

Ultimate Load Formula for Reinforced Concrete Wall Panels with Openings

J. H. Doh* and S. Fragomeni

Department of Civil Engineering, School of Engineering, Griffith University Gold Coast Campus,
PMB 50 Gold Coast Mail Centre, Queensland 9726, Australia

Abstract: Reinforced concrete walls with openings are tested in both one-way and two-way action. The test panels, with a slenderness ratio of 30 or 40, were subjected to a uniformly distributed axial load with an eccentricity of $t_w/6$. Apart from highlighting the experimental set-up; typical crack patterns, failure modes, and load-deflection behaviour are also reported. Utilising these and other published test results, a formula predicting the ultimate load of walls with openings was derived. A comparative study of predicted to test results indicates that the new formula is accurate and reliable.

Key words: concrete wall, openings, axial load, code method, slenderness ratio, design equation.

1. INTRODUCTION

In the past reinforced concrete walls were considered as non-load bearing and as such limited research was done on these elements. Due to the recent popularity of tilt-up construction and concrete cores in tall buildings reinforced concrete walls have become just as important structural element as beams, slabs and columns. The popularity has spread to Australia where prior to the 1990's limited experimental research was done on concrete panels. Since then a number of research projects focusing on the load capacity of concrete walls have been undertaken in Australia.

The simplified wall design equations given in the Australian Standard AS3600-01 and American Concrete Institute code ACI318-02 are intended only for solid load bearing walls supported at top and bottom (one-way action). These code provisions are unable to include the effects on load carrying capacity due to restraints on the side edges (two-way action). They also are restricted to walls with slenderness ratios of less than 30 and cannot account for openings such as doors or

windows. In the applications mentioned, concrete walls with higher slenderness ratios are being used which are outside the restrictions of current formulae; therefore more testing of these slender walls is required.

A wall (with one opening) behaving in one-way action where uniaxial curvature in the vertical direction occurs is depicted in Figure 1(a). In practice, axially loaded walls can also behave in two-way action, as shown in Figure 1(b), where side supports cause curvatures to occur in both parallel and perpendicular directions to that of loading. Typically two-way action occurs in cases where sides form part of a core wall system or where the sides are restrained by other interconnecting walls in the transverse direction.

Many researchers have investigated the behaviour of reinforced concrete walls either in one-way or two-way action. However, only a few studies involved walls with openings.

For solid walls in one-way action, Seddon (1956) contributed to the development of the British Standard (BS8110) formula which is similar to the AS3600-01

*Corresponding author. Email address: J.Doh@griffith.edu.au.

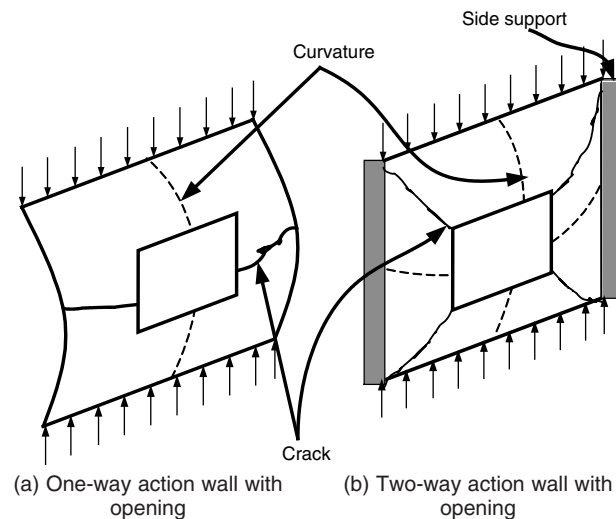
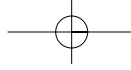


Figure 1. Walls with and without side's supports

equation. On the other hand, Oberlender (1973), Pillai and Parthasarathy (1977), Kripanarayanan (1977), Zielinski *et al.* (1982, 1983) and Saheb and Desayi (1989) have made significant contributions to the development and refinement of the ACI 318 equation. Fragomeni and Mendis (1999) provided a detailed review of these methods. Also, studies on two-way action solid walls were conducted by Swartz *et al.* (1974), Saheb and Desayi (1990a), Fragomeni and Mendis (1997a, 1997b) Sanjayan and Maheswaran (1999). Most of these studies focused on normal strength concrete panels with low slenderness ratios (i.e. $H/t_w < 30$).

Apart from the introductory work of Seddon (1956) on walls with openings, Saheb and Desayi (1990b) carried out a number of tests on walls with openings in both one and two-way action. However, the slenderness ratio (H/t_w) of the panels was lower than 12.

Recently, comprehensive study has been conducted by Doh *et al.* (2002) and Doh and Fragomeni (2005) on the behaviour of solid walls in both one and two-way action. Particular emphasis was given to testing panels with slenderness ratios between 25 and 40. The derived design formula from the comprehensive study is initially summarised herein. This is followed by a description of recent ultimate load tests on 12 half-scale wall panels with openings. The derived design formula for solid walls is then extended to cover walls with openings using these 8 half-scale and previous test results. The 8 half-scale panels were tested in one and two-way action with slenderness ratios between 25 and 40. This paper therefore reports on the experimental set-up, failure loads, typical crack patterns and load-deflection characteristics. Utilising these and other

published test results, a formula predicting the ultimate load of walls with openings was derived.

2. WALL DESIGN FORMULAE FROM SELECTED STUDIES

2.1. Doh and Fragomeni (2005) Design Formula for Solid Panel

Doh and Fragomeni (2005) conducted an extensive test program on normal and high strength solid concrete wall panels in one and two-way action with slenderness ratios (H/t_w) varying from 25 to 40. The study led to the conclusion that the current AS3600-01 method was inadequate and a reliable design formula would be needed. Using the test results and published data for walls with slenderness ratio lower than 25, the AS3600-01 equation was modified. Suitable for solid walls only, the design formula takes the form

$$N_u = 2.0f'_c{}^{0.7}(t_w - 1.2e - 2e_a) \quad (1)$$

where N_u = design axial strength per unit length of wall (N/mm), t_w = thickness of the wall (mm), f'_c = characteristic compressive strength of concrete (MPa), e = eccentricity of the load (mm), e_a = an additional eccentricity due to deflections in the wall (mm), or $e_a = (H_{we})^2/(2500t_w)$ in which $H_{we} = \beta H$ is the effective height (mm).

