NON-RECTANGULAR RC WALLS: A REVIEW ON EXPERIMENTAL INVESTIGATIONS

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ABSTRACT

When compared to tests on reinforced concrete (RC) walls with a rectangular or barbelled cross-section, only very few tests on RC walls with open cross-section exist. Most of these walls were subjected to unidirectional or bidirectional loading along one or both of the principal axes of the wall section. It was rare that tests were done using load paths that did not follow the principal axes. This article presents a review of experimental tests of non-rectangular RC walls with open cross-section. The summary comprises tests on individual non-rectangular walls with different cross-sections such as for example L-shaped or U-shaped walls which were tested under quasi-static or dynamic loads. The tests are described with their prototype structure, test objectives, investigated parameters, loading protocols and test conclusions. Emphasis is placed on observations that are specific to non-rectangular walls, which include out-of-plane buckling of the free end of wall segments and significant deviations from the hypothesis that plane sections remain plane. The importance of bidirectional loading for non-rectangular walls is stressed because the critical loading direction for design may be different from loading in the principal directions of the section and because loading along one direction reduces the stifness of the wall for subsequent loading in the direction orthogonal to the previous one.

INTRODUCTION

Non-rectangular reinforced concrete (RC) walls are walls with cross-sections composed of several rectangular wall segments that are aligned along two different (usually orthogonal) directions. The rectangular wall segments have comparable lengths and thicknesses resulting in comparable in-plane stiffness of the wall in the two orthogonal directions. Due to this property, the walls are expected to provide stiffness and strength in both horizontal directions, which renders their behaviour under seismic loading more complex than that of individual rectangular wall segments. Testing of non-rectangular walls calls therefore for a bidirectional loading protocol.

This review comprises individual non-rectangular RC walls that have been tested under quasi-static cyclic loading or earthquake input motions at the base using shake table tests. Loading was performed along one horizontal loading direction (unidirectional loading) or along several horizontal loading directions (bidirectional). The loading directions were along the principal axes of the section or along arbitrary horizontal directions.

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The review in this paper is divided into two parts: the first part comprises tests under unidirectional loading and the second part comprises tests under bidirectional loading. Test campaigns are then grouped according to the wall cross-section shape in order to point out specific behaviour and issues depending on the wall section geometry. Conclusions are summarised in the final section, with respect to specific behaviour of non-rectangular walls.

UNIDIRECTIONAL LOADING

Flanged (I-shaped) walls

Oesterle et al. (1976) and Oesterle et al. (1979) tested two flanged walls F1 (Fig.1a) and F2 together with several other rectangular or barbell-shaped walls. The main objective of the tests was to investigate the load-deformation characteristics for a wide range of walls configurations under reverse cyclic loading and to compare them to the behaviour under monotonic loading. Test unit F2 included a confined boundary element at the intersection of web and flange while F1 did not, and for F1 no axial load was applied. Both specimens failed due to web crushing at drifts of ~2.0% (F1) and 2.8% (F2). Oesterle et al. (1979) found that the stiff boundary elements behaved as large dowels decreasing the shear deformations of the wall and the shear slip across construction joints.

NUPEC (Nuclear Power Engineering Corporation of Japan) conducted two large-scale shake table tests on two flanged shear walls (NUPEC, 1996). The two walls (U-1 and U-2) were identical (Fig.1b) and they were subjected to the same input motions in order to assess the reproducibility of the test. Both walls failed due to shear sliding failure at the base of the web. The authors confirmed the reproducibility of the test after obtaining similar maximum response values for inertial force and top displacement for both walls. Test results formed the basis of an international prediction competition in order to assess the reliability of different modelling approaches in predicting the test results. Due to large variations in predictions of load and displacement capacities of the walls, Palermo and Vecchio (2002) tested another two flanged walls very similar to the NUPEC walls under quasi-static cyclic loading at the University of Toronto. The objective of the test was to increase the available data for walls with complex geometries and to aid the development of analytical models for shear dominated walls.

The two tests at University of Toronto, DP1 and DP2 differed with regard to the applied axial load ratio: the axial load ratio of DP1 was 0.054 while DP2 was tested without axial load. DP1 failed due to web crushing along six vertical planes (due to the influence of the flanges) while DP2 failed due to crushing of the concrete and sliding shear in the web at the top of the wall where the concrete was weaker.

T-shaped walls

Several T-shaped walls have been tested under uniaxial quasi-static loading along the direction of their web. Goodsir (1985) tested a T-shaped wall at 1:3 scale (Fig.2a) along with three other rectangular walls. A floor slab stub was provided above the first floor of the wall with lateral sway prevented at that floor slab level. The objective of the test was to investigate the inelastic out-of-plane instability of the web toe. The axial load ratio was varied from 0.02 when the flange was in compression to 0.12 when the web toe was in compression. This was done to maximize inelastic tensile and compressive strains at the web toe. Failure of the wall initiated in the unconfined concrete adjacent to the web toe and later extended to the confined concrete of the web toe. The crushing was the results of high ductility demands and out-of-plane displacements of the web toe. Goodsir (1985) concluded that for walls with large design compression depths, the confinement reinforcement should be extended further inside the section than just the “the outer half of the compression block”.

Thomson and Wallace (1995) tested two T-shaped walls at 1:4 scale (Fig.2c-d) along with two rectangular walls as part of the wall system of a six storey building in a high seismic region. The difference between the two T-shaped test units laid in the confinement reinforcement of the web toe (increased length and decreased spacing of stirrups for TW2) and in the design procedure of the wall (TW1: flange and web designed as two independent walls; TW2: flange and web considered as coupled). TW1 failed due to buckling of all the longitudinal bars in the web toe at a drift of 1.25% while TW2 failed at 2.5% drift due to out-of-plane buckling of the web toe after severe concrete spalling. Thomson and Wallace (1995) concluded that the confinement detailing of the web toe was more critical than that of the interface web-flange and stressed the importance of accurate estimates of the effective flange width when determining wall capacity in the web direction.

Choi et al. (2004) presents results from experimental tests of two T-shaped walls with confinement details complying with code provisions (Fig.2b). The varying parameter was the confinement length: $0.1l_{web}$ for the first test unit TC and $0.15l_{web}$ for the second test unit TC-aw. The specimens failed due to concrete crushing of the web toe and buckling of vertical reinforcement in the lower part of the web. The drift capacities were 1.79% for test unit TC and 1.82% for test unit TC-aw, which corresponded only to a minor increase. The authors concluded that the confinement lengths were insufficient to reach the 2% drift as specified by the code.

