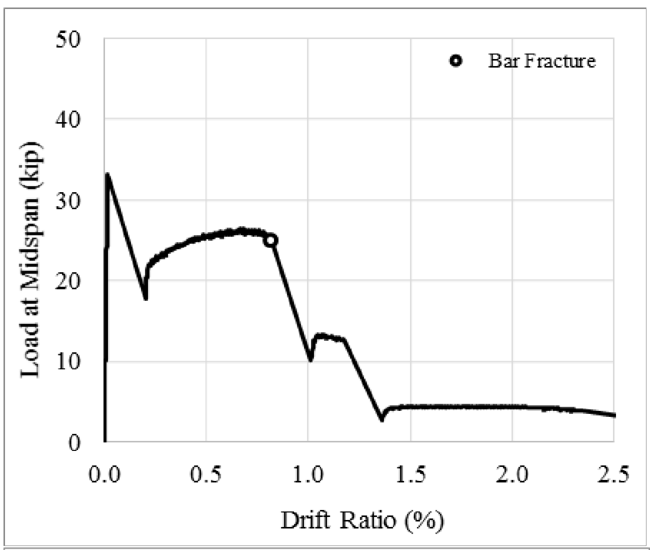
Specimen Name	Test Date	Compressive Strength, f_c', psi (MPa)	Split Tensile, psi (MPa)	Split Tensile, $\sqrt{f_c'}$ f_c' in psi	Modulus of Rupture, psi (MPa)	Modulus of Rupture, $\sqrt{f_c'}$ f_c' in psi	Measured Yield Stress, ksi (MPa)
W1-120-0.07	26-Oct-15	7600 (52)	550 (3.8)	6.3	690 (4.8)	7.9	135
W1-120-0.14	19-Nov-15	7800 (54)	600 (4.1)	6.8	-	-	135
W1-60-0.14	2-Nov-15	7800 (54)	550 (3.8)	6.2	-	-	76
W1-60-0.24	30-Nov-15	8000 (55)	520 (3.6)	5.8	-	-	76