For one-way action, the effective height factor β ,

$$\beta = 1 \text{ for } H/t_w < 27, \text{ and } \beta = \frac{18}{\left(\frac{H}{t_w}\right)^{0.88}} \text{ for } H/t_w \geq 27$$

For two-way action, the effective height factor,

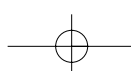
$$\beta = \alpha \frac{1}{1 + \left(\frac{H}{L}\right)^2} \text{ for } H \leq L, \text{ and } \beta = \alpha \frac{L}{2H} \text{ for } H > L$$

In these equations, the eccentricity parameter

$$\alpha = \frac{1}{1 - \frac{e}{t_w}} \text{ for } H/t_w < 27 \text{ and}$$

$$\alpha = \frac{1}{1 - \frac{e}{t_w}} \times \frac{18}{\left(\frac{H}{t_w}\right)^{0.88}} \text{ for } H/t_w \geq 27.$$

Comparison studies (Doh and Fragomeni 2005) were made between Eqn 1 and published methods including the equations recommended in AS3600-01 and ACI318-02 and those developed by Saheb and Desayi (1989, 1990a) and Fragomeni and Mendis (1997a,



J.H. Doh and S. Fragomeni

1997b). The findings indicated that Eqn 1 gave reliable results and may be used for the design of normal and high strength concrete wall panels subjected to one or two-way action. However, Eqn 1 is only applicable to solid concrete wall panels.

2.2. Saheb and Desayi Formula (1990b) for Walls with Openings

For panels with various types of openings (See Figure 2), Saheb and Desayi (1990b) proposed that the ultimate load is found by

$$N_{uo} = (k_1 - k_2\chi)N_u \quad (2)$$

where N_u is the ultimate load of an identical solid wall panel given by the Saheb and Desayi (1989, 1990a) design formulas:

for one-way action,

$$N_u = 0.55[A_g f'_c + (f_y - f'_c)A_{sv}] \left[1 - \left(\frac{H}{32t_w} \right)^2 \right] \quad (3)$$

for two-way action,

$$N_u = 0.67 f'_c A_g \{ 1 - [L/(120t_w)]^2 \} \{ 1 + 0.12(H/L) \} \quad (4)$$

k_1 and k_2 are respectively, obtained from the test results, 1.25 and 1.22 for one-way action and 1.02 and 1.00 for two-way action. The non-dimensional quantity

$$\chi = \left(\frac{A_o}{A} + \frac{\eta}{L} \right) \text{ in which } \eta = \left(\frac{L}{2} - \bar{\eta} \right) \text{ with } \bar{\eta} \text{ being the distance from the left vertical edge to the centre of gravity of the cross-section of the panel with openings, or } \bar{\eta} = \left(\frac{\frac{1}{2}t_w L^2 - t_w L_o \eta_o}{Lt_w - L_o t_w} \right); \text{ and, } A_o = L_o t_w.$$

In these equations L and L_o are the length of the wall panel and that of the opening, respectively. Figure 3 identifies some of the symbols used where G_1 and G_2 = centres of gravity of wall cross section with and without opening, respectively; G_3 = centre of gravity of the opening.

Eqn 2 combined with Eqn 3 or Eqn 4 is only applicable to concrete walls with slenderness ratio $H/t_w < 12$ and normal strength concrete only. Beyond such slenderness ratio or high strength concrete panels, the formulas may lead to inaccurate predictions. Hence, the scope of application is limited and further work is required to cover higher H/t_w ratios and high strength concrete.

3. TEST SPECIMENS AND TEST SET UP

3.1. Test Panels

In an attempt to extend a previously published ultimate load formula for solid walls (Eqn 1), 12 half-scale reinforced concrete wall panels with and without openings were tested to failure. The dimensions and material properties of the test panels are detailed in Table 1 where the symbols OW and TW indicate one and two-way action tests, respectively. The first digit following the symbols

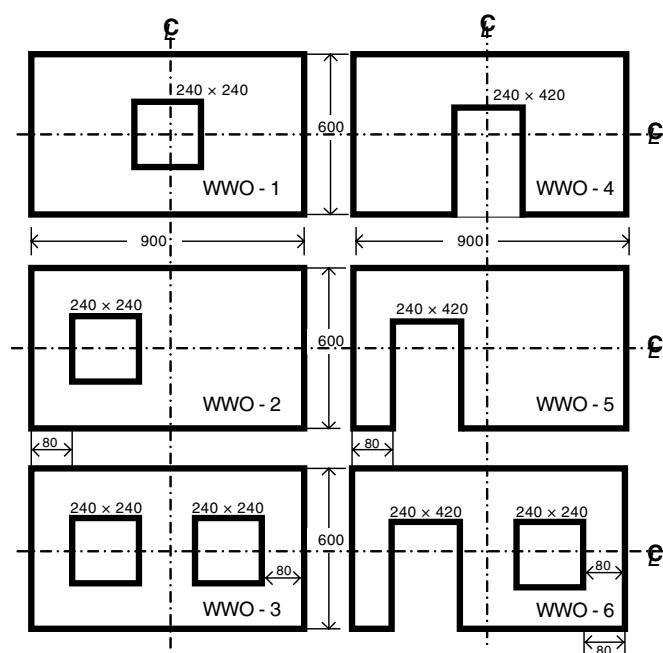


Figure 2. Details of Saheb and Desayi (1990b) wall panels (All dimensions in mm)

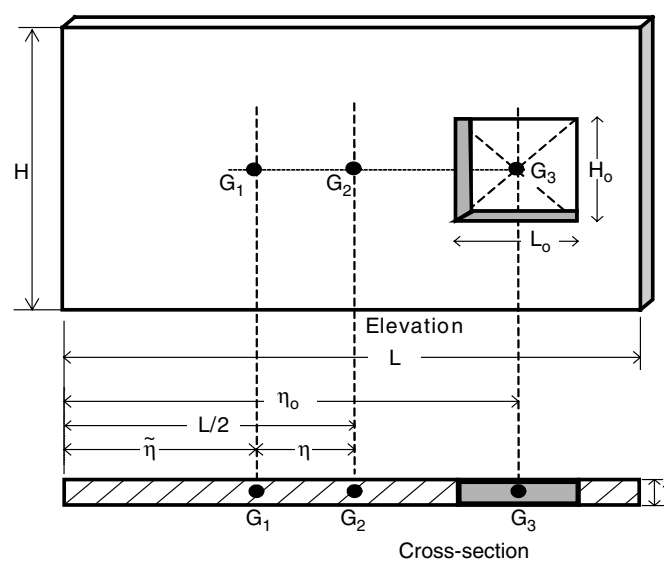


Figure 3. Geometry of wall openings in elevation and cross-section plan (Saheb Desayi 1990b)

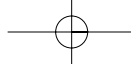


Table 1. Reinforced concrete wall dimensions and concrete strengths

Wall Panel	Height (H: mm)	Length (L: mm)	Thickness (t _w : mm)	Opening size (mm × mm)	Concrete strength (f' _c : MPa)	H/t _w
OW01	1200	1200	40	None*	35.7	30
OW02	1600	1600	40	None*	51.0	40
OW11	1200	1200	40	300×300	53.0	30
OW12	1600	1600	40	400×400	47.0	40
OW21	1200	1200	40	300×300	50.0	30
OW22	1600	1600	40	400×400	51.1	40
TW01	1200	1200	40	None*	37.0	30
TW02	1600	1600	40	None*	45.8	40
TW11	1200	1200	40	300×300	50.3	30
TW12	1600	1600	40	400×400	50.3	40
TW21	1200	1200	40	300×300	50.3	30
TW22	1600	1600	40	400×400	50.3	40

** Previous test result (Doh and Fragomeni, 2005).

denotes the number of openings; the second digit denotes the slenderness ratio with 1 for H/t_w = 30 and 2 for 40.