Panagiotou and Restrepo (2011) reported tests of a full-scale seven-storey T-shaped wall on the shake table at the University of California, San Diego (Fig.2e). The test specimen was more representative of a building slice rather than an individual T-shaped wall, comprising also floor slabs at each level and two gravity columns at the toe of the web.. Loading was applied unidirectionally, along the web direction of the wall. The objective of the tests was to validate a displacement-based design approach of a reference building which resulted in a base shear coefficient equal to 50% of the coefficient obtained with the equivalent static method. The T-shaped wall test was the second phase of the test campaign which evaluated the influence of coupling the flange and the web that were initially decoupled in the first phase. Panagiotou and Restrepo (2011)concluded that the displacement-based design method gave satisfactory results for buildings with T-shaped walls, and that all performance objectives were met despite the reduced amount of longitudinal reinforcement that resulted from the design. Panagiotou (2008) had found that the framing effect, from the slabs and gravity columns with the T-shaped wall, led to an increase of strength and stiffness of the system.

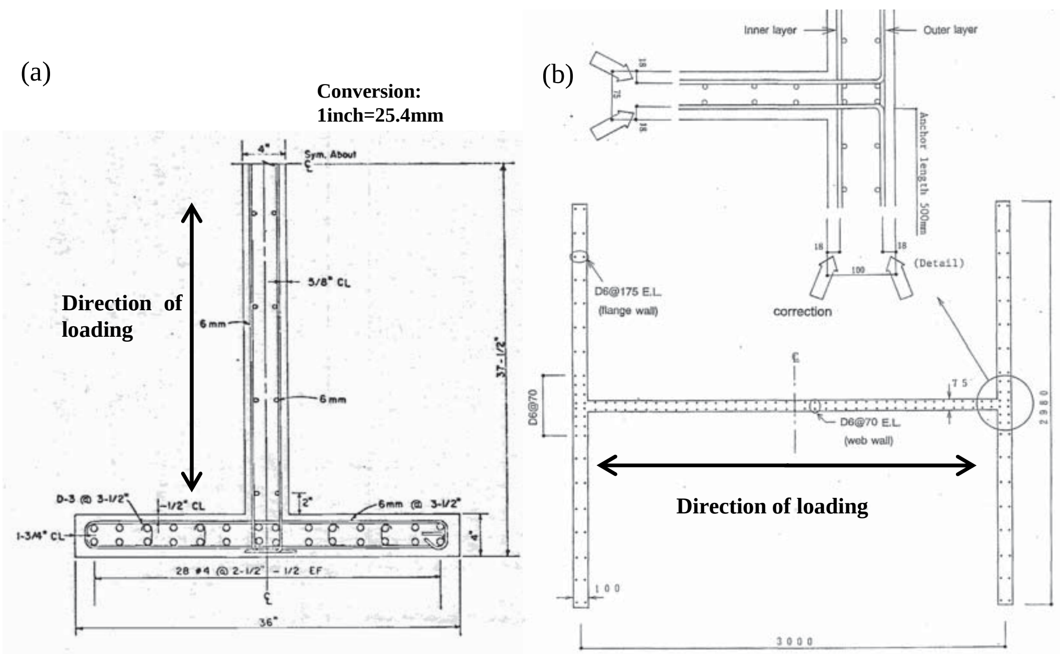

Figure 1. Flanged walls sections: half-section of F1 (a) (Oesterle et al., 1976) and (b) (NUPEC, 1996)

L-shaped walls

Most L-shaped walls tests were performed under quasi-static cyclic loading along the symmetry axis of the L-section. Nakachi et al. (1996) reported a test campaign on four L-shaped walls at 1:8 scale

(Fig.3a). The objective of the campaign was to study the influence of the confinement amount and area of the L-shaped corner on the deformation capacity of the wall. The test specimens represented the lower three storeys of a wall as part of a wall system in a 25 storey building and had very large axial load ratios of 0.6-0.8. The parameters varied were the axial load ratios and confinement details. All four walls failed due to concrete crushing that initiated in the corner of the L-shaped section. The authors concluded that adding confinement in the area near the corner improved the deformation capacity of the wall significantly.