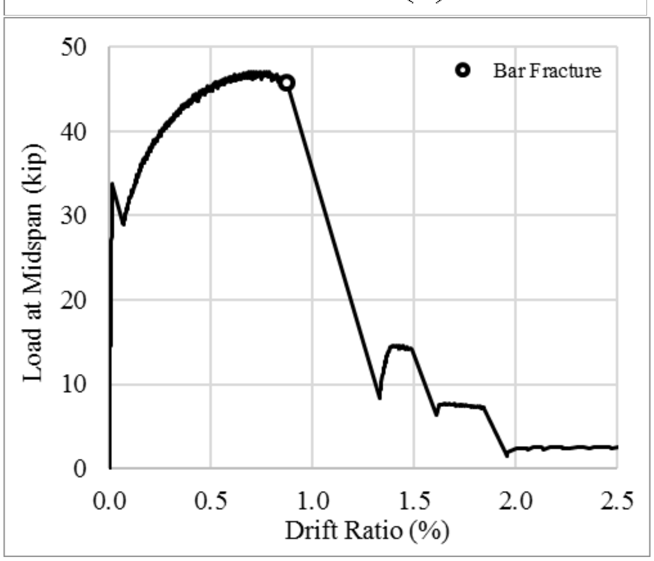




a) W1-120-0.07

b) W1-120-0.14





c) W1-60-0.14

d) W1-60-0.24

Fig. 4 – Load vs. drift ratio

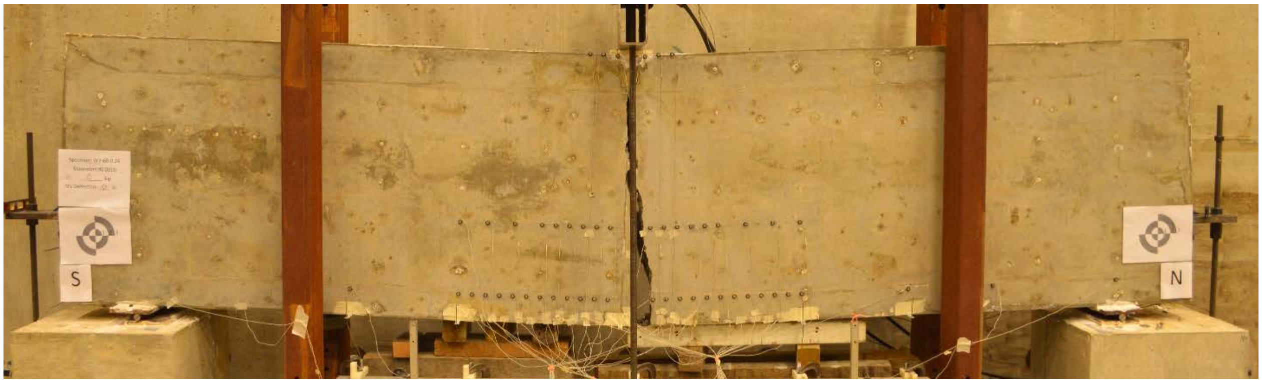


Fig. 5 – Specimen W1-60-24

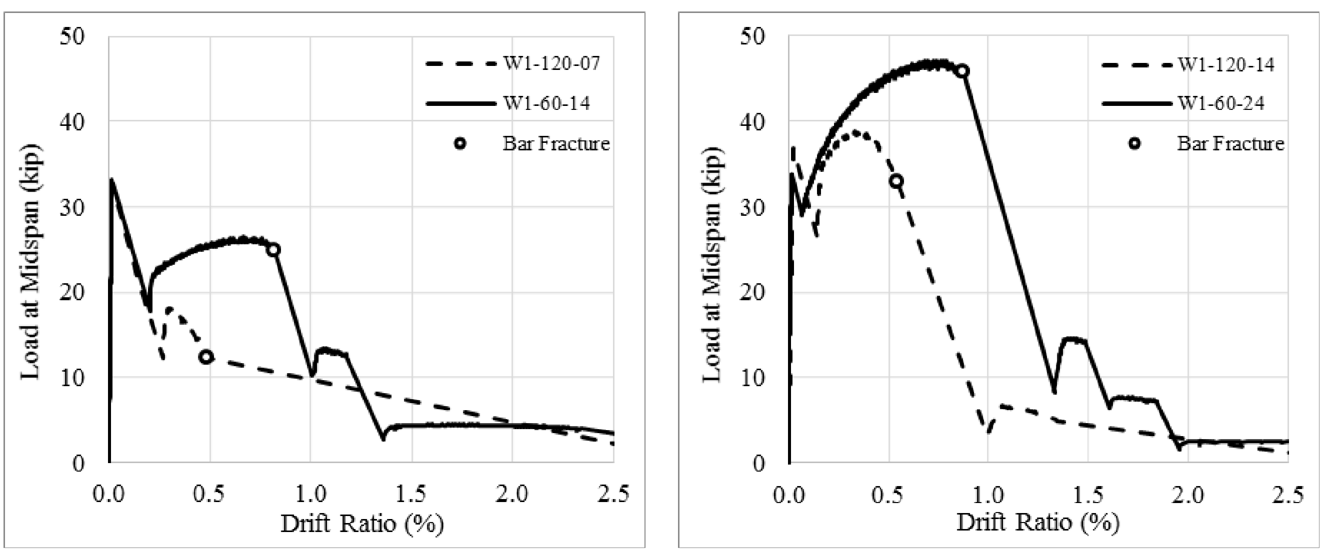
In Fig.6, test results are compared for two sets of two walls each:

Set 1- Specimens W1-120-0.07 and W1-60-0.14 (Figs. 4a, 4c and 6a)

Set 2- Specimens W1-120-0.14 and W1-60-0.24 (Figs. 4b, 4d and 6b)

Each set has one specimen with conventional Gr. 60 longitudinal reinforcement and one with Gr. 120 longitudinal reinforcement. The specimen with Gr. 120 steel had approximately half as many reinforcing bars of the same size as the specimen with Gr. 60 steel. Both specimens in each set were designed to have similar nominal strengths.

Both specimens in each set reached similar loads just before cracking. There was a sudden drop in load in both specimens after the formation of the first crack. The longitudinal bars fractured at a load lower than 80% of the maximum load at first cracking in set 1. Specimens in set 2 resisted loads larger than the load at cracking. The longitudinal bars fractured at a load larger than or approximately equal to the load at cracking in set 2. On average, the specimens with Gr. 60 longitudinal reinforcement resisted larger loads and failed at larger drift ratios than those with approximately half as much Gr. 120 longitudinal reinforcement.



a) Set 1 b) Set 2

Fig. 6 – Comparison of load vs. drift ratio

A preliminary method to estimate drift capacity at bar fracture was explored. All deformation was assumed to concentrate around a single crack over a length of six bar diameters. Drift ratio was estimated as follows:

$$DR = \frac{\varepsilon_{su}}{d} 6d_b \quad (1)$$

Where *DR* is the drift ratio, ε_{su} is measured fracture strain from coupon tension tests, d is distance from the compression fiber to the centroid of the outermost layer of tension steel and d_b is nominal diameter of the longitudinal bars.