All wall panels were reinforced with a single F41 mesh, placed centrally within the panel cross-section. The F41 mesh has a design yield strength of 450 MPa and the minimum tensile strength was 500 MPa. The reinforcement ratios ρ_v and ρ_h were 0.0031 for all panels, satisfying the minimum requirements in the Australian Standard and the ACI code.

Reinforcing was also placed diagonally in each corner of the openings to prevent shrinkage cracking. This consisted of strips cut from F41 mesh. Three of these strips were tied in each corner. The length of each strip was the same as the dimension of the side of the opening; 300 mm for the small walls and 400 mm for the larger walls. The fixing and the layout of the corner reinforcement are shown in Figure 4.

The concrete was supplied by the local ready-mix

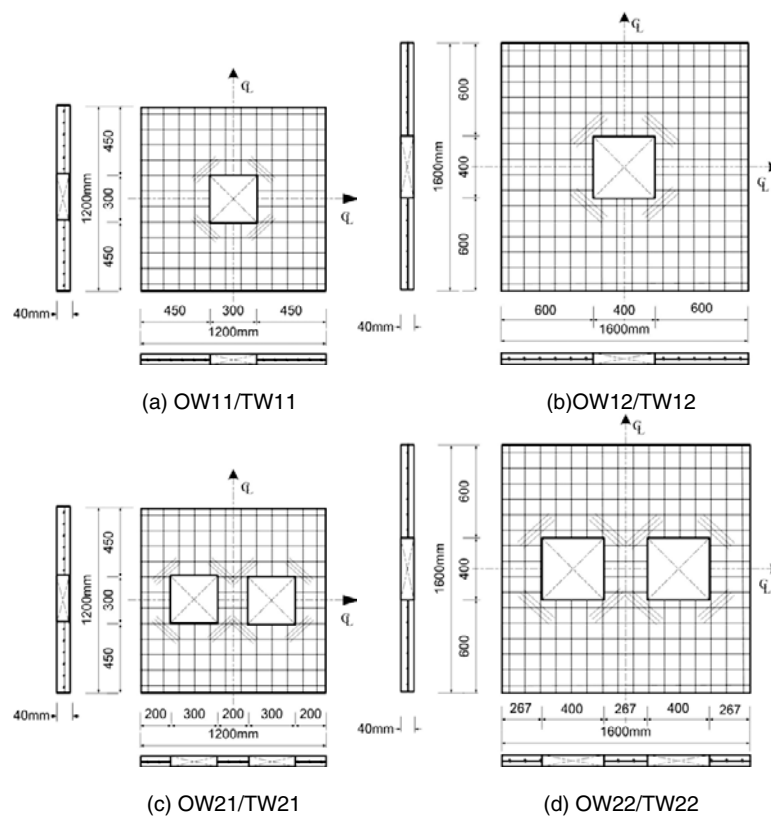
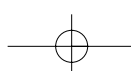


Figure 4. Dimension and arrangement of steel mesh in wall specimens



J.H. Doh and S. Fragomeni

company. The concrete requirements were a compressive strength, 32 or 50 MPa, a slump of 80 mm and a maximum aggregate size of 10 mm.

3.2. Test Set-Up

An arrangement of the test set-up is shown in Figure 5. The test frame was designed to support three independent hydraulic jacks each of 80 tonne capacity. The jacks transmit a uniformly distributed load across the top through a loading beam at an eccentricity of $t_w/6$.

The top and bottom hinged support conditions were each simulated by placing a 23 mm diameter high strength steel rod on a 50 mm thick steel plate of 150 mm width and varying lengths which corresponded to the different test panel dimensions. Two 20 mm x 20 mm angle sections were clamped to the thick plate by bolts and the 23 mm diameter rod was welded along the length of the plate. Details of the simply supported top hinged edge are shown in Figure 6.

To achieve the hinged support conditions for two-way action, the edges of the panels had to be effectively stiffened so that rotation about the x-axis was prevented while they were free to rotate about the y-axis. To achieve this, two parallel flanged channel sections separated by a square hollow section extend along the height of both sides of the test panel (see Figure 7).

Dial gauges were used to measure the lateral deflections of the wall panels during testing. The positioning of the dial gauges for the wall panels with one opening are indicated in Figure 5 and Figure 8 (a). For the wall panels with one opening, all dial gauges were positioned midway between the edges of the panel and the edges of the opening. For the wall panels with two openings, the top, bottom and side gauges were all positioned midway between the edges of the panel and the edges of the opening. The final dial gauge was placed in the very centre of the panel as shown in Figure 8 (b).

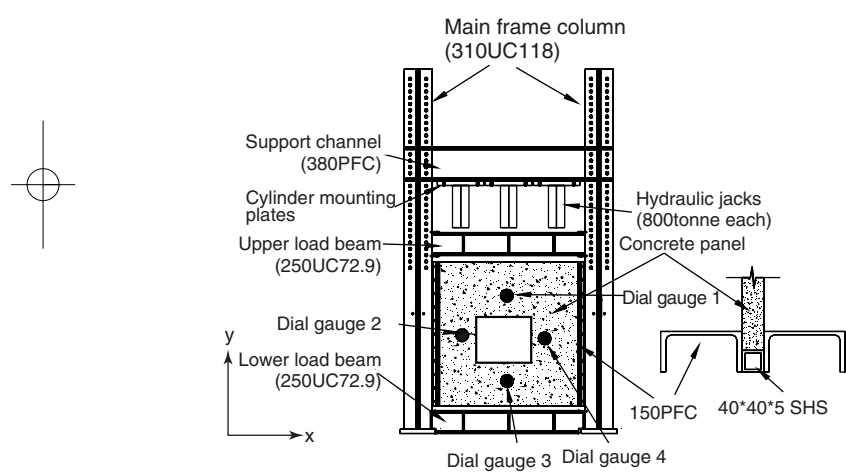


Figure 5. Test frame arrangement and side supports

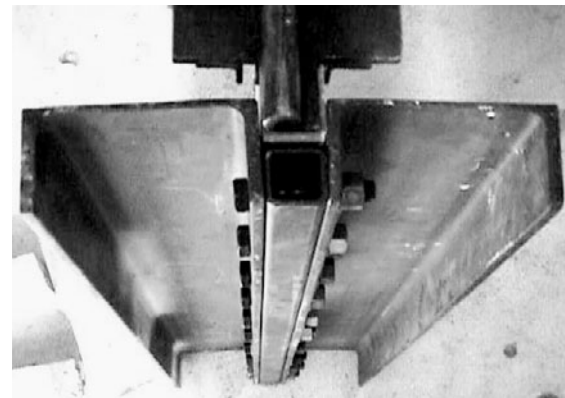
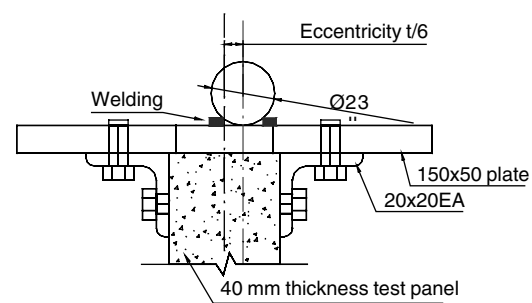