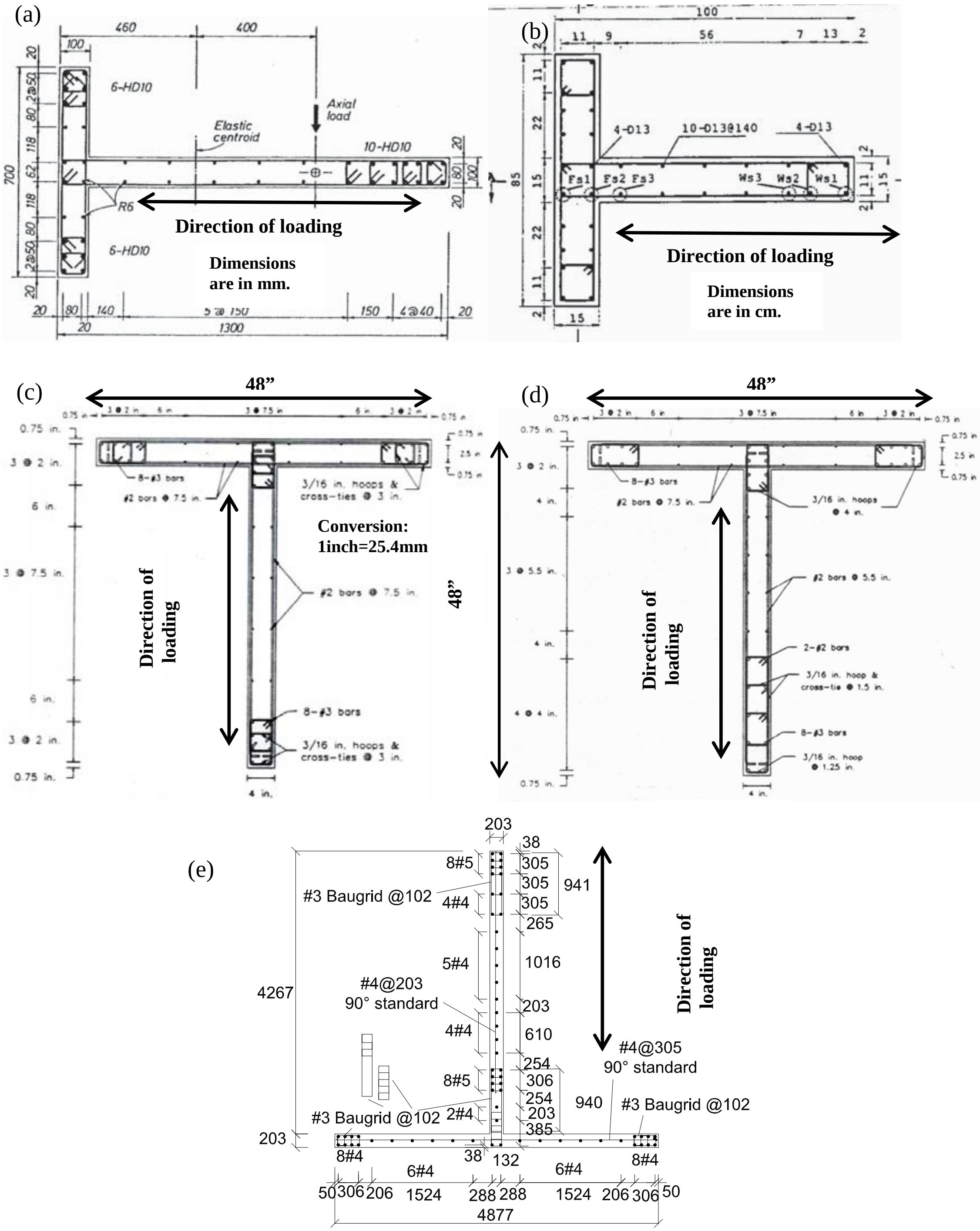

Figure 2. T-shaped wall test specimens: Wall 3 (a) (Goodsir, 1985), TC (b) (Choi et al., 2004), TW1 (c) TW2 (d) (Thomson and Wallace, 1995) and T-wall shake table test (Panagiotou and Restrepo, 2011)

Hosaka et al. (2008) discusses the experimental results of another test campaign on four L-shaped walls (L-1, L-2, L-5, L-6) representing the lower part of a core wall that is part of a wall system in a 30 storey building. The objective of the test campaign was the improvement of the wall deformation capacity when the corner of the L-shaped section was in compression. The applied axial load ratio was 0.15 at zero lateral displacement and varied from zero when the flange toes were in compression to 0.4 or 0.45 when the wall corner was in compression. The parameters of the test were the concrete compression strength and the layout of the confining reinforcement (stirrup geometry and spacing, confinement length and vertical bars in the confined core). L-1 and L-2 (Fig.3b) had twice the confinement lengths in the flange toes than what L-5 and L-6 had. Additionally L-1 and L-2 had overlapping stirrups over the corner area while for L-5 and L-6 the stirrups were not overlapping (Fig.3b) over the confinement length. The walls developed compression failure in the corner of the wall at the base. L-1 and L-2 reached larger ultimate drifts with the corner of the wall in compression than L-5 and L-6 confirming hence the benefit of an increase confinement length on the displacement capacity.

Inada et al. (2008) presented another test campaign on three L-shaped walls (Fig.3c) which investigated the influence of loading direction (loading along the axis of symmetry or along the one of the flange axes) and of section geometry (equal length of flanges or different length). The test prototype represented the lower three storeys of a RC core wall, part of a wall system in a 40 storey building. For all tests, the compressive axial load ratio was varied from zero when the flange ends were in compression to 0.2-0.3 when the corner of the L-section was in compression. The wall that was loaded along the flange direction failed due to crushing of the flange toe when the flange toe was in compression and due to concrete crushing in the flange perpendicular to the loading direction. The other two walls failed due to concrete crushing in the flanges starting from the corner region. For the wall with unequal flange lengths, the damage was more significant in the short flange than in the long one. The strain distribution from strain gages on the vertical reinforcement indicated that the vertical strains were no longer linearly distributed in the cross section, i.e., the hypothesis of plane sections remaining plane did no longer apply, after the concrete started to crush when the corner of the L-section were in compression and the flanges in tension. Inada et al. (2008) recommended increasing the confinement area around the corner of the L-shaped section.

Kono et al. (2011) reported two other tests on L-shaped walls which studied the influence of the axial load on the seismic behaviour of core walls (Fig.3d). The axial load was applied in a similar fashion as in the test campaign by Inada et al. (2008). For these tests, the axial load ratio varied from zero when the flange toes were in compression to 0.35 when the corner was in compression for test unit L45C and from 0.05 to 0.5 for L45D. Both walls failed due to crushing of the concrete in the wall corner and failure of the concrete struts in the flanges with some differences between the two in the extent of damage. Kono et al. (2011) concluded that the plane-section assumption is no longer valid after the concrete starts to crush.

A new test campaign on L-shaped walls and T-shaped walls was recently completed in Nanyang Technological University, Singapore. Publication of test results is currently underway (Zhang and Li, 2014).

C-shaped walls

Sittipunt and Wood (1993) tested two C-shaped walls at 1:4 scale under quasi-static cyclic loading along the symmetry axis of the wall (Fig.4a). The objective of this campaign was to investigate the behaviour of non-rectangular walls, determine their effective stiffness at various displacement levels and provide experimental data for verification of finite element results. The varying parameter between the two tests was the amount of web reinforcement (same amount for both longitudinal and horizontal) as in Fig.4b-c. Both walls failed due to crushing of the concrete in the boundary elements at the corners of web and flanges and bar buckling at 2% drift. Sittipunt and Wood (1993) based afterwards their conclusions on the numerical analyses performed using a shell model validated against the experimental data. They concluded that adding longitudinal and horizontal reinforcement to the unconfined concrete part of the web and flanges does not improve the energy dissipation capacity of the shear walls.

