

Comparison of Existing Minimum Vertical Reinforcement Requirements for Ductile Reinforced Concrete Walls

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Biography

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Abstract

Recent research and post-earthquake observations have raised doubts over the crack distribution and deformation capacity of reinforced concrete (RC) walls with vertical reinforcement contents close to minimum requirements. To investigate the seismic behaviour of lightly reinforced concrete walls, the minimum vertical reinforcement requirements for ductile RC walls from concrete design standards worldwide were first examined. A series of numerical analyses were conducted to compare these requirements using a finite element model that was developed and validated using existing test data. The results from the analyses showed that RC walls designed in accordance with requirements for minimum distributed vertical reinforcement did not generate a large number of cracks within the plastic hinge region. Lumping additional reinforcement at the ends of the wall, as

required by some design standards, generally improved the distribution of cracking and ductility. However, lumping too much reinforcement at the ends of the wall resulted in the web region being vulnerable to the development of large widely spaced cracks that may contribute to poor seismic performance. Therefore it is recommended that the ratio between lumped reinforcement and distributed web reinforcement be controlled to prevent premature web reinforcement fracture. Furthermore, concrete and reinforcing steel strengths were shown to significantly influence the crack pattern and deformation capacity of the RC walls with minimum vertical reinforcement and should be accounted for when developing design standard requirements.

Key words: reinforced concrete; wall; seismic design; finite element model; minimum vertical reinforcement; drift capacity; reinforcement fracture; design standards.

INTRODUCTION

Reinforced concrete (RC) walls with minimum required vertical (or longitudinal) reinforcement are common when the dimensions of the wall are larger than that required for strength, or when axial loads provide sufficient flexural capacity. Requirements of minimum vertical reinforcement for RC walls are imposed by most concrete design standards worldwide, in part to mitigate shrinkage and temperature effects, but also to prevent non-ductile failure modes¹. For a wall with an extremely low quantity of vertical reinforcement, the cracking moment may exceed the nominal flexural strength, and inelastic action is unlikely to extend beyond the section at which the first crack forms². This behaviour is non-ductile with strength dropping immediately after first cracking and should be prevented for all walls designed to resist seismic actions. In addition, RC walls designed to exhibit ductility during earthquakes must have sufficient vertical reinforcement for well distributed secondary flexural cracks to develop to ensure a good spread of plasticity. Insufficient vertical reinforcement can result in behaviour characterised by a limited number of cracks in the plastic hinge region, reducing the inelastic deformation capacity, and resulting in concentrated strains that

1 may lead to premature fracture of vertical reinforcement ³.

2 The potential deficiencies of RC walls with minimum vertical reinforcement was highlighted
3 previously by Wood⁴ following the 1985 Chilean Earthquake, during which an eight-story lightly
4 reinforced concrete wall building experienced collapse with fracture of vertical reinforcement
5 occurring. Wood⁵ analysed the results of 37 RC walls tested prior to 1985, and concluded that walls
6 with a total vertical reinforcement content less than 1% were susceptible to premature fracture of
7 vertical reinforcement. More recently, similar behaviour of lightly reinforced concrete walls was
8 observed during the 2010/2011 Canterbury earthquakes in New Zealand. Several RC walls in multi-
9 storey buildings, designed in accordance with minimum vertical reinforcement requirements from
10 former versions of the New Zealand Concrete Structures Standard (NZS 3101:1995)⁶, formed only
11 a limited number of cracks in the plastic hinge region as opposed to the expected distributed
12 cracking. In response to the observed performance of lightly reinforced concrete walls in
13 Christchurch, doubts were raised regarding whether current minimum vertical reinforcement
14 requirements were sufficient to generate a large number of secondary cracks in the plastic hinge
15 regions^{2, 7}.

16 Although each concrete design standard has specific minimum vertical reinforcement limits for RC
17 walls, these requirements appear to be mostly based on engineering experience and may not be
18 sufficient to ensure good seismic behaviour. For example, Lu et al.³ tested six RC walls designed in
19 accordance with the current minimum vertical reinforcement requirements in NZS 3101:2006
20 (Amendment 2), and found that the behaviour was controlled by 1-3 large primary flexural cracks at
21 the wall base. In addition, modelling results presented by Lu and Henry⁸ further highlighted that the
22 behaviour of lightly reinforced walls was significantly influenced by wall size, reinforcement
23 properties, and concrete strength, which are not always considered in minimum vertical
24 reinforcement requirements.

1 An investigation was conducted to comprehensively evaluate and compare the behaviour of RC
2 walls with minimum vertical reinforcement in accordance with different concrete design standard
3 requirements. A summary of minimum vertical reinforcement limits for RC walls from different
4 standards worldwide is presented and a finite element model was used to conduct a series of
5 numerical analysis that investigated RC wall behaviour, including lateral load response, crack
6 pattern, reinforcement strains, and failure mode.

7 **RESEARCH SIGNIFICANCE**

8 Significance attention has been given to minimum vertical reinforcement requirements for RC walls
9 following the observed poor performance of lightly reinforced concrete walls during recent
10 earthquakes. An investigation was conducted to comprehensively evaluate and compare the
11 behaviour of RC walls with minimum vertical reinforcement in accordance with different concrete
12 design standard requirements. recommendation from this research can be used to inform
13 amendments to minimum vertical reinforcement requirements for ductile RC walls that resist
14 seismic actions.

15 **MINIMUM VERTICAL REINFORCEMENT REQUIREMENTS**

16 The minimum vertical reinforcement requirements for ductile seismic load resisting RC walls in six
17 different concrete standards were compared and a summary is shown in Table 1. The standards
18 investigated included the US Building Code Requirements for Structural Concrete, ACI 318-14
19 (ACI)⁹, New Zealand Concrete Structures Standard, NZS 3101:2006 Amendment 2 (NZS-A2)¹⁰ and
20 NZS 3101:2006 Amendment 3 (NZS-A3)¹¹, Eurocode 8: Design of structures for earthquake
21 resistance (Eurocode)¹², the Canadian Design of Concrete Structures standard, CSA A23.3-14
22 (CSA)¹³, and the Chinese Code for Design of Concrete Structures, GB 50010-2010 (GB)¹⁴. For
23 NZS-A2, the distributed reinforcement ratio refers to total reinforcement ratio, calculated by
24 $A_t/(b_w l_w)$, in which A_t is the total reinforcement area, and b_w and l_w are the width and length of the

1 wall respectively. For ACI, NZS-A3, Eurocode, CAS and GB, the distributed reinforcement ratio
2 refers to the reinforcement ratio between the wall end zone or boundary element (the region in
3 which concentrated vertical reinforcement is placed), calculated by $2A_s/(b_w s)$ for doubly reinforced
4 walls, in which A_s is the area of one single bar, and s is the spacing of the distributed vertical
5 reinforcement.

6 As shown in Table 1, ACI requires a minimum distributed vertical reinforcement ratio of 0.25% for
7 special structural walls designed to resist earthquake loads. Despite encouraging the concentration
8 of reinforcement in the boundary elements, ACI currently has no requirement for minimum vertical
9 reinforcement to be placed at the wall ends. Furthermore, the 0.25% distributed reinforcement limit
10 is intended to control inclined shear cracks and can be reduced to the minimum shrinkage
11 requirements for all walls of 0.15% when the shear force is below a specified limit. NZS-A2 is
12 similar to ACI, in that only a distributed minimum reinforcement limit is stated. However, the
13 minimum vertical reinforcement ratio of $\sqrt{f'_c}/4f_y$ (in MPa, $0.095\sqrt{f'_c}/f_y$ in ksi) in NZS-A2 is
14 dependent on the concrete and reinforcement strengths and is required in all RC walls irrespective
15 of the design ductility. Other standards including Eurocode, GB, CSA and NZS-A3 require
16 additional vertical reinforcement to be concentrated at the ends of the wall. In Eurocode, a vertical
17 reinforcement ratio in the wall ends of at least 0.5%, and a distributed vertical reinforcement ratio of
18 at least 0.2% through the web region, are required for walls designed in accordance with medium
19 and high ductility class. The CSA and GB standards state that in addition to a minimum distributed
20 vertical reinforcement ratio of 0.25%, concentrated vertical reinforcement shall be provided at wall
21 ends for all classes of ductility. For ductile walls, CSA states that a minimum of four bars should be
22 placed in at least two layers in the boundary elements and that the minimum area of vertical
23 reinforcement in the wall boundary elements in plastic hinge regions shall be at least $0.0015b_w l_w$. In
24 the GB code the minimum vertical reinforcement content required in the ends of the wall is
25 dependent on the seismic design category. Table 1 shows an example of 1.0% for seismic design

1 level I (highest ductility class) in the GB code. Following the Canterbury Earthquakes and
2 subsequent research, the minimum vertical reinforcement requirement for ductile RC walls was
3 revised in NZS-A3. The minimum distributed vertical reinforcement in all walls was maintained at
4 $\sqrt{f'_c}/(4f_y)$ (in MPa, $0.095\sqrt{f'_c}/f_y$ in ksi) and an additional requirement was added for the vertical
5 reinforcement at the ends of ductile walls to ensure both primary and secondary cracks occurring in
6 plastic hinge region. The minimum required vertical reinforcement at the wall end zones (length =
7 $0.15l_w$) was derived to ensure that sufficient tensile reinforcement existed to develop secondary
8 cracks when considering long term concrete tensile strengths and dynamic strain-rate effects,
9 resulting in a doubling of the minimum reinforcement ratio to $\sqrt{f'_c}/2f_y$ (in MPa, $0.19\sqrt{f'_c}/f_y$ in
10 ksi)^{15, 16}. Furthermore, NZS-A3 requires that the distributed vertical reinforcement ratio in the wall
11 web should be at least 30% of the vertical reinforcement ratio in the wall ends to control inclined
12 web cracks.