Figure 7. Top view of side restraint

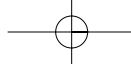


(a) Side view of top hinged edge



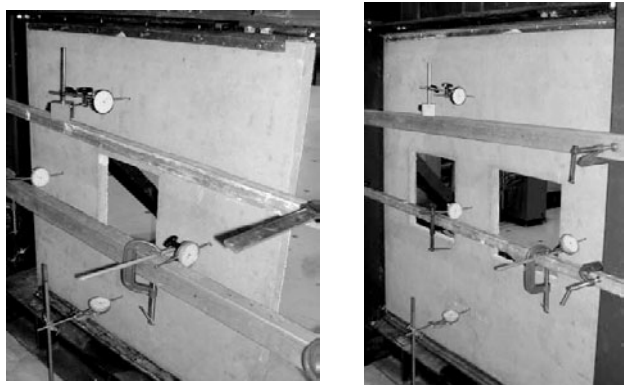
(b) Details of top hinged edge

Figure 6. Top and bottom restraint



3.3. Test Procedure

A load cell was positioned between the centre hydraulic jack and the upper load beam. The walls were loaded in 20 kN increments, the load reduced close to failure. At each load increment, crack patterns and deflections were recorded. The latter allowed the load-deflection history to be accurately traced.



(a) Wall panel with one opening (b) Wall panel with two openings

Figure 8. Dial gauges locations

4. RESULTS

4.1. Crack Pattern

Figures 9 to 18 show the crack patterns observed on the flexural tension faces after failure of selected test panels.

The one-way action solid panel (see Figure 9) deflected in a single curvature in the vertical direction with maximum deflection occurring along the centre of the panels. The crack patterns were horizontal (perpendicular to the loading direction) with failure occurring near the centre of the panels, signifying bending failure. On the other hand, the crack patterns of two-way action solid panel (see Figure 10) developed in a two-way, biaxial curvature manner. In addition, cracks were observed at the corners of the wall panels. Typically, explosive types of failures were observed for all panels.

For one-way walls with openings (see Figures 11 to 14), it was noted that horizontal crack patterns

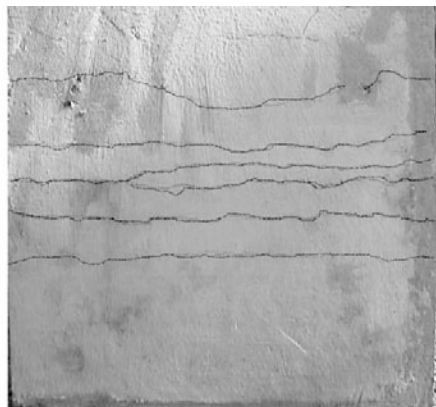


Figure 9. Crack pattern of OW01

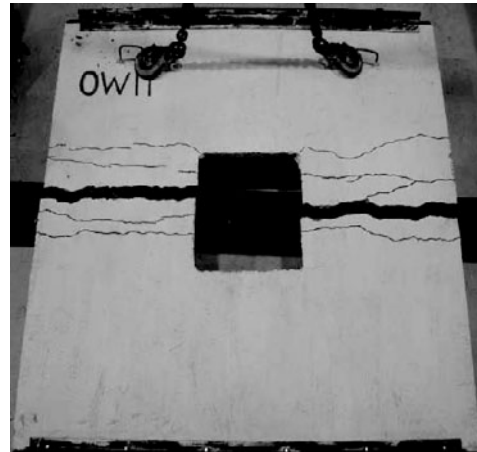


Figure 11. Crack pattern of OW11

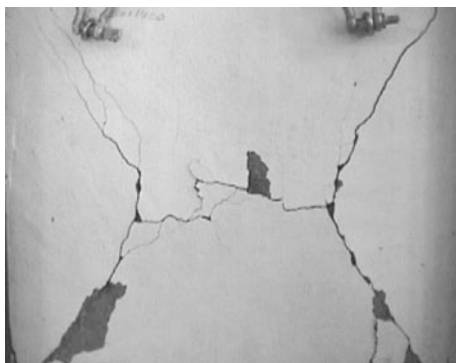


Figure 10. Crack pattern of TW02

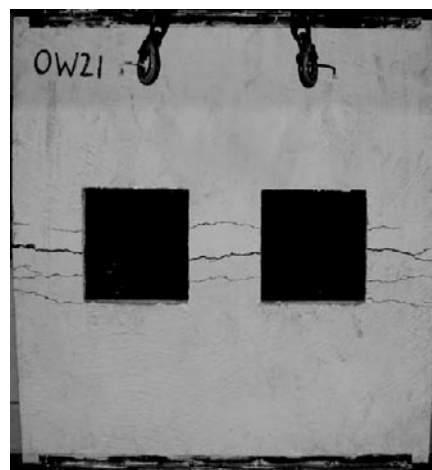
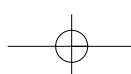