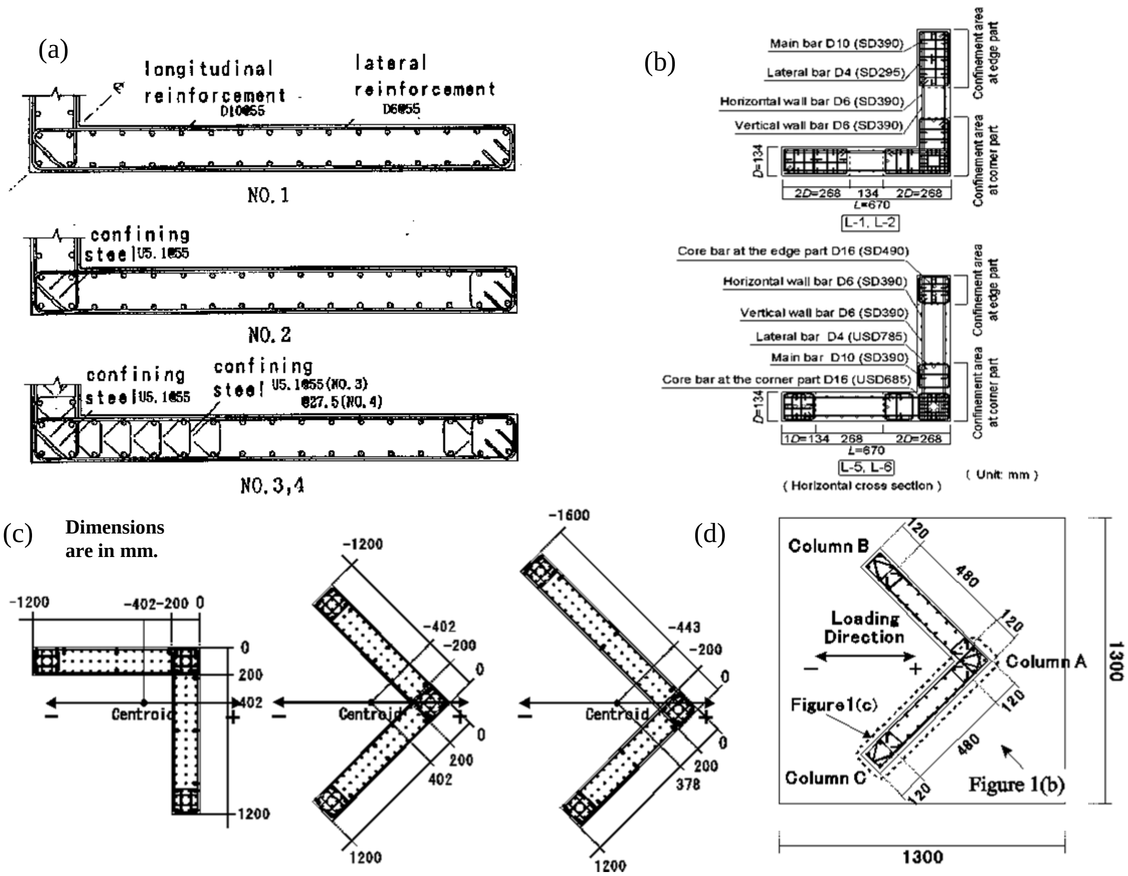

Figure 3. L-shaped test walls cross-section details: No.1 to No.4 (a) (Nakachi et al., 1996); L-1, L-2, L-5 & L-6 (b) (Hosaka et al., 2008); L00A, L45A & L45B (c) (Inada et al., 2008) and L45C&L45D (d) (Kono et al., 2011)

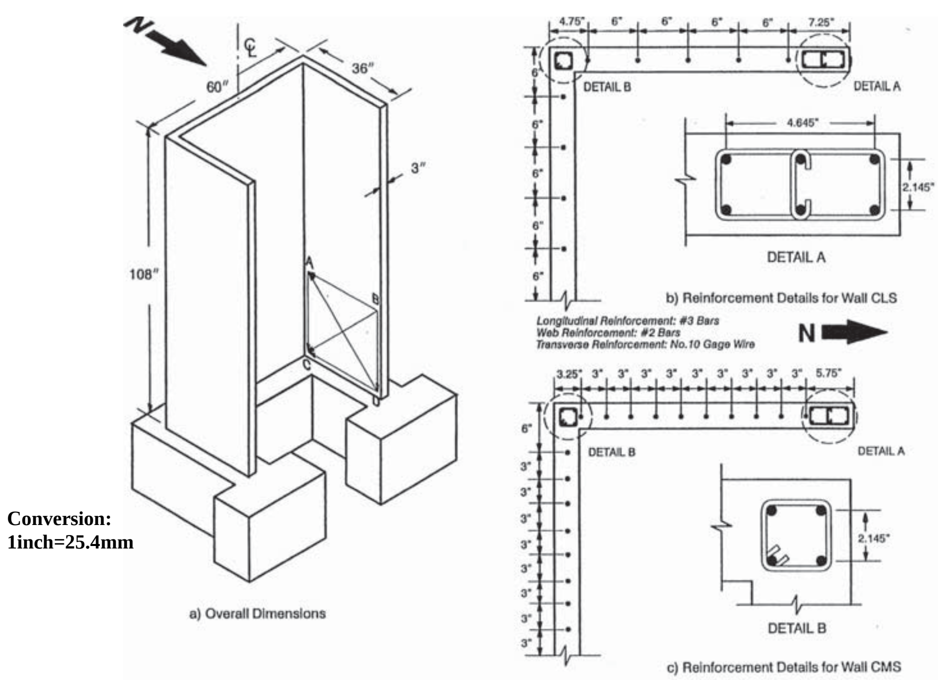

Figure 4. C-shaped wall tests: 3D view and cross sections of test units CLS and CMS (Sittipunt and Wood, 1993)

H-shaped wall

Maruta et al. (2000) tested nine H-shaped walls at 1:12 scale representing the lower part of a core wall in a high-rise building (Fig.5a). The objective was to investigate the behaviour of such walls under simultaneous lateral load and torsion. The test parameters were the ratio of torsional moment to bending moment and the lateral load direction with respect to the principal axes of the section (Fig.5b). Based on the test results Maruta et al. (2000) concluded that the resisting mechanism of an H-shaped wall subjected to both torsion and lateral load depends on the lateral loading direction. They established nearly spherical interaction surfaces between torsional and bending moments for H-shaped core walls for each loading direction. Also they concluded that a torsional to bending moment ratio less than 0.25 does not influence the bending moment capacity of the wall.

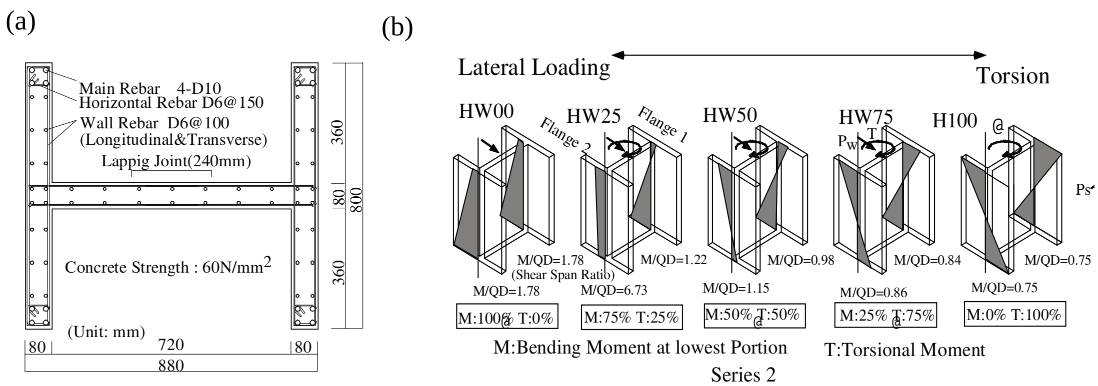

Figure 5. H-shaped wall tests: wall cross-section (a) and applied loading (b) (Maruta et al., 2000)