13 It is interesting to note that the minimum vertical reinforcement requirement in NZS account for the
14 concrete and reinforcing steel strengths in a similar way to the limits that apply to RC beams,
15 whereas the minimum vertical reinforcement limits most other design standards are either a fixed
16 quantity or independent of material properties. The commuted minimum distributed and end zone
17 vertical reinforcement ratios for a range for concrete and reinforcement strengths in accordance
18 with NZS-A3 are illustrated in Fig. 1 alongside the fixed limits from other design standards. As
19 shown in Fig. 1-a, the distributed reinforcement ratio specified by NZS-A2 and NZS-A3 ranges
20 from 0.23% to 0.70% for concrete strengths ranging from 30 MPa (4.4 ksi) to 70 MPa (10.2 ksi)
21 and reinforcing steel yield strengths ranging from 300 MPa (43.5 ksi) to 600 MPa (87 ksi). The
22 NZS limit is similar to the 0.25% fixed limit required by ACI, CSA, and GB when using 30 MPa
23 (4.4 ksi) concrete and reinforcing steel with yield strength of 500 MPa (72.5 ksi), but exceeds other
24 design standards when the concrete strength is increased or the reinforcement strength decreased.

1 As shown in Figure 1-b, the end zone reinforcement ratio specified by NZS-A3 ranges from 0.46%
2 to 1.40% for concrete strengths ranging from 30 MPa (43.5 ksi) to 70 MPa (10.2 ksi) and
3 reinforcing steel yield strengths ranging from 300 MPa (43.5 ksi) to 600 MPa (87 ksi). The NZS-A3
4 end zone limit spans from the 0.5% fixed limit in Eurocode to beyond the 1.0% limit in GB and
5 CSA. When using a 40 MPa (4351 psi) concrete and 500 MPa (72.5 ksi) yield steel strength, the
6 minimum lumped reinforcement ratio is calculated as 0.63%, which is 25% higher than Eurocode
7 but 37% lower than GB and CSA.

8 **FINITE ELEMENT MODEL**

9 To comprehensively study the behaviour of RC walls with minimum vertical reinforcement in
10 accordance with each design standard, a finite element model was developed and verified against
11 existing test data. The model is briefly described in the following sections and a more detailed
12 description of the model development and verification has been published separately⁸.

13 **Model description**

14 Fig. 2 shows the numerical model developed for RC wall using nonlinear finite element program
15 VecTor2¹⁷. Four-node rectangular elements with a uniform thickness were used for the wall section.
16 Wall horizontal reinforcement and additional transverse reinforcement ties were distributed
17 uniformly over the wall height so they were modelled as smeared reinforcement. For walls with
18 transverse reinforcement ties in the ends of the wall, two different regions were modelled consisting
19 of unconfined concrete with smeared horizontal reinforcement, and confined concrete with smeared
20 horizontal reinforcement and transverse reinforcement ties. In order to accurately represent the
21 vertical reinforcement location and strains, two-node truss elements with uniform cross sectional
22 area were used to represent each layer of vertical reinforcement. The axial compression due to self-
23 weight and gravity load actions was held constant during the analyses, whereas the lateral load
24 applied at the top of the wall was monotonically increased in a displacement-controlled mode.

1 The constitutive law for concrete in compression used the Hognestad parabola model for the
2 ascending curve, with a Park-Kent¹⁸ descending branch. The concrete compression softening was
3 considered by the model proposed by Vecchio¹⁹. The model proposed by Lee et al. was chosen to
4 represent the concrete tension stiffening^{17, 20}, with the peak concrete tensile strength for all the test
5 wall models determined in accordance with *fib* model code²¹ recommendation that is considered to
6 represent state-of-the-art knowledge on concrete properties and a more realistic expression for the
7 average tensile strength compared to other design standards. The cyclic stress-strain parameters for
8 the concrete model was based on that proposed by Palermo and Vecchio²². The reinforcing steel
9 stress-strain behaviour implemented in the wall model used the nonlinear hysteric model proposed
10 by Seckin²³ with a back-bone that included an initial linear-elastic response, a yield plateau, and a
11 non-linear strain hardening phase until rupture (options HP4, P=4 in VecTor2)¹⁷. In addition, the
12 model employed regularization of the reinforcing steel material response using the steel yield
13 energy and mesh-dependent lengths^{24, 25}.

14 A foundation was also modelled at the base of the wall. The bottom edge of the foundation was
15 fixed in the model. The vertical reinforcement in the wall extended to the bottom of the foundation
16 to simulate the anchorage of the reinforcement. The mesh size in the wall and foundation was
17 chosen to be 75×75 mm (2.95×2.95 in) as this size was calibrated to best represent the overall
18 and local wall behaviour after conducting a mesh sensitivity study⁸. The mesh size used for the
19 concrete cover was governed by the wall dimensions.

20 **Experimental validation**

21 To examine the suitability of the VecTor2 model for lightly reinforced concrete walls, the model
22 was verified against the experimental test data. Full verification of the numerical model has been
23 published by Lu and Henry⁸, and the results of the model for test wall C1 reported by Lu et al.³ are
24 shown here as an example. Wall C1 had a length of 1.4 m (55.1 in), height of 2.8 m (100.2 in) and a

1 thickness of 150 mm (5.9 in), as shown by the drawing in Fig. 3. The total vertical reinforcement
2 ratio was 0.53%, resulting in 14 D10 (deformed bar, diameter = 10 mm or 0.39 in) bars placed in
3 two layers at 225 mm (8.86 in) centers over the wall length. R6 (round bar, diameter = 6 mm or
4 0.24 in) stirrups were used for horizontal reinforcement distributed evenly at 150 mm (5.9 in)
5 centers over the wall height. Grade 300E New Zealand reinforcing steel was used in the test walls
6 with a measured yield strength of 300 MPa (43.5 ksi), an ultimate strength of 409 MPa (59.3 ksi),
7 and an ultimate strain of 18.1% at fracture based on a gauge length of 100 mm (3.9 in). Therefore,
8 the ultimate strain indicating reinforcement fracture in the model can be estimated as 12.1% (ϵ_{reg})
9 based on a gauge length of 150 mm (5.9 in) using the regularization technique described earlier. The
10 average concrete compressive strength at the time of test was 38.5 MPa (5584 psi) and the
11 corresponding tensile strength was calculated as 3.42 MPa (496 psi) in accordance with the *fib*
12 model code.

13 A comparison of model and test results for test wall C1 are shown in Fig. 4. Overall, the model
14 captured the measured and observed response of the wall with reasonable accuracy. The lateral
15 strength and stiffness of the wall were well represented for most of the lateral drift cycles, as shown
16 in Fig. 4-a. The crack pattern during test is compared against the calculated crack pattern at 1.5%
17 lateral drift in Fig. 4-b. During the test, the behaviour of wall C1 was controlled by 2-3 main
18 primary flexural cracks with the bottom crack opening the widest and dominating the response. The
19 crack pattern predicted by the model was also controlled by a two dominant primary cracks with the
20 bottom primary crack opening significantly wider than the second primary crack. The top lateral
21 deformation was mostly attributed to the wide crack at the wall base during both in the test and the
22 model.

23 Comparisons between the measured and modelled vertical reinforcement strains over the wall
24 height at 1.5% lateral drift are shown in Fig. 4-c. The strain measured and calculated strains were

both based on the gauge length of 150 mm. When considering that the experimental results were often affected by various experimental and specimen uncertainties, such as the crack initiation, the model results are considered to be in reasonable agreement with the measured strains. As shown in Fig. 4-c, during both test and model the reinforcement strains were peaked at locations of wide flexural cracks instead of yielding consistently over the entire plastic hinge region. Furthermore, the drift capacity when the vertical reinforcement fractured could be reasonably estimated by the model based on reinforcement tensile strains. The calculated reinforcement strains immediately prior to reinforcement fracture during the test was 15.2%, slightly higher than 12.1% that was defined as the ultimate strain for reinforcement using the previously stated regularization technique.

WALL ANALYSES

The VecTor2 model developed was used to investigate the lateral load response of walls designed in accordance with the minimum vertical reinforcement requirements from each of the design standards discussed earlier. Key parameters that were selected for the modelled walls are summarised in Table 2. From the research by Lu and Henry⁸, wall length or scale was shown to have a significant effect on the behaviour of lightly reinforced concrete walls and the half scale walls tested by Lu et al.³ may have overestimated the drift capacity. In order to include the size effect, the section of the modelled walls herein was selected as twice that of the walls tested by Lu et al.³. All the modelled walls had identical dimensions with a length of 2.8 m (110 in), a height of 8.4 m (331 in) and thickness of 300 mm (11.8 in), resulting in a shear span ratio of 3, representative of flexure-dominant behaviour. The axial load ratio for all the modelled walls was chosen to be 3.5% which was consistent with the test wall C1 and considered representative for lightly reinforced walls.

A total of 25 wall models were investigated using six vertical reinforcement layouts that were designed in accordance with the minimum requirements from each of the standards discussed

1 earlier. The resulting reinforcement details for each of the walls modelled are listed in Table 2 and
2 the drawings of the sections are shown in Fig. 5. Fictitious bar diameters were used to achieve the
3 exact minimum required vertical reinforcement ratio while keeping the spacing consistent between
4 walls. The first set of walls was designed with a fixed distributed minimum (FM) vertical
5 reinforcement ratio of 0.25%, resulting in two layers of 13 D10.2 bars (diameter = 10.2 mm or 0.4
6 in) placed at 225 mm (8.86 in) centers over the wall length. Wall FM-e included a reduced vertical
7 reinforcement ratio of 0.15%, corresponding to 13 D7.9 (diameter = 7.9 mm or 0.31 in) bars placed
8 at 225 mm (8.86 in), to investigate walls with a reinforcement ratio corresponding to typical
9 shrinkage and temperature requirements. The FM walls represent an unlikely, but still possible and
10 compliant, design as per many design standards with no limit on the vertical reinforcement
11 concentrated at the wall ends (such as ACI). The NZS-A2 wall section had a reinforcement layout
12 and spacing consistent with the FM walls, but the reinforcement diameters were varied between
13 D11.5 (diameter = 11.5 mm or 0.45 in) to D12.6 (diameter = 12.6 mm or 0.5 in) based on the
14 concrete and reinforcement properties, as per the NZS-A2 minimum vertical reinforcement formula
15 shown previously in Table 1. The wall sections with additional vertical reinforcement concentrated
16 at the ends designed in accordance with Eurocode, CSA, GB, and NZS-A3, the reinforcement
17 layout and spacing was also kept consistent. Four reinforcing bars were placed in two layers at ends
18 of the wall and two layers of nine reinforcing bars at 225 mm (8.86 in) centers were distributed
19 along the wall web region. The end zone length was taken as $0.15l_w$, (equal to 420 mm or 16.5 in) in
20 accordance with Eurocode, CSA, GB and NZS-A3 requirements. The reinforcement diameters were
21 calculated based on the minimum vertical reinforcement ratio as required by each standard and
22 summarised previously in Table 1. For Euro walls, four D14.1 (diameter = 14.1 mm or 0.56 in) bars
23 were placed at wall end and two layers of nine D9.1 (diameter = 9.1 mm or 0.36 in) bars were
24 distributed along the wall web. The reinforcement ratios for CSA and GB walls were calculated to
25 be the same. Four D20 (diameter = 20 mm or 0.79 in) bars were placed at wall end and two layers