Figure 12. Crack pattern of OW21



J.H. Doh and S. Fragomeni



Figure 13. Wall Crack pattern of OW12

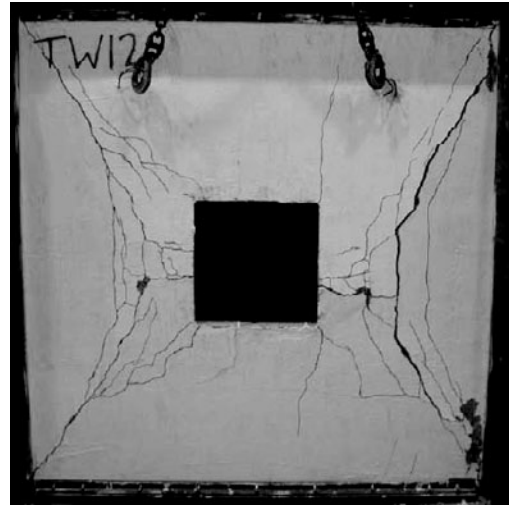


Figure 16. Crack pattern of TW12

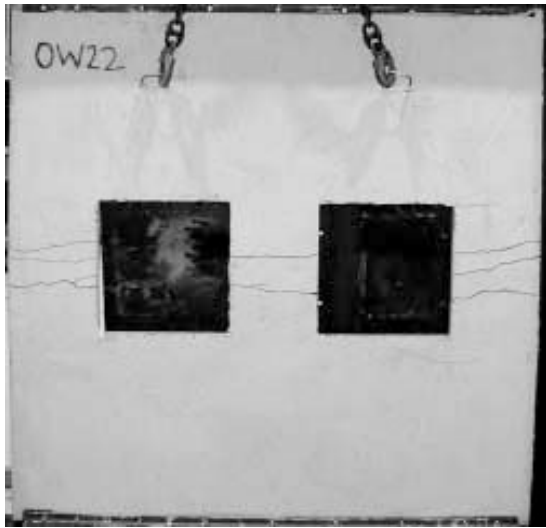


Figure 14. Wall Crack pattern of OW22

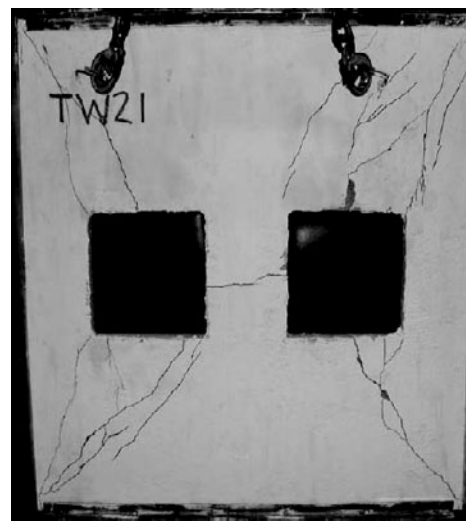


Figure 17. Crack pattern of TW21

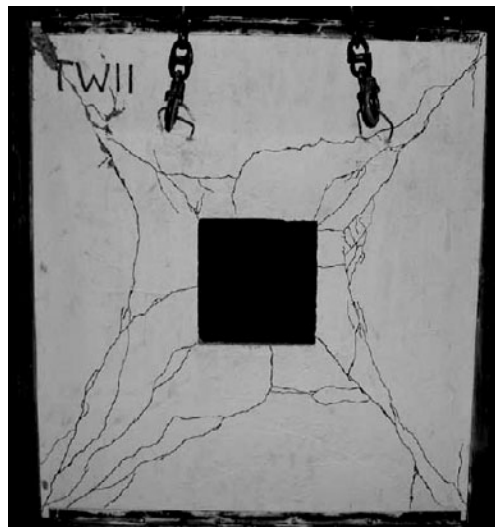


Figure 15. Crack pattern of TW11

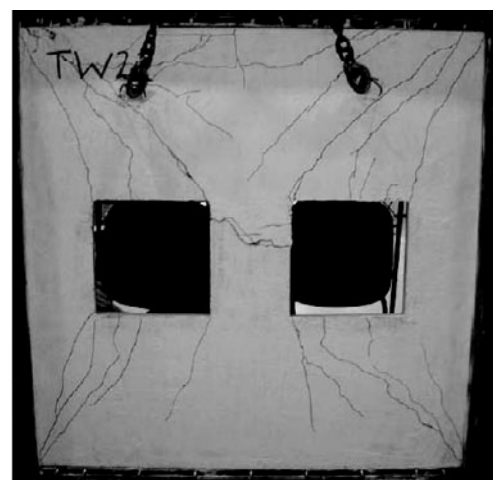
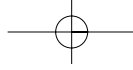


Figure 18. Crack pattern of TW22



developed in the column sections only, which is logical since this is the area where the cross-section is reduced.

For two-way wall panels it is evident, from Figures 15 to 18, that cracking is typical and similar to two-way solid panels. However the panels with two openings (Figures 17 and 18) show failure occurs horizontally through the middle column element as well as from outer edges, thus again illustrating that failure is critical within the column element. Also cracking is also initiated at the corners of the openings. This suggests that the interior corners act as stress concentrators and thus initiating failure.

4.2. Deflection

Typical load versus lateral deflection relationships for the wall panels tested in both one and two-way actions

are shown in Figures 19 to 23. It should be observed that the maximum deflections shown were obtained just prior to failure load being reached. Most of the panels tested failed in a brittle mode and the sudden failure of these panels made it difficult to record deflection precisely at failure. Thus in these figures, the absolute maximum failure loads and the corresponding maximum deflections are not shown.

Figures 19 and 21 show that the deflections at the wall centre are generally proportional to those at the side points. This indicates that the centre and side points move in an approximate single curvature manner in the vertical direction, which is typical of one-way behaviour.

The curves for the one-way action solid panels (see Figure 19) show that the walls exhibited ductile failure