BIDIRECTIONAL LOADING

T-shaped wall

Two half-scale T-shaped core walls were tested under quasi-static cyclic loading at the MAST Laboratory of the University of Minnesota. The specimens represented the lower part of a six storey core wall subjected to lateral load testing using an inverted triangular distribution. The first specimen (NTW1) included the first four storeys of the wall while the second one (NTW2) included just the first two storeys. Floor slabs were also included in the test specimen at each floor level. Lateral load and superimposed moment were applied at the top of the specimens to achieve the desired moment shear ratio at the base of the wall. The objective of the test was to investigate the behaviour of the core wall under multi-directional loading. The main investigated parameters were the longitudinal reinforcement distribution in the flange and the length of the confined boundary elements (Fig.6a-b). The applied loading pattern included cyclic loading in the principal axes of the section and biaxial bending: (1) following the envelope of the yield drift, (2) an hourglass pattern and (3) skew loading at 45° and 72° (Fig.6c). Both walls first reached failure in the web direction with the web toe in compression due to concrete crushing in the confined core and bar fractures after previous buckling at 2.0% drift (NTW1) and 2.5% drift (NTW2). For the flange direction the walls failed due to concrete crushing and bar fracture after buckling in the boundary elements. Both flange direction failures occurred at 4.0% drift. Brueggen (2009) concluded that computing the confinement lengths from the drift capacities in the principal axes directions is appropriate and more advanced analysis to account for other loading directions is not required. However for further research they recommended to test walls with different proportions to failure in loading directions other than the principal ones.

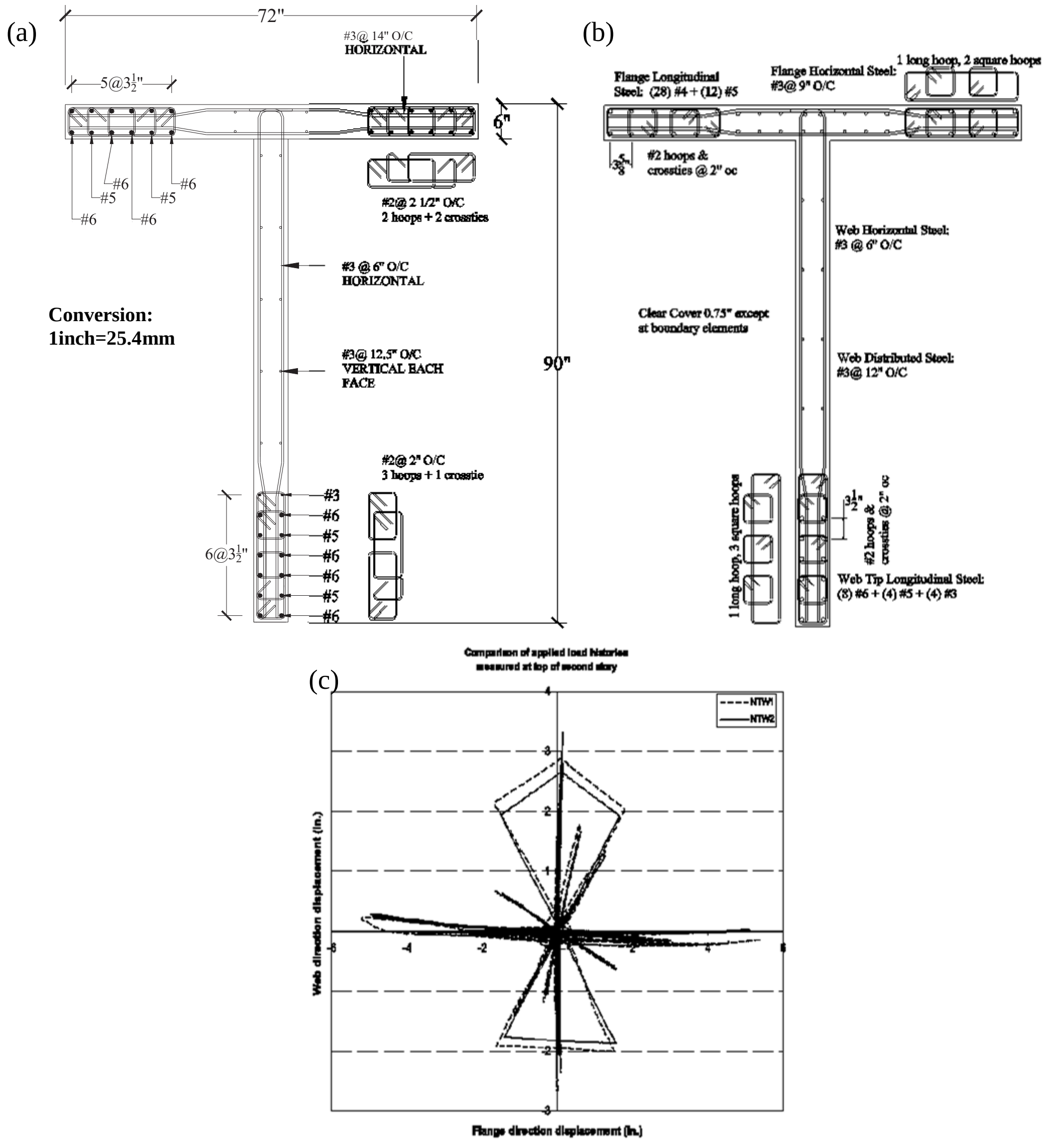

Figure 6. Minnesota T-shaped wall tests: cross-sections of test units NTW1 (a), NTW2 (b) and complex bidirectional loading history (c) (Brueggen, 2009)

U-shaped walls

Reynouard and Fardis (2001) reported a test campaign on six cantilever U-shaped walls to validate the 2001 version of Eurocode 8 (EC8) design provisions for RC walls (CEN, 2001). The campaign comprised two phases: unidirectional shake table tests and quasi-static cyclic tests. The U-shaped wall prototype represented a small elevator shaft. Three walls at 1:0.6 scale were subjected to unidirectional shake table tests along the symmetry axis of the section. The main test parameter was the spacing of the confinement stirrups. Wall behaviour was dominated by the rupture of longitudinal reinforcement bars which fractured prematurely due to very small ultimate strain capacity of 2.47%-2.61%. Very little crushing of the concrete was observed.