1 of nine D10.2 (diameter = 10.2 mm or 0.4 in) bars were distributed along the wall web. For NZS-
2 A3-1 walls, the reinforcement diameters were varied based on concrete and reinforcement
3 properties as shown in Table 2, resulting in a range of 4 D16 (diameter = 16 mm or 0.63 in) to
4 4 D17.7 (diameter = 17.7 mm or 0.7 in) in the ends of the wall and between D11.5 (diameter = 11.5
5 mm or 0.45 in) and D12.7 (diameter = 12.6 mm or 0.5 in) distributed along the wall web region. To
6 investigate the effect of reinforcement distribution in the wall central web region, a second NZS-A3
7 wall section was designed using larger diameter reinforcing bars with the spacing increased to
8 450 mm (17.7 in). As a result of this variation, wall section NZS-A3-2 had between D15.4
9 (diameter = 15.4 mm or 0.6 in) and D17.1 (diameter = 17.1 mm or 0.67 in) vertical reinforcement
10 distributed along the web region.

11 In order to investigate the effect of material properties on the behaviour of the walls designed in
12 accordance with minimum vertical reinforcement requirements, each of the wall sections were
13 analysed with three different concrete strengths, equal to 38.5 MPa (5583 psi), 50 MPa (7252 psi)
14 and 60 MPa (8702 psi), and three different types of reinforcement, consisting of G500E New
15 Zealand reinforcement and Class B and Class C European reinforcement. As shown in Table 2, the
16 baseline wall for each group had a concrete strength of 38.5 MPa (5583 psi) and a vertical
17 reinforcement grade of G500E. The concrete tensile strength used in the models for walls with
18 concrete strength 38.5 MPa (5583 psi), 50 MPa (7252 psi) and 60 MPa (8702 psi) were calculated
19 as 3.42 MPa (496 psi), 4.07 MPa (590 psi), and 4.60 MPa (667 psi), respectively, in accordance
20 with the *fib* model code provisions²¹. The reinforcement properties were all derived from average
21 test results rather than being lower characteristic or minimum values. The properties of the G500E
22 reinforcement were obtained from average test results available from Pacific Steel in New Zealand,
23 and consisted of a yield strength (f_y) of 544 MPa (78.9 ksi), ultimate strength (f_u) of 653 MPa
24 (94.7 ksi), and ultimate strain based on a gauge length of 100 mm (3.9 in) (ϵ_u) of 12.4%. The
25 properties of the Class B and Class C were obtained from the reinforcing steel used for test walls

WSH 2 and WSH3 reported by Dazio et al.²⁶. The Class B ($f_y=484.9$ MPa (70.3 ksi), $f_u=534.5$ MPa (77.5 ksi), $\epsilon_u=5.76\%$) and Class C ($f_y=601$ MPa (87.2 ksi), $f_u=725.5$ MPa (105.2 ksi), $\epsilon_u=7.7$) reinforcement had a lower ductility and ultimate strain capacity than the G500E reinforcement.

The horizontal shear reinforcement was designed following procedures in each of the design standards. For all wall designs, only minimum horizontal reinforcement was required. To keep the designs consistent, a horizontal reinforcement ratio of 0.25% was used for all the modelled walls, resulting in D8 (diameter = 8 mm or 0.31 in) stirrups distributed evenly at 130 mm (5.1 in) centers over the wall height, as shown in Fig. 5. Transverse reinforcement ties were also designed in accordance with procedures in each of the design standards. The vertical reinforcement ratio was too low to require transverse reinforcement ties for the walls designed in accordance with ACI and NZS-A2. For the Euro, CSA, GB, NZS-A3 walls, transverse reinforcement ties were placed within the compression region over the lower 2.8 m (110 in) of the wall, as shown in Fig. 5. D6 (diameter = 6 mm or 0.24 in) at 60 mm (2.4 in) stirrups were used for Euro and NZS-A3 walls and D8 (diameter = 8 mm or 0.31 in) at 100 mm (3.9 in) were used for CSA and GB walls.

A displacement-control monotonic loading protocol was applied to each wall model until the reinforcement fractured, as defined by regularised ultimate strain based on a gauge length of 150 mm (5.9 in). According to the regularization technique, the ultimate strains based on a gauge length of 150 mm (5.9 in) for G500E, Class B and Class C reinforcement were 8.5%, 5.1% and 3.9%, respectively.

Fixed minimum walls

The calculated crack pattern, vertical reinforcement strains and the moment-displacement response for each of the FM walls are shown in Fig. 6 and Fig. 7. The behaviour of the baseline wall FM-a that had a concrete strength of 38.5 MPa (5583 psi) and G500E reinforcement was dominated by two primary cracks with no significant secondary cracking over the wall height. The vertical

1 reinforcement strains peaked sharply at the locations of wide flexural cracks rather than yielding
2 consistently over the entire plastic hinge region. The concentrated inelastic deformation resulted in
3 premature reinforcement fracture of the vertical reinforcement at a low drift capacity of 0.57%.
4 Wall FM-b and wall FM-c were identical to FM-a except with a higher concrete strength of 50 MPa
5 (7252 psi) and 60 MPa (8702 psi), respectively. The behaviour of wall FM-b was similar with that
6 of wall FM-a, but wall FM-c exhibited only a single crack at the wall base and as a result the drift
7 capacity reduced to 0.46%. The crack pattern and reinforcement distribution of wall FM-d with a
8 concrete strength of 38.5 MPa (5583 psi) and Class B reinforcement was similar with that of wall
9 FM-c and controlled by a single crack at wall base. Despite having the same concrete strength as
10 FM-a, the reduced strain hardening ratio of the Class B reinforcement prevented the second flexural
11 cracks forming. The drift capacity of FM-d was only 0.25%, significantly lower than that of wall
12 FM-a due to the reduced ductility of the Class B reinforcement. Wall FM-e with a vertical
13 reinforcement ratio of only 0.15% was also dominated by a large single crack at wall base and the
14 drift capacity was 0.54%, lower than that of wall FM-a.

15 RC walls with a fixed distributed minimum vertical reinforcement ratio of 0.25% or lower cannot
16 be expected to form a large amount of distributed cracks in the plastic hinge and as a result exhibit
17 only limited deformation capacity. Additionally, because the reinforcement content and resulting
18 tensile force remains constant for all material strengths, the wall behaviour becomes worse as the
19 concrete strength is increased with reduced cracking and drift capacity.

20 **NZS-A2 walls**

21 The calculated crack pattern, vertical reinforcement strains and the moment-displacement response
22 for each of the NZS-A2 walls are shown in Fig. 8 and Fig. 9. The behaviour of the baseline wall
23 NZS-A2-a that had a concrete strength of 38.5 MPa (5583 psi) and G500E was dominated by two
24 primary cracks with no significant secondary cracking over the plastic hinge region length despite

the wall having a 30% higher reinforcement ratio than that of the FM walls. The vertical reinforcement strains also peaked sharply at locations of wide flexural cracks. However, unlike the FM walls, the crack patterns and reinforcement strain distributions was similar for all the four NZS-A2. The cracking behaviour of the walls with minimum vertical reinforcement was not significantly influenced by material properties because both the concrete and reinforcement strengths are included in the calculation of the required minimum vertical reinforcement. As a result, a drift capacity of 0.6% was calculated for walls NZS-A2-a, b and c. For NZS-A2-d, a drift capacity of 0.32% was calculated, which was lower than the other three walls due to the reduced ultimate strain capacity of the Class B reinforcement.

Based on the results of the NZS-A2 walls, it was concluded that the minimum vertical reinforcement requirements in NZS-A2 are also not sufficient to generate a large number of well distributed secondary cracks in the plastic hinge. However, it is clear that the material properties should be included in the design standard requirements for minimum vertical reinforcement to ensure consistent crack distribution and wall behaviour as material strengths increase or decrease.

Euro walls

The calculated crack pattern, vertical reinforcement strains and the moment-displacement response for each of the Euro walls are shown in Fig. 10 and Fig. 11. The performance of the baseline wall Euro-a that had a concrete strength of 38.5 MPa (5583 psi) and G500E reinforcement was better than that of wall NZS-A2-a. Despite the increased reinforcement ratio of 0.5% in the ends of the wall, the total vertical reinforcement ratio of wall Euro-a was 0.29%, slightly lower than 0.32% of wall NZS-A2-a. Despite similar total reinforcement contents, the concentrated reinforcement ratio of 0.5% in the ends of the wall helped to generate greater secondary cracking in the plastic hinge region. The increased secondary cracking in wall Euro-a improved the spread of inelastic strains in the vertical reinforcement, resulting in a more ductile response than that calculated for wall NZS-

1 A2-a. As a result, the drift capacity of wall Euro-a was 0.85%, larger than the 0.6% calculated for
2 wall NZS-A2-a. Wall Euro-b and Euro-c were identical to wall Euro-a except that they had higher
3 concrete strengths of 50 MPa (7252 psi) and 60 MPa (8702 psi), respectively. As the concrete
4 strength increased, secondary cracking greatly reduced and the reinforcement strains were
5 concentrated in a reduced plastic hinge region. The drift capacity of walls Euro-a, Euro-b and Euro-
6 c was calculated to be 0.85%, 0.71% and 0.64%, respectively. Due to the lower yield strength of
7 Class B reinforcement, the behaviour of wall Euro-d was controlled by only two main cracks and
8 the reinforcement strains concentrated at these wide cracks. Combined with the reduced ultimate
9 strain capacity of the Class B reinforcement, the drift capacity of wall Euro-d was only 0.32%.
10 However, the higher reinforcing steel yield strength of the Class C reinforcement resulted in a
11 significant greater number of secondary cracks in wall Euro-e and reinforcement strains that were
12 more evenly distributed in the plastic hinge region. Even though the ultimate strain capacity of the
13 Class C reinforcement was less than that of the G500E reinforcement, the drift capacity of wall
14 Euro-e was 0.96%, larger than that achieved for walls NZS-A2-a and Euro-a.