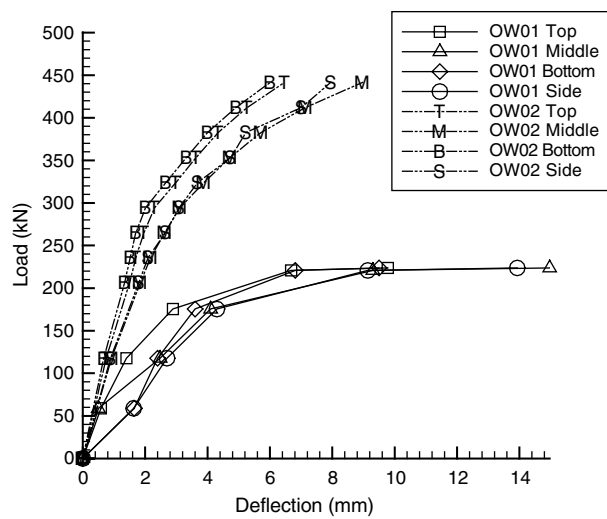


Figure 19. Load versus lateral deflection curves for wall OW01 and OW02

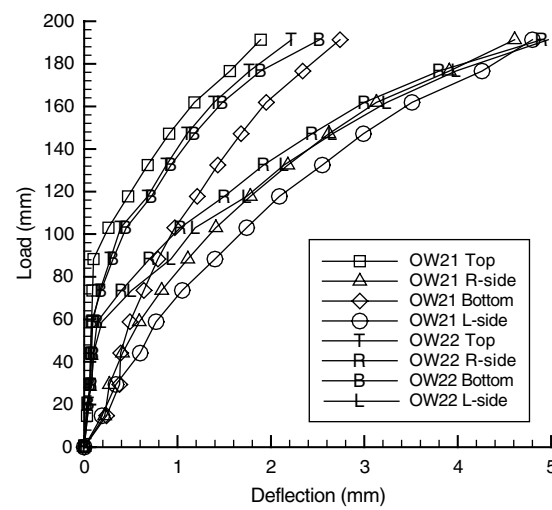


Figure 21. Load versus lateral deflection curves for wall OW21 and OW22

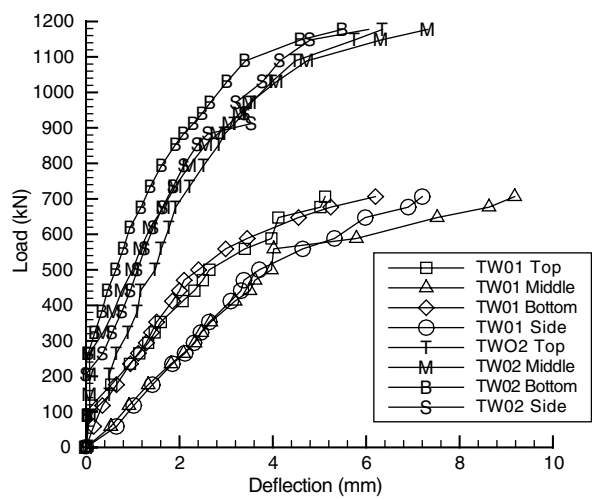


Figure 20. Load versus lateral deflection curves for wall TW01 and TW02

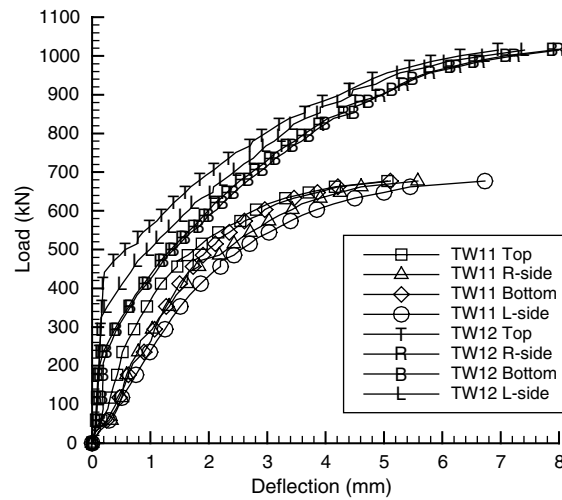
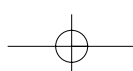


Figure 22. Load versus lateral deflection curves for wall TW11 and TW12



J.H. Doh and S. Fragomeni

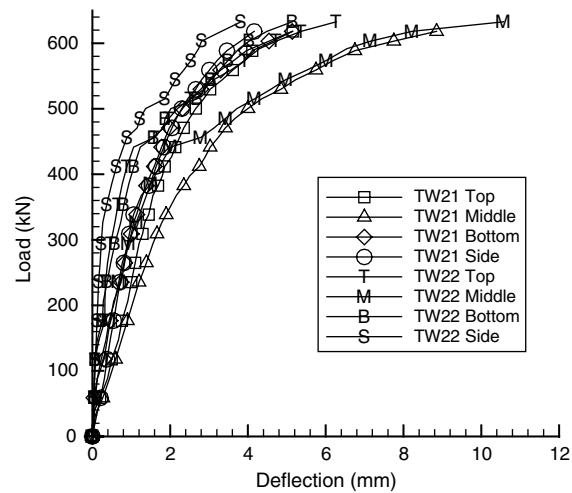


Figure 23. Load versus lateral deflection curves for wall TW21 and TW22

behaviour. This was reflected in the continually increasing values of the deflections as the test loads approached failure. The OW01 curves were linear for the initial loading regimes, and then followed by non-linear trends with lateral deflections increasing rapidly as failure was approached. The linearity of the curves was up to 40 to 50% of the ultimate loads for OW01. Figure 19, also shows that the one-way action panels exhibited a ductile failure type.

For the two-way solid panel of TW02 (see Figure 20), the curves are generally linear for the initial loading regions, followed by an increasingly non-linear curve as the lateral deflections increase rapidly, as the loads approach failure. The linearity was up to about 70 % of

the ultimate loads in the concrete panels. TW01 and TW02 specimens showed a brittle type of failure in which they were unable to sustain any further loading after reaching the maximum load.

The maximum deflection could not be obtained for the two-way action panels with one opening. Hence, the deflection readings were taken midway between the edge of the panel and the edge of the opening. In the case of two-way panels with one opening the maximum deflection would actually occur equally around the very edge of the opening. Generally, the deflections at quarter point over the length of the walls at mid-height were quite similar to top and bottom-points deflections. This was obvious as the side, top and bottom-quarter points should have moved, approximately, by the same amount due to the curvatures taking place in both the vertical and horizontal directions.

Figures 22 and 23 show that the lateral deflections of the panel with openings in two-way action are much larger than the lateral deflections of the corresponding walls in one-way action. This is mainly due to the fact that the two-way walls had a much greater failure load than the same wall tested in one-way action. If the lateral deflections of a one-way and two-way wall of the same model are compared at the same load, the one-way wall has a much larger deflection.

4.3. Failure Load

The ultimate loads and the axial strength ratios, $N_u/f'_c L t_w$, together with the concrete strengths are shown in Table 2. for all the test panels.

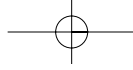
It is obvious in Table 2 that the failure loads for the two-way panels are much higher than the corresponding

Table 2. Deflection, failure load and axial strength ratio of panels

	Wall Panel	Concrete Strength (f'_c :MPa)	Failure Load (kN)	Axial strength ratio ($N_u/f'_c L t_w$)	Reduction strength ratio (%)
One-way action	OW01	35.7	253.10	0.148	8% ⁺
	OW02	51.0	441.45	0.135	
	OW11	53.0	309.02	0.121	19%
	OW12	47.0	294.30	0.098	
	OW21	50.0	185.41	0.077	22%
	OW22	51.1	195.71	0.060	
Two-way action	TW01	37.0	735.75	0.414	3%
	TW02	45.8	1177.20	0.402	
	TW11	50.3	750.47	0.414*	-2%
	TW12	50.3	1030.05	0.427*	
	TW21	50.3	618.03	0.512*	21%
	TW22	50.3	647.46	0.402*	

* Effective length was used for L where $L_{eff} = L - L_0$ where L_0 is opening size.

⁺ Reduction strength ratio was obtained as $(OW01 - OW02) \times 100 / OW01$.



one-way specimens with comparative strength increases ranging from 2.4 to 3.5. The axial strength ratio for panels (with or without openings) in one-way action tends to decrease with an increased slenderness ratio. This is also generally true for two-way panels except for panel TW12. The axial strength ratios for the two-way panels are higher than those for the one-way panels. This ratio decreases as the opening in the one-way panels increased from one to two. This effect is not obvious in the two-way panels.