The three full-scale walls tested under quasi-static loading were identical from the point of view of geometry and reinforcement (Fig.7a). The varying parameter of these tests was the horizontal loading direction. IspraX and IspraY were subjected to unidirectional loading parallel to the web and the flanges, respectively. They both failed at ~3.1% drift due to longitudinal bar fracture after bar buckling in the boundary elements (in the corner web-flange for IspraX and at the flange end for

IspraY). IspraXY underwent bidirectional loading following a clover leaf pattern (Fig.7b) and reached failure at a drift of ~2.1% along each principal axis direction, which corresponds to a diagonal drift of ~3.0%. Failure of this test unit occurred at loading with one flange end in compression due to fracture of longitudinal reinforcement bars after buckling, followed by shear failure of the flange in compression. Reynouard and Fardis (2001) concluded that new confinement provisions to be included in EC8 were adequate and that shear demands may be critical on the flanges when U-shaped walls are subjected to biaxial bending. Hence they recommended that each wall flange is to be designed to carry individually the entire shear demand of the wall in the symmetry direction of the wall section.

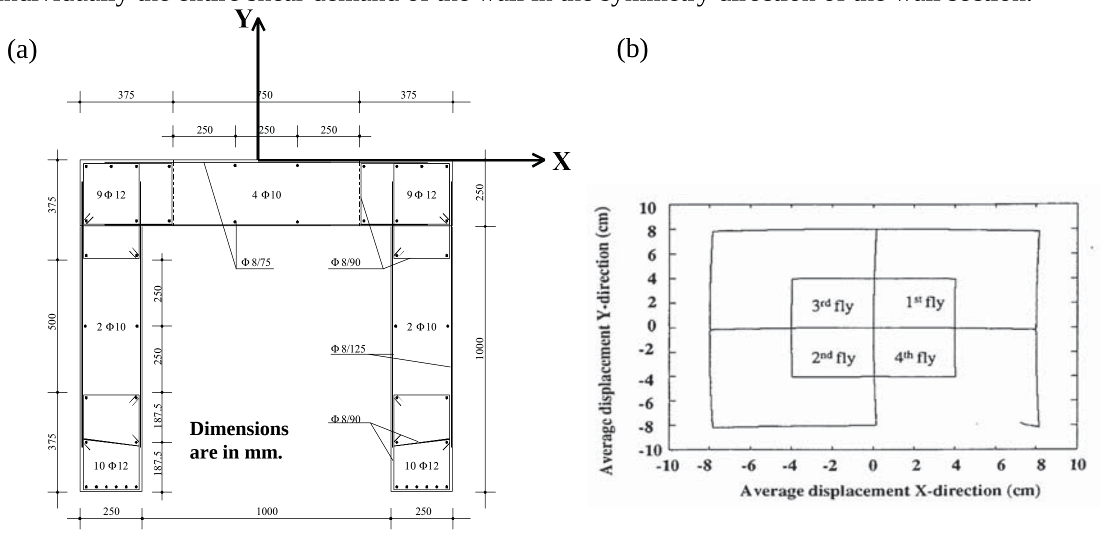

Figure 7. U-shaped wall tests in Ispra: wall cross-section (a) and bidirectional loading protocol (clover leaf pattern) (b) (Reynouard and Fardis, 2001)

Beyer et al. (2008) reported quasi-static cyclic tests on two half-scale cantilever U-shaped walls performed at ETH Zürich, representative for the lower two storeys of single core walls in a six storey building. The objective of the test campaign was to determine the behaviour of U-shaped walls under different loading directions hence the walls were subjected to a complex bidirectional loading history (Fig. 8c), which had been developed by Hines et al. (2002). The varying parameter between the two tests was the wall thickness: 15 cm for TUA and 10 cm for TUB (Fig.8a-b), which corresponded to 30 cm and 20 cm for the prototype unit. TUA failed during the diagonal cycle due to fracture of longitudinal bars after buckling, reaching diagonal drifts of 2.4% at position E and 2.9% at position F. TUB failed during the sweep cycles, when loading from H at 2.6% drift to B, due to crushing of compression diagonals in the unconfined part of the web. Beyer et al. (2008) indicated that the crushing failure of the web of TUB was the result of the complex bidirectional loading which favoured the extensive spalling of concrete in the web. Beyer et al. (2008) also found that the contribution of shear displacement to the total horizontal displacement of the wall: (1) was the largest when the wall section was under net tension, (2) was larger than for most rectangular walls and (3) varied with the loading direction and with wall section.

Constantin and Beyer (2014) presented results of two half-scale U-shaped cantilever walls tested at EPFL. The geometry and test setup of these walls was similar to those tested at ETH Zürich. The objective of the tests was to investigate the U-shaped wall behaviour under diagonal loading hence biaxial bending was applied along the two geometric diagonals of the U-shaped section (Fig.9b). The main test parameter was the axial load ratio: 0.06 for TUC and 0.15 for TUD (Figure 9b). TUC failed due to out-of-plane buckling of the boundary element of one flange toe after reaching a diagonal drift of 2.5%. TUD failed at 1.5% drift due to brittle concrete crushing in the boundary element of one flange toe end and simultaneous extension of concrete crushing to the entire flange. For both walls failure initiated in the confined boundary elements of the flange toes. Constantin and Beyer (2014) concluded that: (1) out-of-plane buckling of the flange toes is particularly promoted by diagonal loading, (2) plane sections do not remain plane under diagonal loading and (3) the critical loading case for determining confining lengths of the flange toes corresponds to the diagonal loading direction.