Drift capacities at bar fracture for the four specimens tested were estimated using Eq. 1. Fracture strains (ε_{su}) were obtained from tensile tests of three coupons for each type of steel used, conducted in accordance with ASTM A370 [15]. Reinforcing bars cut to 3 ft lengths and marked every 8 in. (203 mm) were tested using a Baldwin 120-kip universal testing machine. Elongation after rupture was measured over the marked 8 in. (203 mm) gage length. Average ε_{su} (from tests of three coupons) for Gr. 60 reinforcement was 15% and that for Gr. 120 reinforcement was 7%. d was 36 in. and d_b was 3/8 in. (for a #3 deformed bar). Fig. 7 shows the calculated and measured drift capacities.

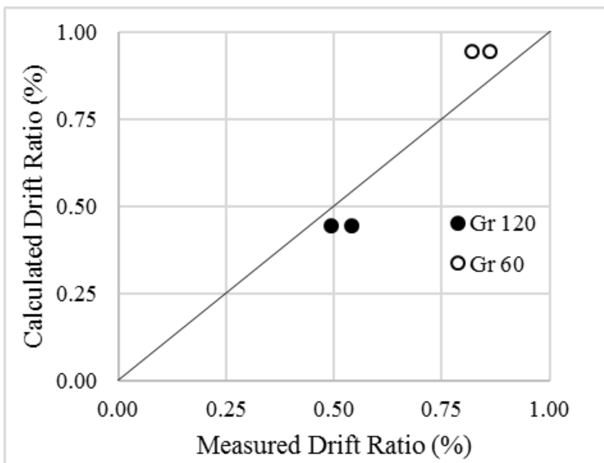


Fig. 7 – Measured vs. Calculated Drift Ratio at Bar Fracture

The length over which deformation is assumed to be concentrated is expected to vary based on the number of cracks that form at the base of the wall. Deformation may spread over a larger distance (> $6d_b$) if failure does not follow soon after initial cracking. More test data are needed to study whether the length over which deformation is assumed to be concentrated depends on longitudinal bar diameter (d_b).

The method to estimate drift capacity presented here is preliminary and based on the limited test data from this experimental investigation. The results should not be extrapolated to other walls outside the ranges described in Tables 1 and 3 without using more test data.

4. Conclusion

Four reinforced concrete wall specimens were tested to investigate the minimum amount of conventional ($f_y \leq 60$ ksi (414 MPa)) or high-strength ($f_y \geq 100$ ksi (690 MPa)) longitudinal reinforcement needed so that fracture of longitudinal reinforcement does not limit drift capacity to an intolerable value.

Walls within the ranges described in Tables 1 and 3 were tested monotonically and without axial load. Longitudinal reinforcement ratios varied from 0.07% to 0.24%. The product of reinforcement ratio (%) and nominal yield stress, ρf_y, was between 8.3 ksi (57 MPa) and 16.5 ksi (114 MPa). For specimens with $\rho f_y \leq 8.3$ ksi (57 MPa) and $M_{cr}/M_n \geq 1.8$ failure took place just after first cracking and at loads lower than that at first cracking suggesting that walls in this range should be avoided in all structures no matter how low the probability of cracking is estimated to be. Specimens with 14.4 ksi (100 MPa) ≤ ρf_y ≤ 16.5 ksi (114 MPa) and $M_{cr}/M_n \leq 1.2$ resisted loads larger than those at first cracking. In all cases failure was controlled by bar fracture at drift ratios less than 1%.

Drift ratios at bar fracture for the four specimens tested were close to the product of ultimate strain ($\boldsymbol{\varepsilon_{su}}$) and $6\boldsymbol{d_b/d}$ where ultimate strain is the strain measured at fracture over an 8in. gage length in a coupon tension test, $\boldsymbol{d_b}$ is the longitudinal bar diameter and d is the distance from the compression fiber to the centroid of the outermost layer of tension steel. Eq. 1 should not be extrapolated to walls outside the ranges described in Tables 1 and 3 without using more test data.

The data presented are too limited to generalize the test results. The results presented here are to alert the profession about risks related to using walls with low reinforcement ratios.

5. Acknowledgements

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