Further, Table 2 indicates that for the one-way panels, an increase in the slenderness ratio from 30 to 40 has led to a strength reduction of 8%, 19% and 22% for the panels with 0, 1 and 2 openings respectively. The higher reduction variation reflects the presence of openings. To check the accuracy of the current code recommendations the panel failure loads in Table 2 are plotted in Figure 24, together with the failure lines predicted by the AS3600-01 and ACI318-02 wall design equations. The conservative nature of the code provisions is evident. For example at $H/t_w = 40$, the test panels, especially those in two-way action, showed significant load carrying capacities whereas neither the Australian standard nor the American code allows for any strength at this slenderness. It should be noted that the current AS3600-01 and ACI318-02 equations are applicable to solid walls in one-way action only. This may explain the apparent conservative nature of the prediction formulae, and demonstrates the need for a revamped formula for high strength concrete two-way wall panels, with and without openings. Finally, it seems that the presence of openings for walls in two-

way action shows inconclusive results on the effect of axial strength ratio of walls when slenderness ratios are higher than 30. More testing should be undertaken to clarify this effect.

5. PROPOSED ULTIMATE LOAD FORMULA

To develop the new ultimate load formula, Eqn 2 by Saheb (1985) was used in conjunction with Eqn 1 by Doh and Fragomeni (2005). That is, the ultimate load capacity is

$$N_{uo} = (k_1 - k_2\chi)N_u \quad (5)$$

where N_u may be calculated using Eqn 1 and χ is as defined previously in Section 2.1.

This equation, however, requires a calibration process to produce new sets of k_1 and k_2 factors. The N_{uo}/N_u ratio is plotted against χ in Figures 25 and 26 for the one-way and two-way panels respectively. Note that the N_{uo} and N_u in these plots are the present test values and those of Saheb (1985). A standard regression analysis yielded $k_1=1.188$ and $k_2=1.175$ for one-way action walls (giving a correlation coefficient of 0.84), and $k_1=1.004$ and $k_2=0.933$ for two-way action walls (giving a correlation coefficient of 0.96).

5.1. Comparison of Test Results

Table 3 gives test panel predicted and test failure loads including the ratio of predicted/test. It shows that the new formula gives conservative predictions with a mean ratio of 0.92 and a standard deviation of 0.08. This indicates the proposed equation gives a good prediction of panel failure loads.

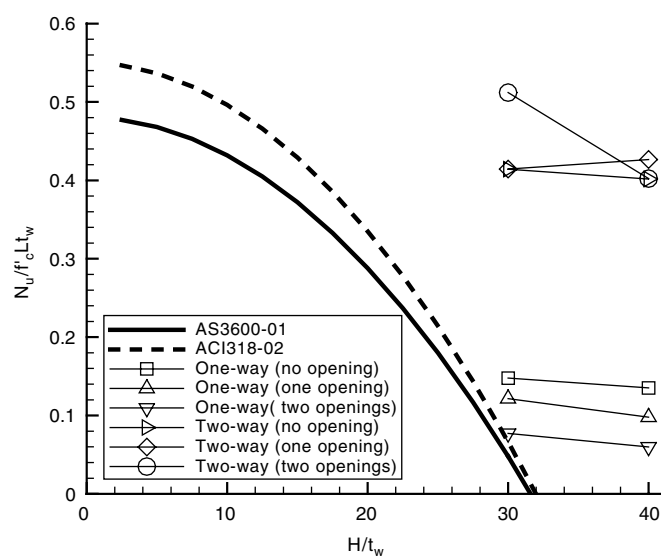


Figure 24. Axial strength ratio versus slenderness ratio ($f'_c = 30$ MPa for both code methods)

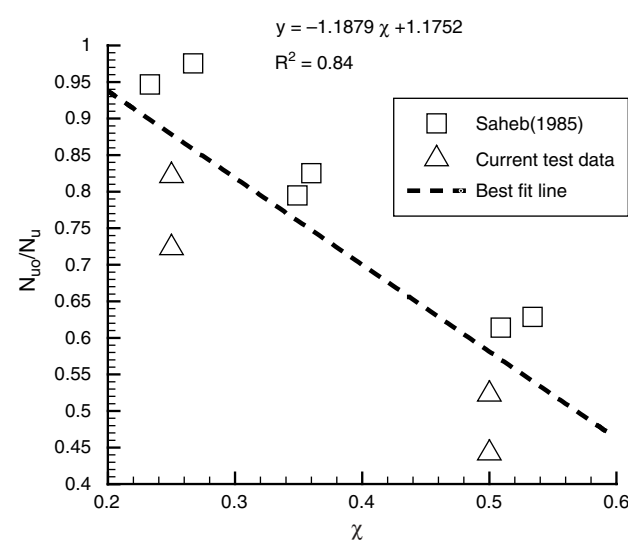
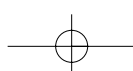


Figure 25. Experimental N_{uo} / N_u vs χ for panels with openings in one-way action



J.H. Doh and S. Fragomeni

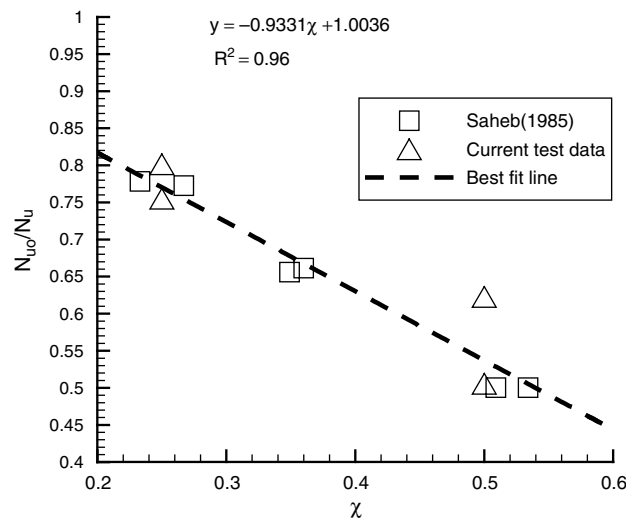


Figure 26. Experimental N_{uo} / N_u vs χ for panels with openings in two-way action

Table 3. Comparison of predicted ultimate loads and current test results

Wall Panels	Test (kN)	Eqn 5* (kN)	Eqn 5* Test
OW01	253.10	250.54	0.99
OW02	441.45	344.64	0.78
OW11	309.02	290.30	0.94
OW12	294.30	285.97	0.97
OW21	185.41	184.38	0.99
OW22	195.71	200.68	1.03
TW01	735.75	707.70	0.96
TW02	1177.20	1067.90	0.91
TW11	750.47	676.21	0.90
TW12	1030.05	878.52	0.85
TW21	618.03	471.44	0.76
TW22	647.46	612.48	0.95
Mean			0.92
Standard deviation			0.08

*New method which is combination of Eqn 1 and modified k factors.

5.2. Comparison with Saheb's Test Results (1985)

Saheb (1985) tested a total of 12 wall panels with openings (six in one-way action and six in two-way action), with various opening configuration as shown in Figure 2.

For Saheb's 12 test panels, the predicted (using Eqn 5 and Eqn 2) and test results are presented in Table 4. The comparison shows that the new formula is accurate but slightly conservative with a mean predicted/Test ratio of 0.95 and a standard deviation of 0.07. It is obvious that Saheb's approach is highly conservative with a mean ratio of 0.65 and a standard deviation of 0.23.