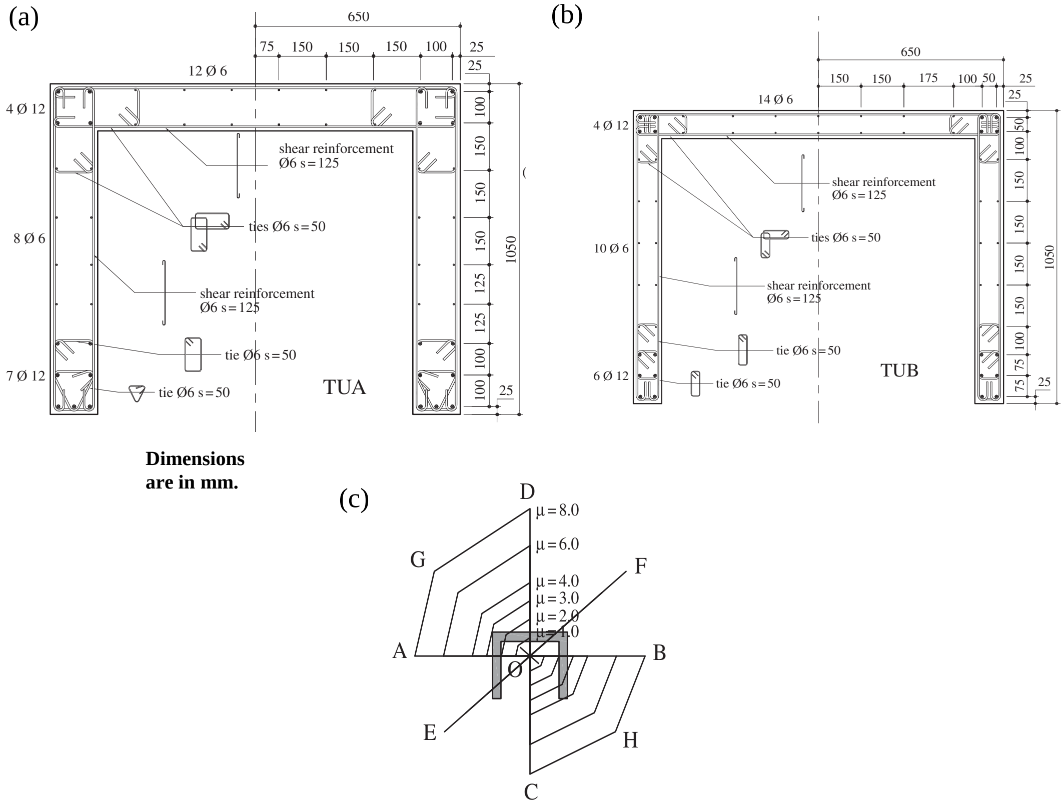

Figure 8. U-shaped wall test at ETH Zürich: TUA (a), TUB (b) and complex bidirectional history (c) (Beyer et al., 2008)

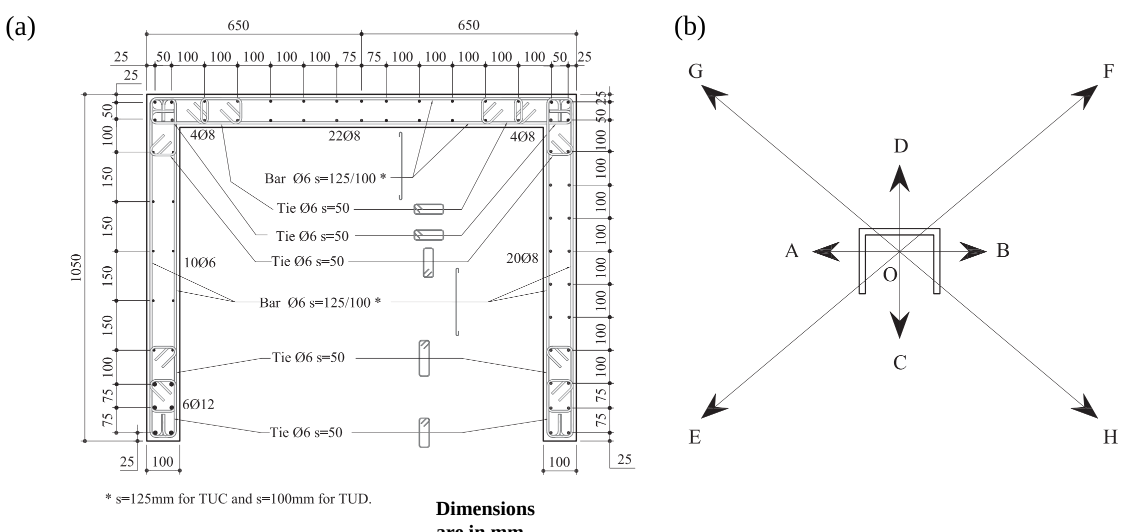

Figure 9. U-shaped walls tests in EPFL: wall cross-section (a) and diagonal loading history (b) (Constantin and Beyer, 2014)

C-shaped walls

Lowes et al. (2013) reported preliminary results of three C-shaped walls at 1:3 scale tested recently under quasi-static cyclic loading (Fig.10a). The prototype structure was a ten storey cantilever C-shaped wall subjected to lateral load under an inverted triangular distribution. The specimens included the lower three storeys of the prototype. The key objectives were to investigate the behaviour of the C-shaped walls under bidirectional loading, under variable axial load and to generate data for model calibration. Wall 6 was subjected to unidirectional loading along the strong-axis direction (direction of the web) while Wall 7 and Wall 8 underwent bidirectional loading along both strong-axis and weak-axis (Fig.10b). Wall 7 was also subjected to some biaxial bending towards the end of the test as a solution to testing equipment limitations. The axial load was kept constant throughout the tests for Wall 6 and Wall 7 while for Wall 8 the axial load was varied with the direction of the lateral load when loading along the weak-axis. This was done to simulate the coupling effect with another C-shaped wall as a core wall is often composed of two C-shaped walls connected by deep coupling beams. All three test specimens failed due to severe damage to the confined boundary element linking web and flanges as a result of concrete crushing and bar fracture after buckling. Damage to these boundary elements was expedited by web sliding at the wall-foundation interface after a large number of bar fractures occurred in the unconfined regions of the wall. Vertical separation between confined boundary elements and the unconfined concrete regions also occurred and it was attributed to the large sliding displacements (Lowes et al., 2013). Comparing the moment–drift hysteresis for the strong-axis direction, for Wall 6 (unidirectional loading) and Wall 7 (bidirectional loading), Lowes et al. (2013) concluded that after the yield drift, bidirectional loading led to a significant stiffness reduction of the wall in the strong-axis direction.