15 The results of the Euro walls confirmed that concentrating a greater portion of vertical
16 reinforcement at the wall ends can improve the lateral load response of lightly reinforced concrete
17 walls. However, despite this improvement, a fixed lumped reinforcement ratio of 0.5% in the wall
18 ends was still insufficient to generate a large number of secondary cracks in plastic hinge region and
19 to ensure large deformation capacity required for ductile walls, especially when higher concrete
20 strengths are used.

21 **GB/CSA walls**

22 The GB and CSA walls were modelled together as the reinforcement ratios required at the ends and
23 web region of these walls were identical for both of these standards. The calculated crack pattern,
24 vertical reinforcement strains and the moment-displacement response for each of the GB/CSA walls

1 are shown in Fig. 12 and Fig. 13. As both the wall end zone and distributed reinforcement ratio of
2 the GB/CSA walls were higher when compared to Euro walls, the performance of wall GB/CSA-a
3 was significantly better than that of wall Euro-a. A greater number of secondary cracks occurred at
4 the ends of the wall and cracks were also distributed over a greater height of the wall. In addition,
5 the reinforcement strains were more evenly distributed over the plastic hinge region, resulting in a
6 more ductile behaviour. The drift capacity of wall GB/CSA-a when web reinforcement fractured
7 was estimated as 2.02%, significantly larger than the 0.85% achieved for the Euro-a wall and 3.5
8 times larger than the 0.57% achieved for the FM-a wall. Although significant secondary cracking
9 occurred in the ends of the wall for wall GB/CSA-a, it should be noted that the wall web region was
10 still vulnerable to the formation of wide cracks at larger drifts. Due to the large difference in the
11 reinforcement ratio between the ends of the wall and wall web ($\rho_{l_end}/\rho_{l_web} = 4$), the cracks in
12 the end of the wall did not propagate continuously and tended to join up to form wide cracks in the
13 web region, as shown in Fig. 12-a. The wide web cracks also caused shear sliding to initiate at large
14 drifts. To clearly illustrate this point, the reinforcement strains in the web are also shown in Fig. 12.
15 The reinforcement strains greatly peaked at the location of a wide crack in the web, leading to
16 premature web reinforcement fracture. This behaviour has also been recognized in both RC wall
17 tests and models conducted by other researchers^{7, 27, 28}. For wall GB/CSA-a, the drift capacity at
18 web reinforcement fracture was 2.02% which was lower than the 2.45% drift required to cause
19 fracture of reinforcement in the end of the wall. In engineering practice, it is common to lump most
20 of the vertical reinforcement at walls ends or boundary elements and use only minimum vertical
21 reinforcement distributed in the wall web. However, this practice is in question as a higher
22 proportion of web reinforcement is required to prevent wide inclined cracks and premature web
23 reinforcing bar fracture.

24 Wall GB/CSA-b and GB/CSA-c were identical to wall GB/CSA-a except using a higher concrete
25 strength of 50 MPa (7252 psi) and 60 MPa (8702 psi), respectively. As the concrete strength

increased, the wall web region became more vulnerable to wide cracks and severe localization of reinforcement strain, resulting in a lower drift capacity. The drift capacity of walls GB/CSA-a, GB/CSA-b and GB/CSA-c at web reinforcing bar fracture were 2.02%, 1.71% and 1.63%, respectively. For wall GB/CSA-d, due to the low ultimate strain of Class B reinforcement, the vertical reinforcement at the end of the wall fractured at a lower drift of 0.57%.

The model results of the GB/CSA walls highlighted the improved lateral load response of RC walls with 1.0% vertical reinforcement at the wall ends in comparison to that of the Euro walls with only 0.5%. However, lumping too much reinforcement in the ends of the wall can leave the web region vulnerable to the formation of wide cracks, increasing the risk of premature of web reinforcement fracture and large shear deformations.

NZS-A3 walls

The calculated crack pattern, vertical reinforcement strains and the moment-displacement response for each of the NZS-A3-1 walls are shown in Fig. 14 and Fig. 15. For wall NZS-A3-1a that had a concrete strength of 38.5 MPa (5583 psi) and G500E reinforcement, the end and web region reinforcement ratios were 0.64% and 0.32%, respectively. The total reinforcement ratio was 0.41% which was slightly lower than the 0.47% for the GB/CSA-a wall. Despite the lower total reinforcement ratio, the performance of the NZS-A3-1a wall was significantly better than that of the GB/CSA walls. A significant number of secondary cracks formed along the wall height both in the end of the wall and in the web region. The web reinforcement was sufficient for secondary cracks at the end of the wall to propagate through the web region continuously without combining into several wide cracks. Both the lumped and web reinforcement strains were distributed more evenly over the plastic hinge region. The wall failure was controlled by vertical reinforcement fracture at the wall end with no premature web reinforcement fracture occurring, resulting in a more ductile response than that of wall GB/CSA-a. The drift capacity when reinforcement fractured for wall

1 NZS-A3-1a walls was around 2.10% which was slightly larger than the 2.02% achieved for wall
2 GB/CSA-a and 3.7 times larger than the 0.57% achieved for wall FM-a. Wall NZS-A3-1b and 1c
3 were comparable to wall NZS-A3-1a except using higher concrete strengths of 50 MPa (7252 psi)
4 and 60 MPa (8702 psi), respectively. The vertical reinforcement ratio in NZS-A3-1 walls was
5 calculated to account for the material properties, and as a result the crack pattern, reinforcement
6 strains and drift capacity of wall NZS-A3-1b and 1c were both similar with that of wall NZS-A3-1a.
7 The drift capacities for wall NZS-A3-1a, b and c were 2.1%, 2.38% and 2.13%, respectively. The
8 crack pattern of wall NZS-A3-1d with Class B reinforcement was also similar with that of wall
9 NZS-A3-1a, b and c. However, as the ultimate strain capacity of the Class B is much lower than
10 G500E reinforcement, the drift capacity of NZS-A3-1d was only 0.82%, but it was still significantly
11 larger than that of the comparable GB/CSA-d wall.

12 The results of the NZS-A3-1 walls confirmed that in addition to having sufficient vertical
13 reinforcement at the ends of the wall to initiate secondary cracks, the ratio of the web to end zone
14 reinforcement needs to be limited to prevent wide web cracks and premature web reinforcement
15 fracture. As with previous models, the NZS-A3-1 walls also highlighted the importance of including
16 the material properties in the calculation of minimum vertical reinforcement.

17 To investigate the influence of the web region reinforcement further a second series of NZS-A3
18 walls were modelled with the distributed vertical reinforcement spacing increased to 450 mm
19 (17.7 in), in excess of the 300 mm (11.8 in) limit in NZS 3101:2006-A3. The calculated crack
20 pattern, vertical reinforcement strains and the moment-displacement response for each of the NZS-
21 A3-2 walls are shown in Fig. 16 and Fig. 17. The baseline wall NZS-A3-2a that had a concrete
22 strength of 38.5 MPa (5583 psi) and G500E reinforcement also exhibited a large number of
23 secondary cracks at ends of the wall. However, due to the increased spacing of the web and
24 boundary reinforcement in this wall, these secondary cracks did not all propagate into the wall web

1 region. As a result, wide cracks formed in the wall web region, similar to that observed for wall
2 GB/CSA-a. The reinforcement strains in the wall web region were inconsistent up the wall height
3 with inelastic strains concentrated at crack locations. The drift of the NZS-A3-2a wall when the
4 reinforcement in the end of the wall fractured was 1.75%, while a lower drift capacity of only 1.5%
5 was achieved due to the web reinforcement fracturing. These drifts were significantly lower than
6 the 2.10% drift capacity of wall NZS-A3-1a. Walls NZS-A3-2b and 2c were identical to wall NZS-
7 A3-2a except used a concrete strength of 50 MPa (7252 psi) and 60 MPa (8702 psi), respectively.
8 The crack pattern and reinforcement distribution of these two walls were similar with that of NZS-
9 A2-2a. The drift capacities of NZS-A3-2b and 2c were 1.40% and 1.95%, respectively when the
10 reinforcement in the end of the wall fractured, and 1.03% and 1.21% at web reinforcement
11 fractured, respectively.

12 The model results of the NZS-A3-2 walls indicated that in addition to the minimum vertical
13 reinforcement limit in both the end and web region, a reinforcement spacing limit is essential. It is
14 recommended that smaller diameter bars with a smaller spacing be used in the wall web region to
15 allow for improved crack control. It should be noted that this has been implemented in the
16 amendments to NZS 3101:2006-A3 with the maximum allowable vertical reinforcement spacing
17 reduced from 450 mm (17.7 in) to 300 mm (11.8 in).

18 **CONCLUSIONS**

19 Requirements for minimum vertical reinforcement in RC walls from different concrete and seismic
20 design standards worldwide were reviewed and compared, including ACI 318-14 (ACI), NZS
21 3101:2006 (A2) (NZS-A2), CSA-A23.3-14 (CSA), Eurocode 8 (Eurocode) and GB 50010-2010
22 (GB) and NZS 3101: 2006 (A3) (NZS-A3). The provisions for minimum vertical reinforcement
23 differ substantially with regards to reinforcement ratio limits and distribution requirements.
24 Eurocode, GB, CSA and NZS-A3 require additional vertical reinforcement to be concentrated at the

ends of the walls, while ACI and NZS-A2 only require distributed minimum reinforcement. The minimum vertical reinforcement limits in NZS-A2 and NZS-A3 are dependent on the concrete and reinforcement yield strengths, while fixed minimum ratios are defined in most other standards. As a result, the minimum distributed reinforcement ratios in NZS-A2 and NZS-A3 are significantly larger than other standards when using typical material properties.