Table 4. Comparison of predicted ultimate loads and Saheb's test results (1985) ($f'_c=28.2$ MPa for all cases)

Wall Panels	Test (kN)	Eqn 5*	Eqn 2**
		Test	Test
One-way action			
WVO-1	672.56	0.95	0.98
WVO-2	568.90	0.98	0.83
WVO-3	433.47	0.93	0.63
WVO-4	652.65	1.03	0.95
WVO-5	548.02	1.09	0.79
WVO-6	423.47	1.01	0.61
Two-way action			
WVO-1P	692.47	0.86	0.77
WVO-2P	592.83	0.89	0.66
WVO-3P	448.38	0.89	0.53
WVO-4P	697.47	0.89	0.23
WVO-5P	587.83	0.95	0.35
WVO-6P	448.38	0.93	0.51
Mean		0.95	0.65
Standard deviation		0.07	0.23

* New method which is combination of Eqn 1 and modified k factors.

** Calculated using Eqn 3 or Eqn 4 due to Saheb (1985).

6. CONCLUSIONS

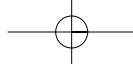
An experimental study was undertaken on twelve reinforced concrete walls with and without openings in one and two-way action. Loaded with an eccentricity of $t_w/6$, these half-scale specimens had high slenderness ratios between 30 and 40.

The test results indicate that the axial strength ratio gradually decreases with respect to increased slenderness ratios. The axial strength ratio of two-way action panels is generally larger than that of one-way panels by at least 2.4 to 3.5 times. Finally axial strength ratios are noticed to decrease with the number of opening in walls.

The Australian Standard (AS3600-2001) and the ACI code (ACI318-2002) wall equations were found to be inadequate for walls with openings, particularly with walls of high slenderness. In view of the significant shortcomings, a more appropriate formula was required for such walls.

Incorporating the test results from the present study and those of Saheb (1985), a new ultimate load formula was developed for reinforced concrete walls with and without openings. Comparisons with the available test results indicate that the new formula is accurate and slightly conservative.

Despite the conduct of extensive tests on wall panels and derivation of a reliable design formula, the research on high strength concrete wall panels with various openings remains relatively unexplored and needs more focused research in the future.

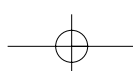


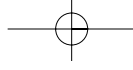
REFERENCES

- ACI 318 (2002). *Building Code Requirements for Reinforced Concrete ACI318-02*, American Concrete Institute, Detroit, pp. 111.
- BS8110-1997 (1997). *Structural Use of Concrete, Part 1 – Code of Practice for Design and Construction*, British Standards Institution, London, 1997.
- Doh, J.H. (2003). *Experimental and Theoretical Studies of Normal and High Strength Concrete Wall Panels*, PhD Thesis, Griffith University, Australia.
- Doh, J.H. and Fragomeni, S. (2005). "Evaluation of experimental work on concrete walls in one and two-way action", *Australian Journal of Structural Engineering*, Vol. 6, No. 1, pp. 37–52.
- Doh, J.H., Fragomeni, S. and Loo, Y.C.(2002). "Evaluation of experimental behaviour of normal and high strength reinforced concrete walls in two-way action", *Proceedings of the 17th Advanced in Mechanics of Structures and Materials*, Gold Coast, Australia, June, pp. 169–173.
- Fragomeni, S. and Mendis, P.A. (1997a), "Design equations for normal and high strength concrete walls", *Concrete in Australia*, Concrete Institute of Australia, North Sydney, Vol. 23, No. 3, pp. 22–24.
- Fragomeni, S. and Mendis, P.A. (1997b). "Instability analysis of normal- and high- strength reinforced-concrete walls", *Journal of Structural Engineering*, ASCE, Vol. 123, No. 5, pp. 680–684.
- Fragomeni, S. and Mendis, P.A. (1999). "Applicability of current ACI318 wall design formula for high strength concrete walls", *Advances in Structural Engineering*, Vol. 2, No. 2, pp. 103–108.
- Oberlender, G.D. (1973). *Strength Investigation of Reinforced Concrete Load Bearing Wall Panels*, PhD thesis, University of Texas, Arlington.
- Pillai, S.U. and Parthasarathy, C.V. (1977). "Ultimate strength and design of concrete walls", *Building and Environment*, London, Vol. 12, pp. 25–29.
- Saheb, S.M. and Desayi, P. (1989). "Ultimate strength of RC wall panels in one-way in-plane action", *Journal of Structural Engineering*, ASCE, Vol. 115, No. 10, pp. 2617–2617.
- Saheb, S.M. and Desayi, P. (1990). "Ultimate strength of RC wall panels in two-way in-plane action", *Journal of Structural Engineering*, ASCE, Vol. 116, No. 5, pp.1384–1402.
- Saheb, S.M. and Desayi, P. (1990b). "Ultimate strength of RC wall panels with openings", *Journal of Structural Engineering*, ASCE, Vol. 116, No. 6, pp. 1565–1578.
- Sanjayan, J.G. and Maheswaran, T. (1999). "Load capacity of slender high-strength concrete walls with side supports", *ACI Structural Journal*, Vol. 96, No. 4, pp. 571–576.
- Seddon, A.E. (1956). "The strength of concrete walls under axial and eccentric loads", *Symposium on Strength of Concrete Structures*, Cement and Concrete Association, May, London.
- Standard Australia. (2001). *Australian standard 3600, Concrete Structure*, Sydney.
- Swartz, S.E., Rosebraugh, V.H. and Berman, M.Y. (1974). "Buckling test of rectangular concrete panels", *American Concrete Institute Journal*, Vol. 71, pp. 33–39.
- Zielinski, Z.A., Troitski, M.S. and Christodoulou, H. (1982). "Full-scale bearing strength investigation of thin wall-ribbed reinforced concrete panels", *American Concrete Institute Journal*, Vol.79, pp. 313–321.
- Zielinski, Z.A., Troitski, M.S. and El-Chakieh, E. (1983). "Bearing capacity tests on precast concrete thin-wall ribbed panels", *Prestressed Concrete Institute Journal*, Chicago, Vol. 28, No. 3, pp. 89–103.



Jeung-Hwan Doh is an Associate Lecturer in the School of Engineering at Griffith University, Gold Coast Campus, Australia. He obtained Bachelor of Honours and Master of Honours in his civil engineering degree from University of Wollongong and obtained Ph.D. from Griffith University. His current research focuses on the reinforced concrete walls with and without openings using normal and high strength concrete.





J.H. Doh and S. Fragomeni



Sam Fragomeni is currently the Head of the School of Engineering at Griffith University Gold Coast, Australia. He obtained his Ph.D from The University of Melbourne, Australia in the area of concrete structures. His research interests include behaviour of concrete wall panels, high performance concrete and high-rise buildings. He is the author of over 60 journal/conference papers in these areas.