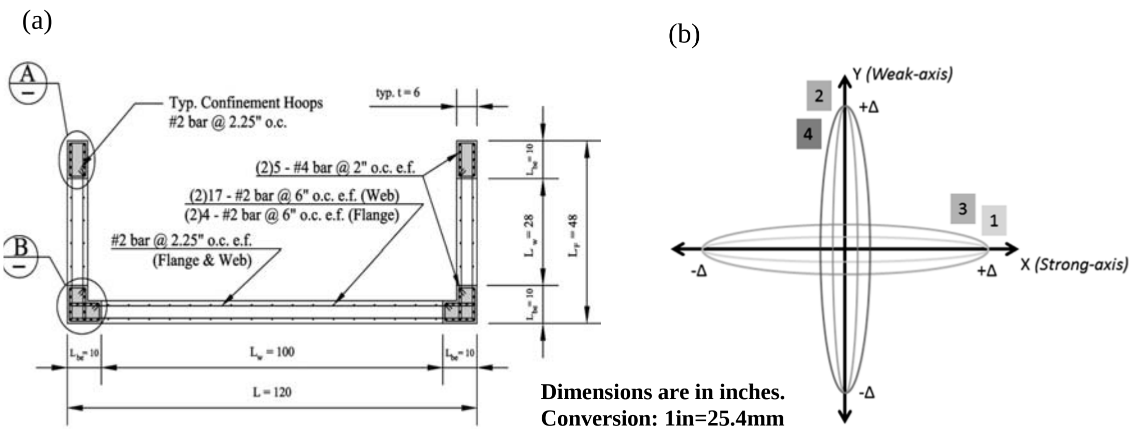

Figure 10. C-shaped wall test: wall section (a) and bidirectional loading history (b) (Lowes et al., 2013)

CONCLUSIONS

The article presents a review of experimental tests of non-rectangular walls. Experimental tests were categorised depending on the loading history and on the wall cross-section shape. Few test campaigns with biaxial bending and/or multi-directional loading were found as compared to unidirectional loading along the axis of symmetry of the section. The importance of bidirectional loading for non-rectangular walls was stressed by several researchers (Reynouard and Fardis, 2001; Beyer et al., 2008; Brueggen, 2009; Lowes et al., 2013; and Constantin and Beyer, 2014) because the determining loading direction for wall design may be different from the principal axes of the section and because loading in one direction reduces the wall strength and stiffness for subsequent loading in an orthogonal direction to the previous one.

REFERENCES

Beyer K, Dazio A, Priestley MJN (2008) "Quasi-static cyclic tests of two U-shaped reinforced concrete walls", *Journal of Earthquake Engineering* 12 (7):1023-1053

Brueggen BL (2009) Performance of T-shaped reinforced concrete structural walls under multi-directional loading, PhD Thesis, University of Minnesota, Twin Cities, MN

Constantin R, Beyer K (2014) "Behaviour of U-shaped reinforced concrete walls subjected to quasi-static cyclic tests under diagonal loading", *Engineering Structures*, to be submitted

CEN Eurocode 8 (2001): Design provisions for earthquake resistance of structures, Part 1: General rules, seismic actions and rules for buildings, Final Project Team Draft (stage 34) PrEN 1998-1:2001, Brussels (Belgium): European Committee for Standardisation

Choi C, Sang-Su H, Li-Hyung L, Young-Hun O, Hyun-Do Y (2008) "Evaluation of deformation capacity for RC T-shaped cantilever walls", *Journal of Earthquake Engineering* 8(3): 397-414

Goodsir WJ (1985) The Design of Coupled Frame-Wall Structures for Seismic Actions, Research Report 85-8, Dept. of Civil Engineering, University of Canterbury, Christchurch, NZ, 383p

Hines EM, Dazio A, Seible F (2006) "Structural Testing of the New East Bay Skyway Piers", *ACI Structural Journal*, January-February, 103(1): 103-112

Hosaka G, Funaki H, Hosoya H, Imai H (2008) "Experimental study on structural performance of RC shear wall with L shaped section", *Proceedings of the 14th world conference on earthquake engineering*, Beijing, China, 12-17 October

Inada K, Chosa K, Sato H, Kono S, Watanabe F (2008) "Seismic performance of RC L-shaped core structural walls", *Proceedings of the 14th world conference on earthquake engineering*, Beijing, China, 12-17 October

Kono S, Sakamoto K, Sakashita M (2011) "Simulation of seismic load resistance of core-walls for tall buildings", *Applied Mechanics and Materials*, 82: 386-391

Lowes L, Lehman D, Kuchma D, Mock A, Behrouzi A (2013) Large scale tests of C-shaped reinforced concrete walls: Summary report, NEES project warehouse; https://nees.org/warehouse/project/104

Maruta M, Suzuki N, Miyashita T, Nishioka T (2000) "Structural capacities of H-shaped RC core wall subjected to lateral load and torsion", *Proceedings of the 12th world conference on earthquake engineering*, Auckland, New Zeeland, 3 January-4 February

Nakachi T, Toda T, Tabata K (1996) "Experimental study on deformation capacity of reinforced concrete core walls after flexural yielding", *Proceedings of the 11th world conference on earthquake engineering*, Acapulco, Mexico, 23-28 June

NUPEC (1996) Seismic shear wall ISP: NUPEC's seismic ultimate dynamic response test, Comparison report, OECD Nuclear Energy Agency, Paris, France

Panagiotou M (2008) "Seismic design, testing and analysis of reinforced concrete wall buildings", PhD Thesis, University of California, San Diego, California

Panagiotou M, Restrepo JI (2011) "Displacement-Based Method of Analysis for Regular Reinforced-Concrete Wall Buildings: Application to a Full-Scale 7-Story Building Slice Tested at UC–San Diego", *ASCE Journal of Structural Engineering* 137(6):677-690

Palermo D, Vecchio FJ (2002) "Behavior of three-dimensional reinforced concrete shear walls," *ACI Structural Journal*, January-February, 99(1): 81-89

Oesterle RG, Fiorato AE, Johal LS, Carpenter JE, Russell HG, Corley WG (1976) Earthquake Resistant Structural Walls—Tests of Isolated Walls—Phase I, Report to National Science Foundation, Portland Cement Association Construction Technology Laboratories, Skokie, Illinois, USA

Oesterle RG, Aristizabal-Ochoa JD, Fiorato AE, Russell HG, Corley WG (1979) Earthquake Resistant Structural Walls—Tests of Isolated Walls—Phase II, Technical Report, Portland Cement Association, Skokie, Illinois, USA

Reynouard JM and Fardis MN, Eds (1993) Shear wall structures, ECOEST/ICONS Thematic Report No5, LNEC-National Laboratory of Civil Engineering, Lisbon, Portugal

Sittipunt C, Wood SL (1993) Finite element analysis of reinforced concrete shear walls, Report to National Science Foundation, Department of Civil Engineering, University of Illinois at Urbana-Champaign, Urbana, Illinois

Thomson IV JH, Wallace JW (1995) Displacement-based design of RC structural wall: An experimental investigation of walls with rectangular and T-shaped cross-sections, Report No. Cu/Cee-95-06, Department of Civil and Environment Engineering at Clark University

Zhang Z, Li B (2014) "Evaluation of seismic performance of slender L-shaped and T-shaped RC structural walls", *Proceedings of the 15th European conference on earthquake engineering*, Istanbul, Turkey, 24-29 August