A previously developed and validated finite element model for flexure dominant lightly reinforced concrete walls was used to comprehensively study the behaviour of walls with minimum vertical reinforcement requirement in accordance with each design standard. The main conclusions drawn from these numerical models included:

- All of the RC walls designed in accordance with minimum vertical reinforcement requirements were tension controlled with failure occurring when the reinforcement fractured. However, the inelastic behaviour, crack distribution and drift capacity varied substantially between the modelled walls that conformed to each standard.
- The RC walls modelled with a fixed distributed minimum reinforcement ratio of 0.25% or less could not generate a large amount of distributed cracks in the wall plastic hinge, resulting in premature reinforcement fracture and low drift capacities of around 0.5%.
- Without increasing the total vertical reinforcement content, concentrating a greater portion of the reinforcement in the ends of the wall can improve the cracking behaviour and ductility of lightly reinforced concrete walls. However, RC walls with lumped and distributed reinforcement ratios of 0.5% and 0.2% respectively do not exhibit sufficient ductility with vertical reinforcement fracture occurring around a modest 0.8% lateral drift for the modelled walls.
- The response of RC walls modelled with a lumped and distributed reinforcement ratio of 1.0% and 0.25% respectively were significantly better than that of walls with a lumped and

distributed reinforcement ratio of 0.5% and 0.2% respectively. However, lumping a too large portion of reinforcement at the ends of the wall can leave the web region vulnerable to large widely spaced cracks, causing premature of web reinforcement fracture and large shear deformations. The ratio between lumped reinforcement and web reinforcement should be limited to ensure good crack control in the web region. Furthermore, it is recommended that smaller diameter bars with smaller spacing be placed in the wall web region to allow the secondary cracks to propagate sufficiently.

- Concrete and reinforcing steel properties have a significant influence on the crack distribution and deformation capacity for RC walls designed in accordance with a fixed minimum vertical reinforcement limits. When the vertical reinforcement content remained fixed, the number of cracks and the drift capacity in the modelled walls were reduced when using higher strength concrete, lower yield strength reinforcement, and/or less ductile reinforcement. Minimum vertical reinforcement limits that incorporate material properties (e.g. NZS 3101:2006) allow for consistent wall lateral load behaviour that is independent of material strengths.
- The new minimum vertical reinforcement limits proposed for NZS 3101:2006 (A3) performed the best out of all the design standards. The concentrated vertical reinforcement at the ends of the wall was sufficient to ensure that well distributed cracks formed in the wall plastic hinge, resulting in the largest deformation capacity prior to reinforcement fracture out of all of the walls modelled.

ACKNOWLEDGEMENTS

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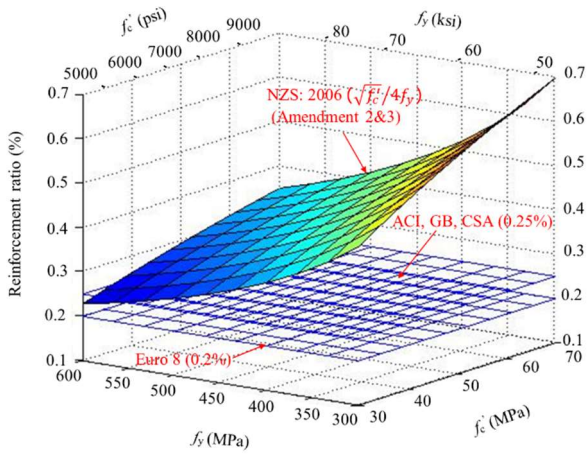
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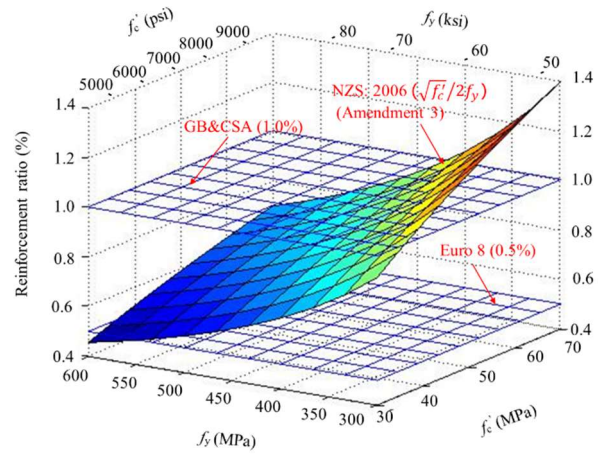
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1

FIGURES



(a) Distributed reinforcement ratio



(b) End zone reinforcement ratio

2

Fig. 1 - Comparison of minimum vertical reinforcement limits

3

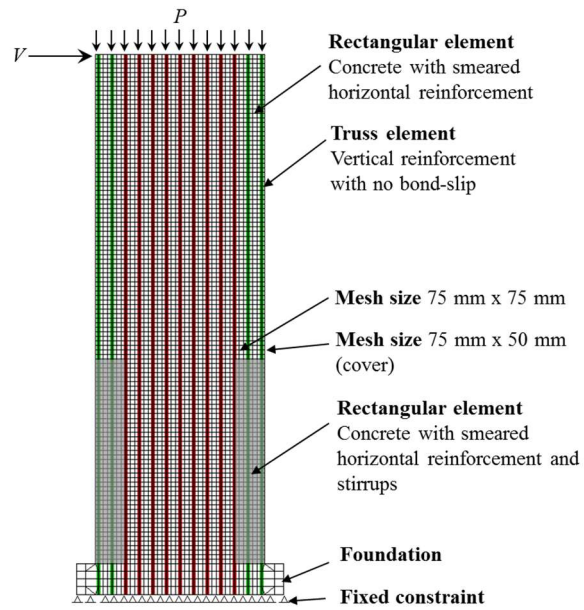
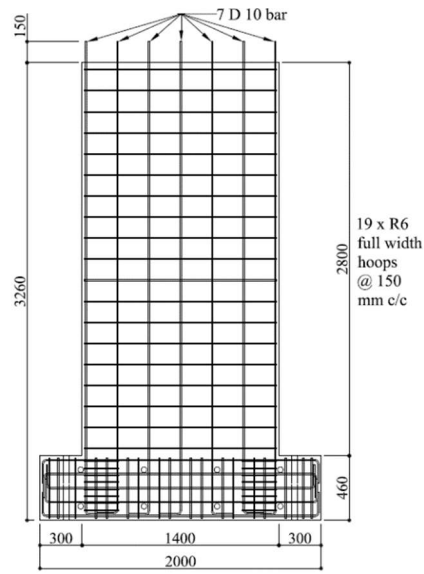
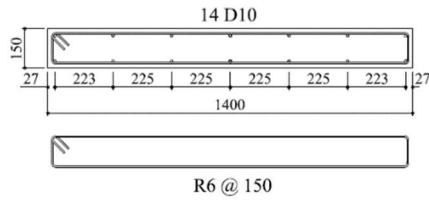


Fig. 2 - Finite element model illustration (1 mm = 0.039 in.)

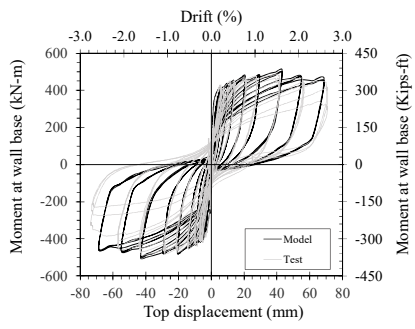


(a) Elevations

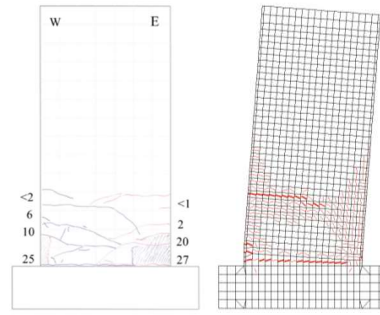


(b) Cross sections

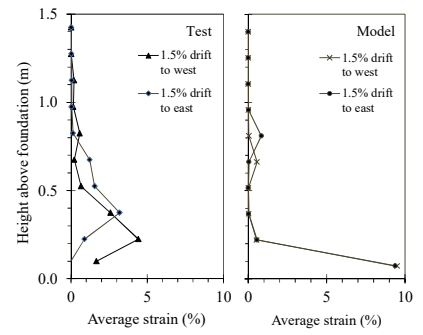
Fig. 3 - Details of test wall C1 (After Lu et al. 2015) (*D* = deformed bar; *R* = round bar; units in mm; 1 mm = 0.039 in.)



(a) Moment-displacement response

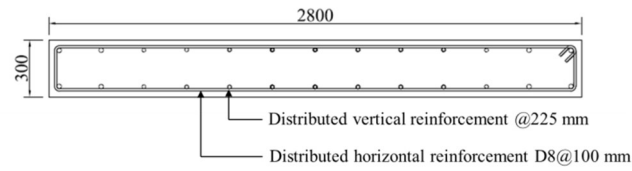


(b) Crack pattern at 1.5% lateral drift

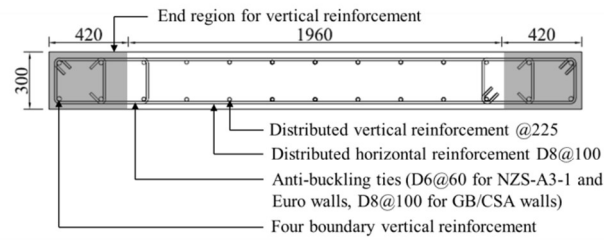


(c) Vertical reinforcement strain

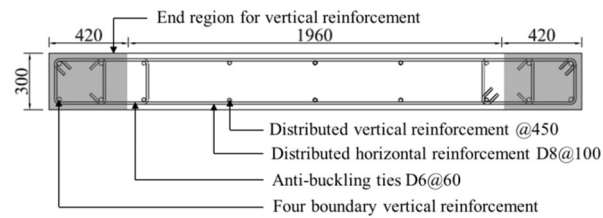
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(a) Section details for FM and NZS-A2 walls



(b) Section details for NZS-A3-1, Euro, GB/CSA walls



(c) Section details for NZS-A3-2 walls

Fig. 5 - Section details of modelled walls (D = deformed bar; units in mm; 1 mm = 0.039 in.)

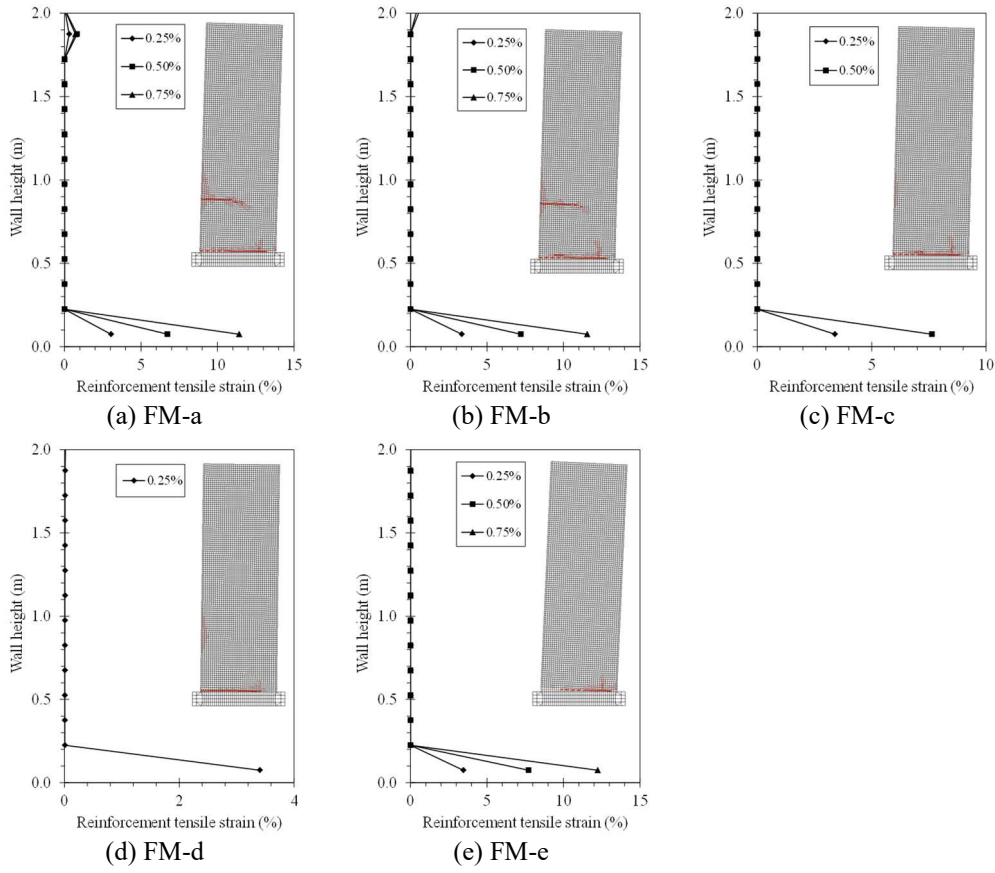


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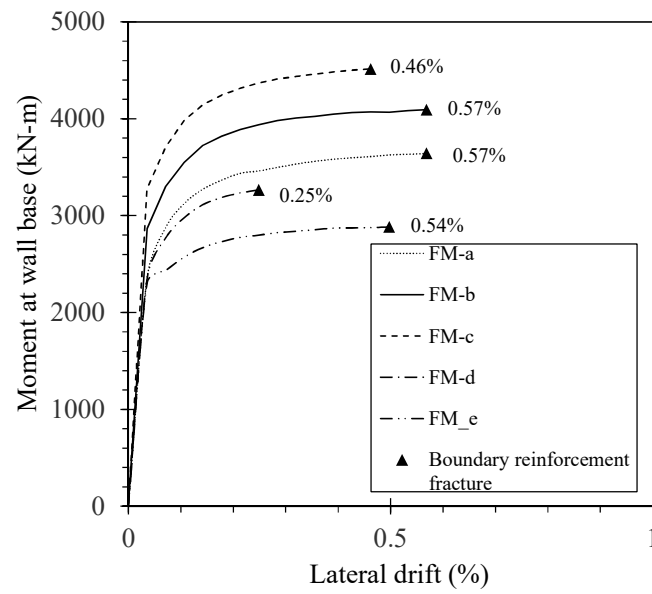


Fig. 7 - Moment-drift response for fixed minimum (FM) walls

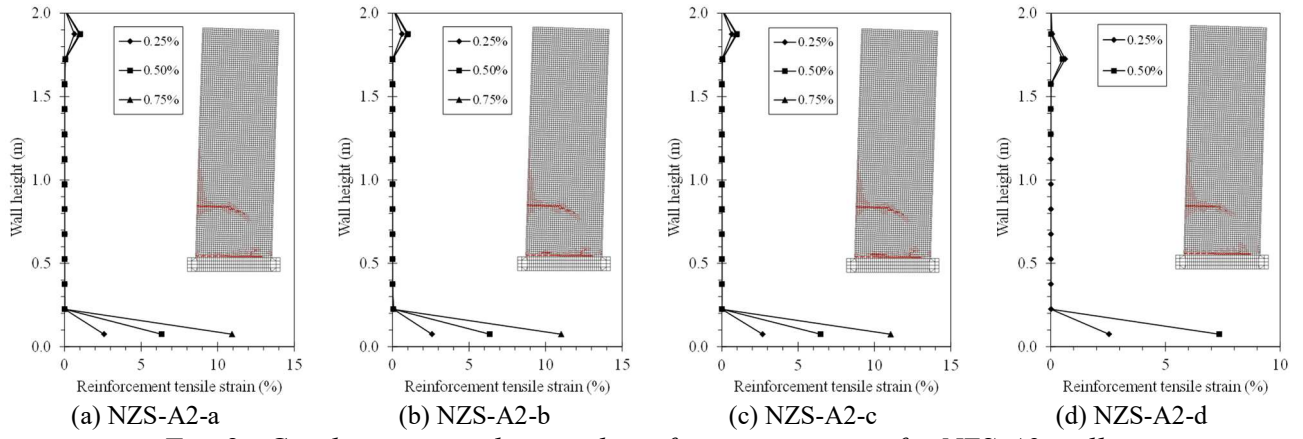


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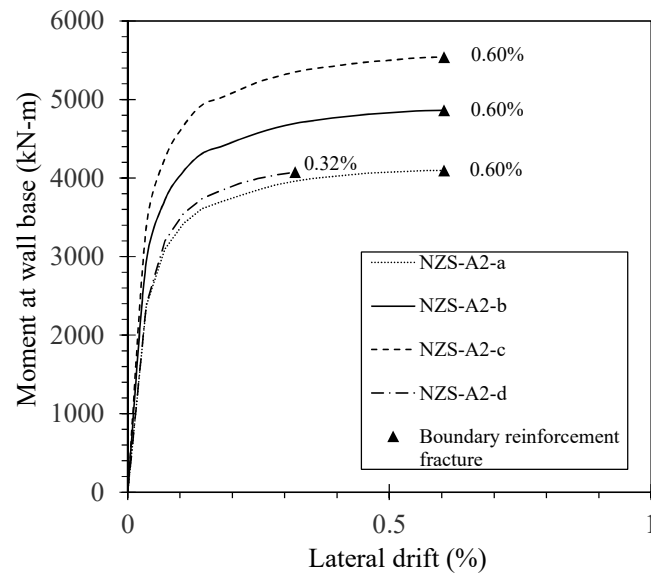


Fig. 9 - Moment-drift response for NZS-A2 walls

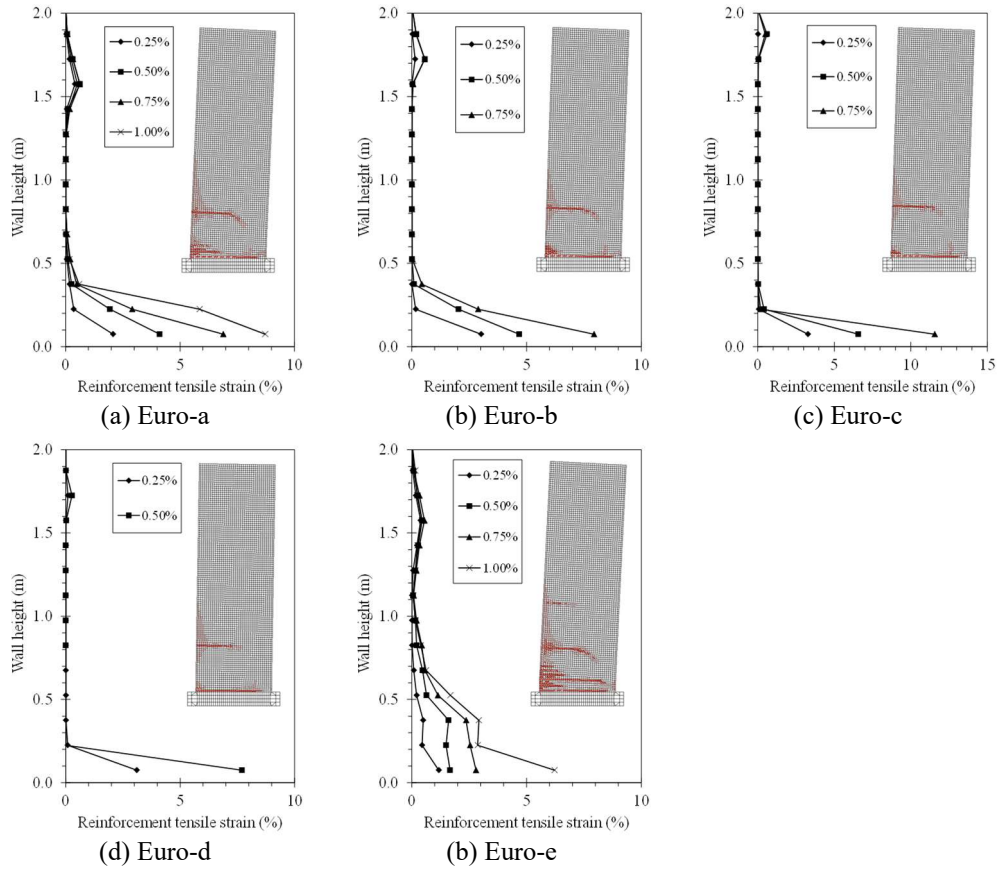


Fig. 10 - Crack pattern and vertical reinforcement strains for Euro walls

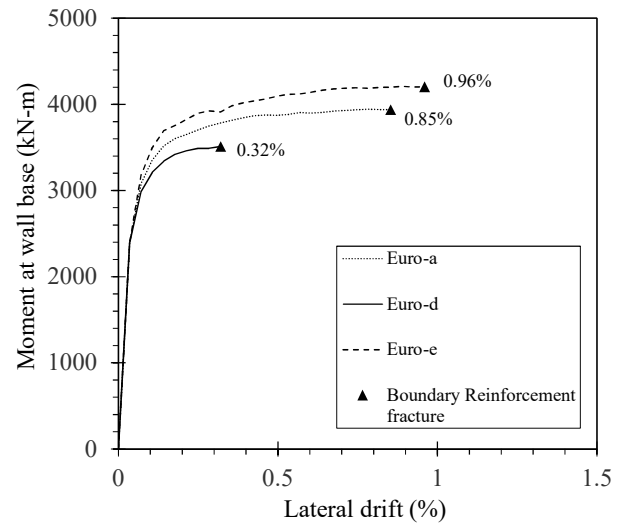
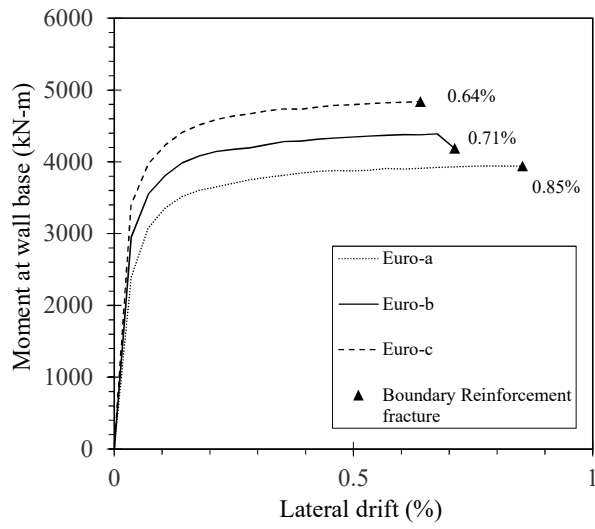
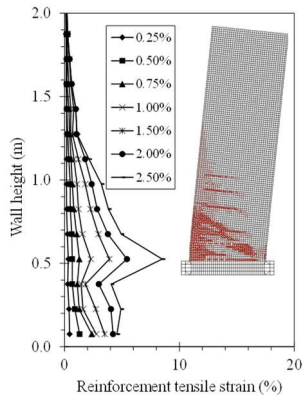
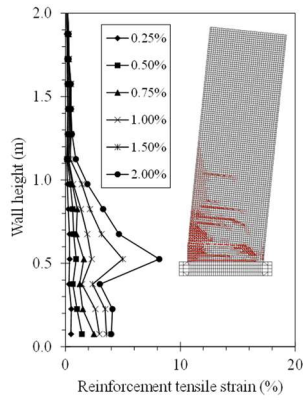


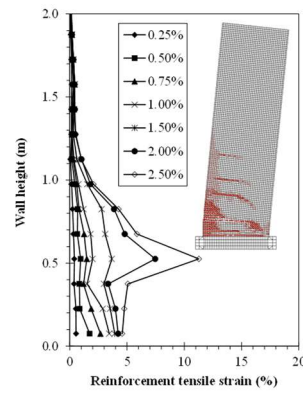
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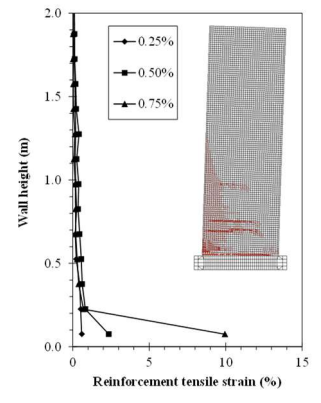
(a) GB/CSA-a at boundary



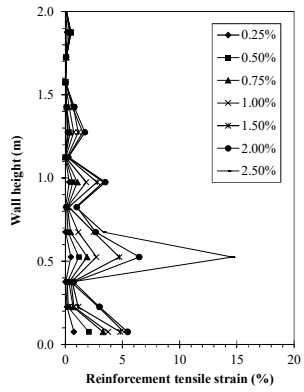
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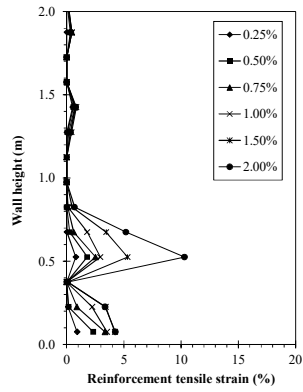
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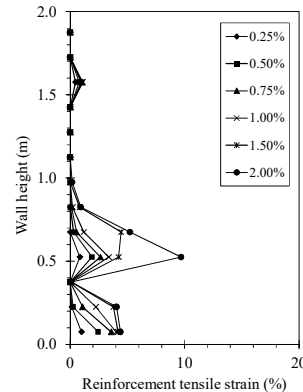
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(e) GB/CSA-a at web



(f) GB/CSA-b at web



(g) GB/CSA-c at web

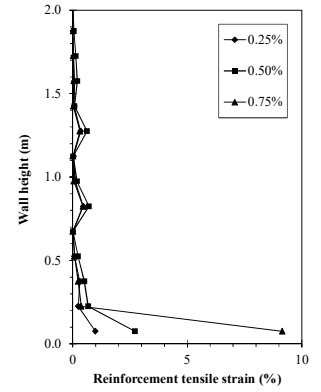


Fig. 12 - Crack pattern and vertical reinforcement strains for GB/CSA walls

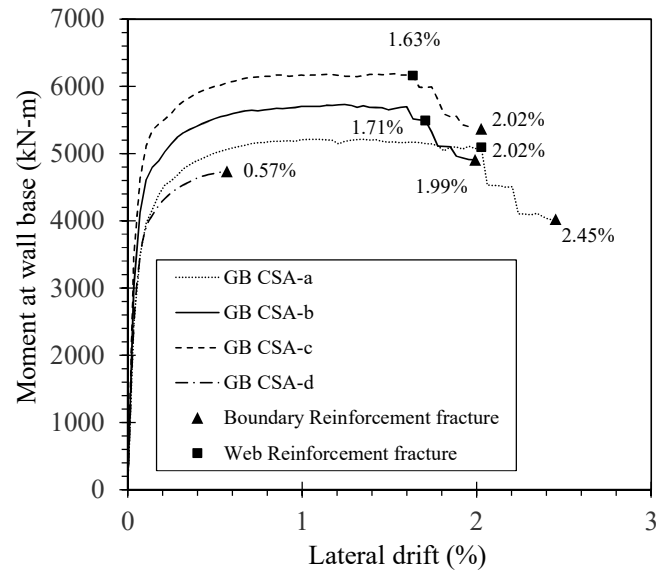


Fig. 13 - Moment-drift response for GB/CSA walls

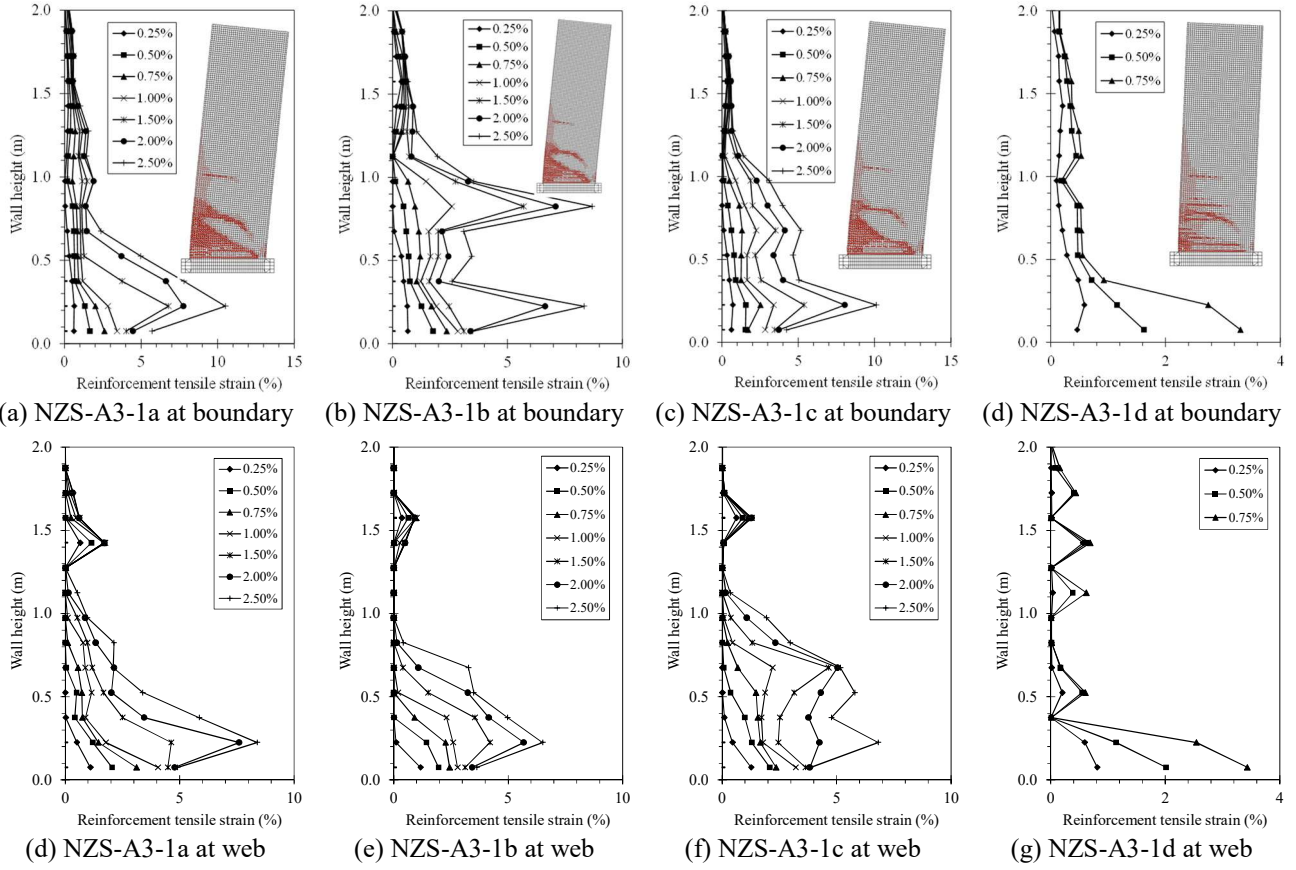


Fig. 14 - Crack pattern and vertical reinforcement strains for NZS-A3-1 walls

1

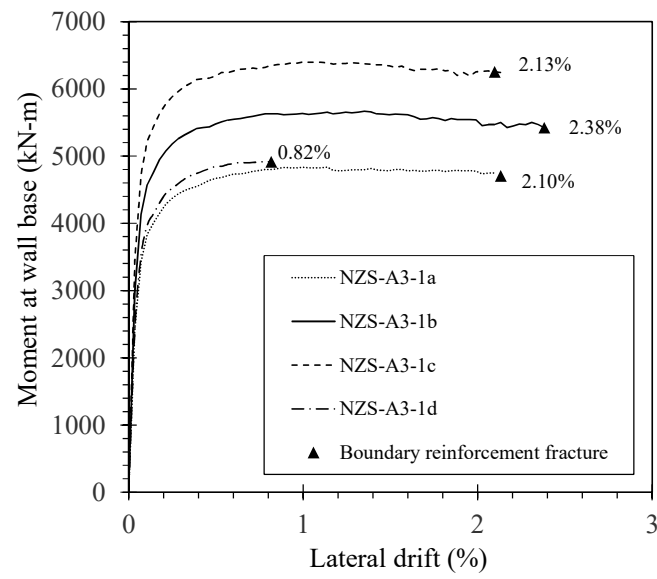


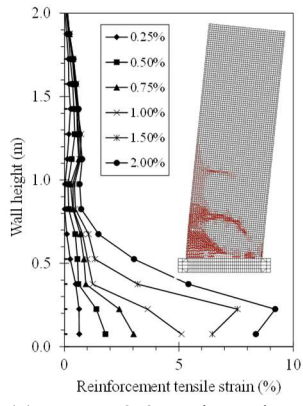
Fig. 15 - Moment-drift response for NZS-A3-1 walls

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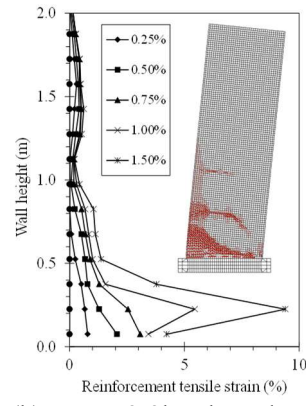
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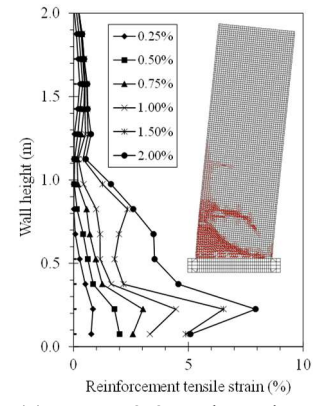
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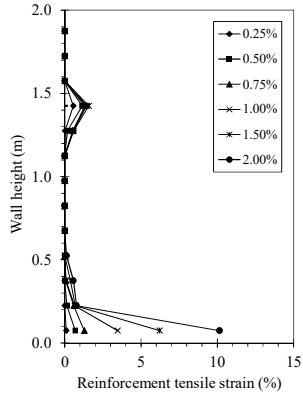
(a) NZS-A3-2a at boundary



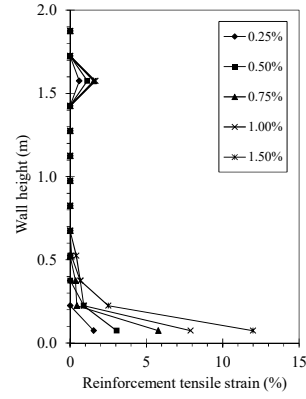
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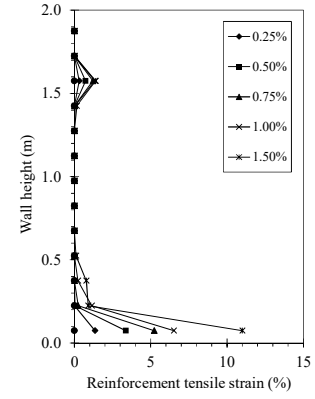
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(d) NZS-A3-2a at web



(e) NZS-A3-2b at web



(f) NZS-A3-2c at web

Fig. 16 - Crack pattern and vertical reinforcement strains for NZS-A3-2 walls

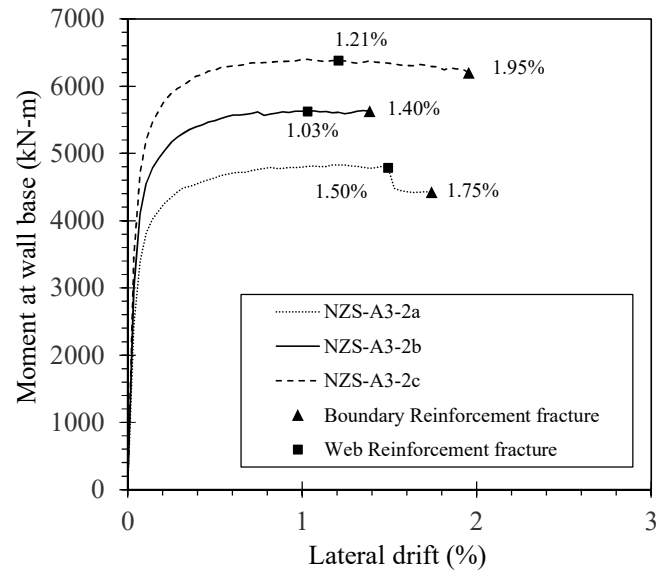


Fig. 17 - Moment-displacement response for NZS-A3-2 walls

TABLES

Table 1 - Comparison of minimum vertical reinforcement requirement for ductile RC walls

Standards	Total/distributed reinforcement ratio	End zone reinforcement ratio
ACI	$>0.25\%$	No requirement
NZS-A2	$>\sqrt{f'_c}/(4f_y)$ (MPa) $>0.095\sqrt{f'_c}/f_y$ (kpi)	No requirement
Eurocode	$>0.2\%$	$>0.5\%$
GB	$>0.25\%$	$>1.0\%$ (seismic level I)
CSA	$>0.25\%$	$>(0.15\%b_wl_w)/(b_wl_b)$
NZS-A3	$>\sqrt{f'_c}/(4f_y)$ (MPa) $>0.095\sqrt{f'_c}/f_y$ (kpi)	$>\sqrt{f'_c}/(2f_y)$ (MPa) $>0.19\sqrt{f'_c}/f_y$ (kpi)

1

Table 2 - Details of modelled walls (D = deformed bar; 1 MPa = 0.145 ksi; 1 mm = 0.039 in.)

Wall No.	Reinforcement Type	f'_c (MPa)	Vertical Reinforcement		Reinforcement ratio			Ties (mm)
			End (mm)	Web (mm)	End (%)	Web (%)	Total (%)	
FM-a	G500E	38.5						
FM-b	G500E	50						
FM-c	G500E	60		26D10.2		0.25	0.25	No
FM-d	Class B	38.5						
FM-e	G500E	38.5		26D7.9		0.15	0.15	No
NZS-A2-a	G500E	38.5		26D11.5		0.32	0.32	
NZS-A2-b	G500E	50		26D12.2		0.36	0.36	
NZS-A2-c	G500E	60		26D12.6		0.39	0.39	No
NZS-A2-d	Class B	38.5		26D12.6		0.39	0.39	
Euro-a	G500E	38.5						
Euro-b	G500E	50						
Euro-c	G500E	60	4D14.1	18D9.1	0.5	0.2	0.29	D6@60
Euro-d	Class B	38.5						
Euro-e	Class C	38.5						
GB/CSA-a	G500E	38.5						
GB/CSA-b	G500E	50						
GB/CSA-c	G500E	60	4D20	18D10.2	1.0	0.25	0.47	D8@100
GB/CSA-d	Class B	38.5						
NZS-A3-1a	G500E	38.5	4D16	18D11.5	0.64	0.32	0.41	
NZS-A3-1b	G500E	50	4D16.9	18D12.2	0.71	0.36	0.46	
NZS-A3-1c	G500E	60	4D17.7	18D12.7	0.78	0.39	0.50	D6@60
NZS-A3-1d	Class B	38.5	4D17.7	18D12.7	0.78	0.39	0.50	
NZS-A3-2a	G500E	38.5	4D16	10D15.4	0.64	0.32	0.41	
NZS-A3-2b	G500E	50	4D16.9	10D16.4	0.71	0.36	0.46	D6@60
NZS-A3-2c	G500E	60	4D17.7	10D17.1	0.78	0.39	0.50	

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